CHAPTER 4

DISCUSSION

4.1 INTRODUCTION

The purpose of Chapter 4 is to analyse, interpret and discuss the experimental results from Chapter 3 against the literature background of Chapter 2.

The mechanical behaviour of particulate materials is controlled by the composition and state of the material, which are functions of the fundamental properties of the particles, the properties and state of interstitial fluids and the state of the packing arrangement, including density and structure. The composition and state of gold tailings are discussed here, based on the results from controlled laboratory experiments on reconstituted as well as undisturbed samples. The discussion leads to a working model of the condition of gold tailings in an impoundment and can be used as a guide to the geotechnical behaviour of this man-made soil.

The piezocone is already established as an important site investigation tool for the characterisation of tailings impoundments. It gives excellent results in defining the stratigraphy and seepage regime in a dam. However, it has not found favour in characterising strength and stiffness properties as yet. This chapter concludes with a re-evaluation of the interpretation of piezocone data in tailings. The ability of this tool to delineate the highly layered and variable tailings profile and to establish an accurate representation of seepage conditions is confirmed. In addition, the cone penetration data in a typical tailings profile are justified with respect to strength characteristics and shear behaviour. This will significantly extend the usefulness of piezocone data in tailings.

4.2 COMPOSITION

The solid phase of a soil plays a major part in determining its engineering behaviour. The most important soil solids are: minerals and products of organic synthesis and decay. Minerals are naturally occurring chemical compounds of definite composition and crystal structure. Tailings solids consist mainly of rock-forming minerals that have been liberated by mechanical and chemical processes in the reduction works, rather than the more natural processes of disintegration and decomposition.

The composition of gold tailings solids is discussed in the following sections in the context of the fundamental properties of the particles based on visual observations and x-ray spectrometry. The natural lighting effect on scanning electron micrographs, and the fact that the size fractions had been separated prior to imaging greatly facilitated interpretation. Energy dispersive x-ray spectrometry (EDS) and x-ray diffraction (XRD) techniques are used to identify the elemental and mineralogical composition of the material. It should be noted that quantitative information on the mineralogy is not as reliable as the qualitative information and should be regarded only as an estimate of the abundance of the various constituents.

4.2.1 Mineralogy

Particle mineralogy is derived from x-ray analyses: EDS spectrometry in the electron microscope for the elemental composition and XRD for the mineral composition. It should be noted that EDS targets are very small areas on the specimen, usually a particle and essentially gives a spot reading. XRD, however, illuminates a comparatively large area on a powdered specimen and therefore gives an estimate of the average mineralogy.

Energy Dispersive X-ray Spectrometry in the SEM

The original EDS spectra in Chapter 3 all show peak counts for carbon (C), oxygen (O) and gold (Au), which might be misleading to interpretation for the following reasons:

- Carbon is picked up from the pure carbon sticker used to mount a specimen onto the microscope stage without losing conductivity.
- X-ray emission can only take place from a solid phase and any oxygen detected must be from chemically bound oxygen, for example in the quartz as SiO_2 or from oxides of many of the metallic elements such as aluminium and iron.
- The gold peak originates from the thin coating of gold used to ensure a conductive surface and to enhance the imaging process.

The abundance of these elements has been ignored when quoting percentages of elements detected using EDS.

Some particles appear overexposed in the micrographs, this happens when a particle is loose or does not make good conductive contact with the microscope stage and becomes charged on its surface during imaging.

EDS is essentially a spot measurement and as such could only be applied to individual particles imaged under the SEM. For the purpose of discussion and in keeping with the classification of EDS data in Chapter 3, tailings particles are divided into sands and slimes with the following distinctions:

(a) *Tailings sands*

- Particles larger than 63 μm.
- Typically either smooth surfaced or rough surfaced bulky grains.

(b) *Tailings slimes*

- Particles smaller than 63 μm
- Exist predominantly as flaky or plate-like particles, either loose or aggregated into flocs.

The results of energy dispersive x-ray spectrometry on the Mizpah whole tailings sample are summarised in Table 4-1.

Table 4-1: Elemental composition of Mizpah whole tailings particles by EDS.

Element	Average Percentages Detected[1]				
	Smooth surfaced Sands	Rough surfaced Sands	Flaky Slimes	Slime Flocs	Hydrometer Precipitate
Na			1	1	43
Mg				2	
Al	2	21	12	17	
Si	93	64	75	57	1
P			1		15
S			1		31
K	1	10	4	6	
Ca			1	1	4
Fe		2	4	13	1
Other	2	2	2	3	4

Table 4-1 shows very high percentages of sodium, phosphor and sulphur detected in the precipitate, in other words in solution in the tailings water. However, it must be considered that the specimen was dispersed using Calgon ($NaPO_3$ and Na_2CO_3), which would have contributed to the abundance of some of these elements. The last column in Table 4-1 can be compared with the effluent analysis in Table 3-1 from Chapter 3, which confirms the high concentrations of sodium, calcium and chlorine in the tailings effluent.

[1] Note that individual columns do not add up to 100% because the numbers represent average percentages of each element detected in more than 100 individual EDS analyses.

Powder X-ray Diffraction

X-ray diffraction is probably the most reliable method for the identification of clay and other minerals and provides information on both the minerals and their abundance in a specimen. Table 4-2 summarises the properties of the main minerals identified with XRD in the tailings.

The silicates are the largest, and by far the most complicated class of minerals. Approximately 30% of all minerals are silicates and some geologists estimate that 90% of the earth's crust consists of silicates.

Tectosilicates are also known as "Framework Silicates" because their structure is composed of interconnected silica tetrahedrons going outward in all directions forming an intricate framework. In this subclass all the oxygens are shared with other tetrahedrons giving a silicon to oxygen ratio of 1:2. In the near pure crystalline state of only silicon and oxygen the mineral is quartz (SiO_2). Quartz is the most common earth mineral. It is hard and tough with no cleavage and resists mechanical weathering better than any other important rock mineral. It will be seen in the discussion that follows that the composition of tailings is also dominated by quartz. In fact, tailings sands consist almost exclusively of quartz grains. Quartz is slightly soluble in a basic environment.

In the Phyllosilicate subclass, rings of tetrahedrons are linked through shared oxygens to other rings in a two dimensional plane which produces a sheet-like structure. The typical crystal habit of this subclass is flat, platy, book-like and displays good basal cleavage. Typically, the sheets are then connected to each other by layers of cat-ions. Cat-ion layers are weakly bonded and often have water molecules and other neutral atoms or molecules trapped between the sheets. These complex aluminium and magnesium silicates are extremely fine grained, with large surface areas per unit mass. Probably all of them have definite crystal structures that include large numbers of atoms arranged in complex three-dimensional patterns and are electrically charged. Mica is a large group with nearly 30 members recognised, but only a few of which are common. Those few however make up a large percentage of the most common rock types found in the earth's crust. Micas often contain iron and magnesium in addition to potassium. Mica flakes are soft and resilient, with pronounced cleavage. They split easily and break to form still smaller, thinner flakes. The Chlorite group is often associated with the clay group due to the flat flakiness of the breakage particles. Muscovite, pyrophyllite, illite, clinochlore and kaolinite are all phyllosilicates with clay-like properties. When these minerals are present in a soil, they can radically change its mechanical behaviour. Although tailings are normally not thought of as clay-like materials, it will be shown that there are fairly high concentrations of phyllosilicates in the fines. These

Table 4-2: Properties of the principal minerals that are present in tailings.

Mineral	Formula	Classification	Habits	G_s	Mohs Hardness
Quartz	SiO_2	*Class:* Silicates *Subclass:* Tectosilicates *Group:* Quartz	*Crystalline:* Coarse - Occurs as well-formed coarse sized crystals. Fine - Occurs as well-formed fine sized crystals.	2.63	7
Muscovite - Barian	$(Ba,K)Al_2(Si_3Al)O_{10}(OH)_2$	*Class:* Silicates *Subclass:* Phyllosilicates *Group:* Mica	*Massive:* Lamellar - Distinctly foliated fine-grained forms. Foliated - Two dimensional platy forms. Micaceous - Platy texture with "flexible" plates.	2.82	2 - 2.5
Pyrophyllite	$Al_2Si_4O_{10}(OH)_2$	*Class:* Silicates *Subclass:* Phyllosilicates *Group:* Pyrophyllite-talc	Earthy - Dull, clay-like texture with no visible crystalline affinities.	2.84	1.5 - 2
Illite	$(K,H_3O)Al_2Si_3AlO_{10}(OH)_2$	*Class:* Silicates *Subclass:* Phyllosilicates *Group:* Mica		2.75	1 - 2
Clinochlore	$(Mg,Fe)_6(Si,Al)_4O_{10}(OH)_8$	*Class:* Silicates *Subclass:* Phyllosilicates *Group:* Chlorite	*Massive:* Fibrous - Distinctly fibrous fine-grained forms. Pseudo hexagonal - Crystals show a hexagonal outline. Granular - Generally occurs as anhedral to subhedral crystals in matrix.	2.65	2 - 2.5

Kaolinite	$Al_2Si_2O_5(OH)_4$	*Class:* *Subclass:* *Group:*	Silicates Phyllosilicates Kaolinites	Earthy - Dull, clay-like texture with no visible crystalline affinities.	2.60	1.5 - 2
Gypsum	$CaSO_4 \cdot 2H_2O$	*Class:*	Sulphates	*Tabular:* Form dimensions are thin in one direction. *Crystalline:* Occurs as well-formed coarse sized crystals. *Massive:* Distinctly fibrous fine-grained forms.	2.30	2
Pyrite	FeS_2	*Class:* *Group:*	Sulphides Pyrite	*Striated:* Faces have parallel lines (e.g. plagioclase). *Druse:* Crystal growth in a cavity which results in numerous crystal tipped surfaces. *Stalactitic:* Shaped like pendant columns as stalactites or stalagmites.	5.01	6.5

will surely be affected by electrical surface charges and other similar effects associated with clays.

The members of the Sulphide Mineral Class form an economically important class of minerals. Most major ores of important metals such as copper, lead and silver are sulphides. Strong generalities exist in this class. The majority of sulphides are metallic, opaque, generally sectile, soft to average in hardness and possess high densities. The Pyrite Group is composed of minerals with a similar isometric structure and related chemistry. It is named after its most common member, pyrite, which is often associated with gold in South African ores. The significance of pyrite in the tailings mix, is the high density of this mineral, of the order of 5 Mg/m^3, and the fact that it reduces to sulphuric acid and iron oxides following oxidation. The oxidation of the sulphide minerals is clearly evident as the yellowish coloration on exposed surfaces on gold tailings impoundments. Changes in pH as a result of the sulphuric acid can lead to precipitation of agents such as silica, but also to a solution of heavy minerals that can cause serious pollution problems.

Contrary to the Mizpah whole tailings specimen used for the EDS work in the previous section (Table 4-1), none of the XRD specimens were treated with a dispersant. Gypsum ($CaSO_4{\cdot}2H_2O$) identified in the Mizpah whole tailings (delivery slurry) and not in the other samples, is probably the result of precipitation of gypsum from solution when the sample was dried. Gypsum was not detected in the samples recovered from the pond areas on the dam probably because it was either siphoned of with the return water or leached out without opportunity to precipitate. Analysis of the hydrometer precipitate in Table 4-1 confirms the presence of Ca and S in the delivery slurry, because these are not found in the dispersing agent.

Discussion

The results of the EDS and XRD analyses are used in this section to determine the mineralogical composition of the tailings samples considered in this study. Although EDS data are quantitative with respect to the elements identified, XRD data are not intended to be quantitative. Nevertheless, in the discussion that follows the XRD data are quantitatively analysed, in an approximate way, and compared with the EDS results. The abundance of minerals in the XRD spectrographs was estimated by adding all the peak counts, percentage-wise, of a specific mineral. The results are summarised in Table 4-3.

Table 4-3 represents the mineralogical composition of the tailings, including that of the dispersed whole tailings sample. To compare these results with the EDS data on the Mizpah

Table 4-3: Approximate mineral percentages from XRD results.

Approximate percentages identified with XRD					
All samples: non-dispersed.					
Sample[2]	**Mizpah Whole**	**Mizpah Fine**	**Mizpah Coarse**	**Pay Dam Fine**	**Pay Dam Coarse**
Quartz	69	75	83	59	79
Muscovite	8	9	8	7	9
Pyrophyllite	5	4	1	17	5
Illite	6	6	5	11	3
Clinochlore	2	3	2	3	3
Kaolinite	2	2	1	3	1
Gypsum	7				
Pyrite				1	
Mizpah whole tailings: separated fractions and dispersed.					
	150 μm (*Sand*)	**75 μm (*Sand*)**	**10 μm (*Slime*)**	**2 μm (*Slime*)**	**1 μm (*Slime*)**
Quartz	90	89	72	43	27
Muscovite			9	21	31
Pyrophyllite			1	4	9
Illite	10	8	9	19	15
Clinochlore		1	4	7	9
Kaolinite		1	3	4	7
Pyrite			1	2	1

whole tailings sample in Table 4-1, the XRD minerals for the sand (150 & 75 μm) and slime (8, 2 & 1 μm) fractions have to be broken down into their constituent elements. This is shown in Table 4-4, where the Mizpah whole tailings minerals are represented by their characteristic elements (Mg, Al, Si, S, K, & Fe) as follows:

- Quartz SiO_2 Si = 1
- Muscovite $KAl_2(Si_3Al)O_{10}(OH,F)_2$ K : Al : Si : F = 1 : 3 : 3 : 2
- Pyrophyllite $Al_2Si_4O_{10}(OH)_2$ Al : Si = 2 : 4
- Illite $(K,H_3O)Al_2Si_3AlO_{10}(OH)_2$ K : Al : Si = 1 : 3 : 3
- Clinochlore $(Mg,Fe)_6(Si,Al)_4O_{10}(OH)_8$ Mg : Fe : Si : Al = 6 : 6 : 4 : 4
- Kaolinite $Al_2Si_2O_5(OH)_4$ Al : Si = 2 : 2
- Pyrite FeS_2 Fe : S = 1 : 2

[2] The terms fine and coarse should not be confused with tailings sands and tailings slimes. The coarsest sample studied contained more than 50% slimes and the finest sample close to 5% sand.

Table 4-4: Mizpah Whole tailings: XRD mineral identification and breakdown of characteristic elements.

Mineral composition: Sand (avg. of 150 & 75 μm) and Slime (avg. of 8, 2 & 1 μm).							
	Quartz	**Illite**	**Clinochlore**	**Kaolinite**	**Muscovite**	**Pyrite**	**Pyrophyllite**
Sand	89	9	1	1			
Slime	47	14	7	5	20	1	6
Ratios of characteristic elements for each mineral from the chemical formulae.							
Mg			0.3				
Al		0.43	0.2	0.5	0.43		0.33
Si	1.0	0.43	0.2	0.5	0.43		0.67
S						0.67	
K		0.14			0.14		
Ca							
Fe			0.3			0.33	
Percentages of characteristic elements in the separate sand and slime fractions.							
Sand:							
Mg			0.15				
Al		3.86	0.10	0.25			
Si	89.00	3.86	0.10	0.25			
S							
K		1.29					
Ca							
Fe			0.15				
Sum	**89.0**	**9.0**	**0.5**	**0.5**	**0.0**	**0.0**	**0.0**
Slime:							
Mg			2.10				
Al		6.00	1.40	2.5	8.57		2.00
Si	47.00	6.00	1.40	2.5	8.57		4.00
S						0.67	
K		2.00			2.86		
Ca							
Fe			2.10			0.33	
Sum	**43.0**	**14.0**	**7.0**	**5.0**	**20.0**	**1.0**	**5.0**

Once the minerals in the sand and slime fractions have been broken down into their constituent elements, a direct comparison can be made with the EDS data, as shown in Table 4-5.

Table 4-5: Comparison of XRD and EDS mineral composition of Mizpah whole tailings.

	SAND			SLIME		
	EDS		XRD	EDS		XRD
Element	**Smooth**	**Rough**		**Flake**	**Flock**	
Mg					2	2
Al	2	21	4	12	17	20
Si	93	64	93	75	57	70
S				1		1
K	1	10	1	4	6	5
Ca				1	1	
Fe		2		4	13	2
Other	4	3		3	4	
Sum	**100**	**100**	**98**	**100**	**100**	**100**

Table 4-5 demonstrates good agreement between the XRD and EDS mineral compositions of the dispersed tailings sands and slimes. Based on these results the mineralogical composition of the gold tailings sands and slimes is as follows:

- *Tailings sands:*

  90% quartz and 5 to 10% illite, with illite mainly detected on rough surfaced particles. It might be conjectured that the rough surfaced particles are pyrophyllite, which would also register silicon and aluminium on the EDS. However, the powder XRD did not detect any pyrophyllite in the 150 and 75 μm Mizpah whole tailings specimens. It is, therefore, concluded that the clay mineral in the sands was illite.
- *Tailings slimes:*

  50% quartz, 20% muscovite, 15% illite, and approximately 5% each of clinochlore, kaolinite, pyrophyllite.

These percentages, although approximate, give insight into the mineralogy of a typical gold tailings. The coarse particles are basically pure quartz except for a small percentage of illite clay, most probably attached to the quartz grains. The slimes, although predominantly quartz, have increasing amounts of pyrophyllite, muscovite and illite, as well as traces of kaolinite, and pyrite. The muscovite could actually be seen glittering in the slimes when held in a bright light.

Thus, referring to the XRD results on the five tailings samples examined, the mineralogy of gold tailings, i.e. delivery slurry as well as fine and coarse deposited layers can be summarised as:

- Quartz (%)[3]: 83 - 75 - 59
- Muscovite (%): 9 - 8 - 7
- Pyrophyllite (%): 17 - 5 - 1
- Illite[4] (%): 11 - 5 - 3
- Small percentages of Clinochlore, Kaolinite and Pyrite.

These percentages are also in good agreement with the composition of the Witwatersrand gold reef according to Stanley (1987); Table 2-1 from Chapter 2 reproduced here as Table 4-6.

Table 4-6: Mineral composition of a typical Witwatersrand gold reef.

Mineral	Abundance
Quartz (SiO_2), primary and secondary	70 - 90 %
Muscovite and other phyllosilicates	10 - 30 %
Pyrites	3 - 4 %
Other sulphides	1 - 2 %
Grains of primary minerals	1 - 2 %
Uraniferous Kerogen	1 %
Gold	~ 45 ppm in the Vaal Reef

4.2.2 Specific Gravity

Table 4-7: Specific gravity of the tailings samples.

Tailings Dam	Description	G_s
Mizpah	Whole tailings	2.74
	Pond fine	2.75
	Pond coarse	2.73
Pay Dam	Penstock fine	2.74
	Penstock coarse	2.73

Table 4-7 shows a very interesting feature of the different tailings samples considered: they have almost exactly the same specific gravity of 2.74 Mg/m^3. This observation suggests that soil-forming processes on a gold tailings impoundment are possibly driven by gravity - at least in the pond areas and in the delivery pulp. The delivery pulp contains specific percentages of quartz, muscovite, pyrophyllite, illite and traces of other minerals with their

[3] The percentages give the full range based on the XRD results and an average value for the 5 samples tested.

[4] Compared with the separated sands and slimes there appears to be a deficiency in illite in the whole samples, however, most of the illite was detected in sand and slime particles larger than 150 μm and smaller than 2 μm. In the full gradings the bulk of the material lies between these sizes with much less illite detected, refer to Table 4-3.

respective densities, but with an overall specific gravity of 2.74. It is speculated here that soil-forming processes on the impoundment will sort these fractions gravitationally, rather than according to size, so that the mixtures of minerals in deposited layers, whether fine or coarse, result in a specific gravity of 2.74 Mg/m^3 for each layer.

4.2.3 Grading

Grading analyses were performed using standard wet-sieve and hydrometer methods. It will be illustrated that the properties of the tailings particles affect the results and produce grading curves which are consistently finer than estimates of particle dimensions based on the electron micrographs.

Effects of Pre-treatment Procedures:

In order to judge the effects of different pre-treatment procedures on the grading distributions, parallel tests were performed with non-treated specimens as a reference. Conclusions from these tests are subsequently discussed:

(a) *Calcareous compounds*

A simple test for calcareous matter, using hydrochloric acid, was performed on the samples as suggested in Chapter 3. None of the samples recovered from the dams showed any response, indicating the absence of carbonate compounds. It is concluded that carbonate bonding does not occur in tailings deposits.

(b) *Dispersion*

Treatment with the standard dispersing agent, Calgon, altered the gradings significantly throughout. With dispersant the gradings are fairly uniformly distributed in the fine sand and silt size ranges with approximately 10% finer than 2 μm. Without dispersant none of the gradings identified any material finer than medium silt sized particles, approximately 10 μm. There are two possible explanations for this behaviour:

- Tailings exist in a flocculated state on leaving the reduction plant, wherein most of the finer material is aggregated into flocs no less than 10 μm in diameter. These flocs are broken down into their individual constituents by the dispersing agent and can be detected in the hydrometer sedimentation test.
- Alternatively the fine flaky slimes attach themselves to the bulky sand and coarse silt particles in the undispersed state. The slimes essentially blanket the coarser particles and settle out together with them. On dispersion the flakes are detached from the coarser particles and individually measured during sedimentation.

Evidence on the electron micrographs, suggests that a combination of both these effects is responsible for the flocculated nature of untreated gold tailings. Figure 4-1 demonstrates an example of each effect on micrographs of untreated (undispersed) tailings from this study.

(c) *Organic matter*

The British standard provides for the removal of organic matter by oxidation using hydrogen peroxide. Organic removal was considered for this study due to the occurrence of wood fibres in the tailings. Tramp pieces of wood are processed with the mined ore, and end up as fine fibres in the tailings. Reaction of the oxidising agent with the samples were highly dependent on the age of the tailings. Fresh tailings produced a violent and rapid reaction, whereas older tailings resulted in a slow and subdued reaction. The fresh tailings were sampled directly from the discharge pipe or from surface deposits and the older material at depth below the surface of a deposit. The author is of the opinion that the hydrogen peroxide not only oxidises organic material, but also sulphide minerals such as pyrite. Visual observation after organic treatment showed that there were still some wooden fibres left, although, much reduced. For these reasons it is not recommended to use organic pre-treatment on tailings. Nevertheless, organic treatment on undispersed tailings result in grading curves somewhere between fully dispersed and non-dispersed gradings as illustrated in Table 4-8.

Table 4-8: Effects of organic treatment on fresh and older tailings specimens.

Tailings Dam	Description	Age (Depth of sampling)	Result
Mizpah	Pond Fines	Surface	Violent reaction No change in grading
	Pond Coarse	Surface	Violent reaction Intermediate grading
Pay Dam	Penstock Fines	4.3 m below surface	Mild reaction Intermediate grading
	Penstock Coarse	2.0 m below surface	Moderate reaction Intermediate grading

Fully Dispersed Gradings

The gradings performed on all samples in this study indicated very little, less than 2% per mass, coarser than 200 μm (limit of fine sand) and generally of the order of 10% smaller then 2 μm (clay sized). The remaining material was distributed in the silt and fine sand size ranges as summarised in Table 4-9.

Table 4-9: Summary of dispersed grading properties of gold tailings.

Tailings Dam	Description	Median Particle Size D_{50}		*CU*	*CC*
		(μm)	Description		
Mizpah	Whole Tailings	30	Coarse silt	28	0.9
	Pond Fines	10	Medium silt	11	0.8
	Pond Coarse	60	Coarse silt	25	2.8
Pay Dam	Penstock Fines	6	Fine Silt	5	0.8
	Penstock Coarse	25	Coarse silt	22	0.9

The gradings in Table 4-9, although representative of the pond areas of tailings dams, almost cover the full range of published gradings in Figure 2-11. The Coefficient of Uniformity (*CU*) is less than 36 throughout, which is the value for the "Ideal" Fuller Curve (Fuller & Thompson, 1907). The Fuller curve describes the uniformity properties of spherical particles for the densest possible state of packing; the largest particles barely touch each other, while there are enough intermediate-size particles to occupy the voids between the largest without holding them apart, smaller particles subsequently occupy voids between intermediate-sizes, etc. A sample of spherical particles with *CU* less than 36 has an abundance of fines so that the coarser particles cannot all be in contact and it is less dense than the optimum density. If *CU* is greater than 36, the voids between coarser particles are not fully occupied by the fines; the density is therefore also lower than the optimum density. It is interesting to note that:

- $CU < 36$: Addition of a small amount of fines will result in a less dense packing arrangement as the ideal structure, $CU = 36$, is further disrupted.
- $CU > 36$: Addition of a small amount of fines will result in a more dense packing arrangement, until void spaces between the coarser particles are filled ($CU = 36$).

The fine graded specimens in this study all have an abundance of fines, which tend to coat and push the coarser particles apart. The behaviour of these materials should, therefore, be governed by the fines fraction. The coarser grades, especially the whole tailings mix, are much closer to, but still less than the Fuller value (22, 25 and 28 compared with 36). It could, therefore, be argued that all of the samples shown in Table 4-9 will generally reduce in stiffness as their fines content increases. The soil structure of these tailings is controlled by the fines rather than by a skeleton of coarser particles as suggested by the following evidence:

- Coefficients of Uniformity are less than 36 throughout. Although tailings particles are far from spherical, it is assumed that the basic principals stated in the previous paragraph still hold.
- All the gradings have more than 50% passing 63 μm, and of the order of 10% passing 2 μm, where the fines contain significant amounts of clay minerals. Thus, even the coarser samples contain more than 50% slimes.
- Micrographs of undisturbed samples shown in Chapter 3 show an abundance of fines so that the coarser bulky grains are always covered and displaced by these.
- In Section 4.3.3 it will be proved that the shear strength of all samples studied is unaffected by grading. This is consistent with a soil structure that is controlled by the fines.

Soils of this nature would have their compression curves displaced towards higher void ratios in e:log*p*' space with the addition of more fines. This is confirmed in Section 4.3.1, where samples have elevated and more steep compression curves with increasing fines content. As the compression curves are raised they also become more steep, thus reducing the bulk stiffness of the sample.

Sieve and Hydrometer Grading vs. "Visual" Grading

A "visual"-grading was derived from dispersed Mizpah whole tailings based on the electron micrographs presented in Chapter 3. SEM specimens were collected for this purpose from the sieve and hydrometer sedimentation tests. During the sedimentation test the sediment level was marked for each timed reading of the hydrometer. When the test had been completed, representative specimens were carefully extracted from each of these layers. Each specimen, representative of a specific size fraction, was then imaged under the SEM. The average diameter of the particles on each micrograph was subsequently measured, and applied to the mass fraction represented by the specimen photographed. For example, the fraction retained on the 150 μm sieve, with a measured diameter of approximately 200 μm, represents 4% of the total mass of the sample, 95.5% having passed this sieve the other 0.5% retained by sieves with a larger aperture. This process was greatly simplified by the fact that the original sample was already separated into size fractions by the sieve and sedimentation tests.

Figure 4-2 compares the grading determined from sieve and hydrometer tests to that derived from the SEM micrographs. The SEM "visual" grading, although approximate, is consistently coarser than the grading predicted by sieve and hydrometer tests. The reasons for this are two-fold:

- Test sieves in the sand-size range all have square apertures, where the sieve size measures the side length of an aperture. For a 150 μm sieve the diagonal across the square opening would be 212 μm thus allowing a flat or elongated particle of about 210 μm to pass through the sieve. The result is that the 4.5% mass fraction smaller than 150 μm, according to the sieve analysis, is actually representative of particles smaller than approximately 200 μm, due to the flattened nature of these particles.
- The hydrometer sedimentation test is interpreted using Stokes' Law after Sir George Stokes (1891), which among other things calculates the terminal velocity of the particles assuming small spheres. Electron micrographs of the particles smaller than 63 μm clearly show that these are plate-like and flat and would have a much reduced terminal velocity compared with a spherical particle of the same mass. Hence the "visual" coarser grading compared with the grading calculated using Stokes' theory.

4.2.4 Particle Shape

The shape of soil particles is as important as their size-range in determining the engineering behaviour of the material.

Tailings sands are bulky particles. The smooth surfaced grains imaged on SEM micrographs are highly angular to angular and generally flattened, sometimes elongated with sharp edges. Rough-surfaced sand particles are more rounded and can be sub-angular to sub-rounded. These observations are consistent with the products of rock crushing and grinding; the initial product is angular, but becomes sub-angular as the sharpest edges are smoothed by subsequent action.

Angularity has a profound influence on engineering behaviour. Under load, angular corners can break and crush, but particles tend to resist shear displacement. More rounded particles are less resistant to displacement, but less likely to crush. Soils composed of angular bulky grains are capable of supporting heavy static loads with little deformation. However, vibration and shock cause loose arrangements of angular bulky grains to be displaced easily. Such behaviour may be expected from clean tailings sands, which in addition should be non-plastic. Angularity also has the effect of increasing the angle of internal friction, ϕ' (Mittal & Morgenstern, 1975).

Tailings slime particles are generally flaky grains consisting of disintegrated mica, clay and quartz minerals with very sharp edges. The slimes contain some silt-sized quartz particles. These small but bulky grains have similar properties to the smooth surfaced sands. Compared with the bulky sands, the slimes should be more compressible and behave like an intermediate plasticity clay. The fine sample collected from the Pay Dam penstock site consisted of at least 90% slimes and had a PI of 17%. The slimes particles will be much more susceptible to surface and electromagnetic forces than the body force of gravity, which is the predominant force acting on the sand particles. The fact that the slimes can be flocculated is evidence of the effects of the surface forces.

4.2.5 Surface Texture

Inspection of the electron micrographs presented in Chapter 3 reveals the coarser or sand tailings particles to exist either with smooth or rough and irregular surfaces. The surfaces on the smooth sands appear to be the result of splitting and breaking of larger particles in the crushing and grinding processes of comminution. These surfaces show the typical concave geometry of pure quartz when broken. On the other hand, sand particles with irregular surfaces may have formed by fines attaching themselves to the particles and/or as a result of shattering and chipping of the particle surface rather than splitting during the reduction process. Evidence from EDS spectrometry on the rough surfaced particles indicates that there are at least some clayey minerals (illite) present on these otherwise pure quartz particles.

Individual particles of tailings slimes smaller than 63 μm have very smooth and flat surfaces. However, even dispersed there were some agglomerations and flocs of these flakes, which as a whole present a rough and irregular surface.

4.2.6 Summary of composition

The composition of gold tailings, as determined from the samples considered in this study, is summarised in Table 4-10. Although the terms sand and slime are used to distinguish between the coarser bulky tailings particles and the fines, there is no definite size separation between the two. For the tailings samples studied in this project the properties of the particles gradually change from sands to slimes in the fine sand to coarse silt-size ranges.

4.3 STATE

4.3.1 Normalised Compression Behaviour

Background

The idea of normalising the compression behaviour of a soil and linking it to commonly used geotechnical parameters such as the Atterberg limits is not a new one. Such

Table 4-10: Summary of fundamental particle properties of gold tailings.

Property	Tailings Sands	Tailings Slimes
Mineralogy	+90 % Quartz up to 10 % Illite	45 % Quartz 20 % Muscovite 15 % Illite 20 % Pyrophyllite, Kaolin & Clinochlore
	The mineralogy of gold tailings consists almost entirely of Tectosilicates and sheet-like Phyllosilicates. The coarser particles are mostly pure silica quartz, but as the fineness increases more and more mica-clay minerals are present. In the case of Mizpah dam the virgin slurry delivered contained some 40% tailings sand and 60% slime. Depositional conditions and sorting processes on the impoundment will result in deposited layers with a coarser composition - more sand less slime, or finer composition - less sand, more slime. Typically at the beach pond interface coarser layers contain 50% sand and slime, whereas the finer layers have 10% sand and 90% slime.	
Grading	The grading of gold tailings is highly dependent on whether a dispersing agent is used or not. *Dispersed:* Uniformly distributed in the fine sand and silt size ranges with approximately 2% coarser than 200 μm and 10% finer than 2 μm. *Non-dispersed:* Fines either flocculated (> 10 μm) or attached to coarser particles.	
Particle shape	Bulky but flattened.	Some silt sized particles similar to the sands, but mostly thin plate-like flakes, with high aspect ratios and large specific areas.
	Highly angular to sub-rounded.	Naturally flocculated.
Particle surface texture	Ranges from smooth to rough.	

normalising techniques and empirical correlations have been published for natural clay sediments for example by Schofield and Wroth (1968) and by Burland (1990).

To clarify the terms and definitions used in this discussion a brief review of critical state soil modelling will be given. For a more detailed treatment of the subject reference should be made to: Schofield and Wroth (1968), Atkinson and Bransby (1978) and Muir Wood (1990). Roscoe, Schofield and Wroth developed Critical State Soil Mechanics (CSSM) at Cambridge University in the 1950's to model soil behaviour theoretically. Critical state by definition implies an ultimate failure state, where a soil mass is deforming in shear strain at constant shear strength, volume and effective stresses. The steady state, later defined by Casagrande (1969), differs from the critical state in that the failing mass, in addition, has to deform at constant velocity after particle orientation has reached a statically steady state condition and after all particle breakage if any is complete. There is some uncertainty in the literature about whether the critical state (CS) and steady state (SS) describe different conditions in a failing soil mass; Poulos et al. (1985), Poorooshasb (1989) and Ishihara in his 1993 Rankine lecture propose that both define the same ultimate state, but Castro (1969) and Alarcon-Guzman et al. (1988) disagree. Nevertheless, the term critical state will be used hereafter to indicate a state of ultimate failure.

In the theory of CSSM a set of invariant and fundamental soil parameters are used, which is entirely dependent on soil composition and independent of soil state and loading conditions. These parameters can, therefore, be derived from simple laboratory tests on reconstituted samples. The effects of soil state and the loading conditions are then modelled mathematically using well known constitutive relationships for soils including linear elastic theory, Cam-Clay (Schofield & Wroth, 1968), Modified Cam-Clay (Roscoe & Burland, 1968), Nor-Sand (Jefferies, 1993), etc.

Critical state stress paths are usually represented in two-dimensional invariant space consisting of a *Stress Plane* (q' vs. p') and a *Compression Plane* (e vs. p' or lnp') with,

- q', the deviator stress, equal to $(\sigma'_1 - \sigma'_3)$ or $\frac{3}{\sqrt{2}}\tau'_{oct}$ for the triaxial test (τ'_{oct} = effective octahedral shear stress).
- p', the mean normal effective or isotropic stress, equal to the effective octahedral normal stress, σ'_{oct} or $\frac{1}{3}(\sigma'_1 + 2\sigma'_3)$ for the triaxial test.
- e, the void ratio, is a measure of the density state[5].

Two reference soil states can be represented as logarithmic functions on these planes, i.e. the isotropic normally consolidated state or line (NCL) and the critical state failure line (CSL). These lines are functions of the following fundamental critical state parameters:

- λ – slope of the isotropic NCL in the compression plane.
- κ – slope of an isotropic rebound curve or swelling line (SL) in the compression plane.
- N – intercept of the NCL on the compression plane at p' = 1 kPa.

[5] Some researchers prefer using specific volume as a normalised density parameter, where specific volume is the volume of a sample containing a unit volume of soil solids. However, numerically specific volume is equal to 1 + e.

- M – slope of the CSL in the stress plane, measures frictional properties of the material.
- Γ – Similar to N, locates the CSL in the compression plane at p' = 1 kPa.

Using these parameters the NCL and CSL become:

NCL: $e = N - \lambda \ln p'$ and $q' = 0$ **Eq. 4-1**

CSL: $e = \Gamma - \lambda \ln p'$ and $q' = M \cdot p'$ **Eq. 4-2**

Schofield and Wroth (1969) noted that when the experimental log-linear CSL's of five different clays were extrapolated in the compression plane they appeared to converge at a focus point, Ω, with e_Ω = 0.25 and p'_Ω = 10.3 MPa, see Figure 4-3a. Such an extrapolation is certainly practically unjustified because of particle fracture and degradation at high pressure as well as the fact that the CSL must become asymptotic to the stress axis at zero void ratio. However, this geometric extrapolation allows some interesting analyses to be carried out, assuming that both the NCL and CSL's are theoretically parallel. Schofield and Wroth showed that points on the CSL corresponding to the liquid limit (LL) and the plastic limit (PL) of each individual clay tend to gather around the mean normal effective stresses of p' = 5.5 kPa and 550 kPa respectively; in other words $p'_{PL} \approx 100.p'_{LL}$. When the failure data are re-plotted as Liquidity Index, I_L vs. lnp', all CSL's collapse into a unique normalised line through the Ω-focus (Figure 4-3b), with the equation:

$$I_L = 1.371 - 0.217 \ln(p') \quad \textbf{Eq. 4-3}$$

where

$$I_L = \frac{w - PL}{LL - PL} \quad \textbf{Eq. 4-4}$$

w = moisture content

LL = liquid limit

PL = plastic limit

Thus, for any one soil,

$$e_{PL} - e_\Omega = \lambda \ln \frac{p'_\Omega}{p'_{PL}} \quad \textbf{Eq. 4-5}$$

Substituting the co-ordinates of Ω and p'_{LL} = 551.6 kPa into Eq. 4-5,

$$e_{PL} - 0.25 = \lambda \ln \frac{10342}{551.6} = 2.93\lambda \quad \textbf{Eq. 4-6}$$

or

$$\lambda = 0.341(e_{PL} - 0.25) \quad \textbf{Eq. 4-7}$$

Assuming G_s = 2.7 results in an approximate relation with the PL of

$$\lambda = 9.2 \cdot 10^{-3}(PL - 9.26) \qquad \textbf{Eq. 4-8}$$

Using the same arguments it can also be proved that,

$$\lambda = 3.6 \cdot 10^{-3}(LL - 9.26) \qquad \textbf{Eq. 4-9}$$

According to Schofield and Wroth, the better correlation with experimental results were obtained with Eq. 4-8.

The value of, λ, i.e. the slope of the CSL, is theoretically also the slope of the NCL as shown in Eqs. 4-1 and 4-2. If a similar focus to the Ω-point exists for normal compression, the properties of this focus together with Eqs. 4-8 and 4-9 could be used to reconstruct the compression behaviour of a clay.

Burland explored the normalised compression behaviour of natural clays. He proposed using the Void Index, I_v, as a normalising parameter for one-dimensional compression behaviour with,

$$I_v = \frac{e - e_{100}}{e_{100} - e_{1000}} \qquad \textbf{Eq. 4-10}$$

where e = void ratio

e_{100} = void ratio at σ'_{vo} = 100 kPa

e_{1000} = void ratio at σ'_{vo} = 1000 kPa

The void index serves as a measure of the compactness of a reconstituted clay and collapses e : logσ'_{vo} curves for different clays onto a single line called the Intrinsic Compression Line (ICL), Figure 4-4. This unique ICL confirms the assumption that the compression behaviour of a clay is truly log-linear as suggested by Eq. 4-1. The sign of the void index can be linked to the state of the material as follows: a "+" void index indicates a compact sediment and a "-" void index, a loose sediment. The intrinsic constants of compressibility, e_{100} and e_{1000}, can be linked to the critical state compressibility, λ, and clearly has a close analogy to the Liquidity Index.

Burland compared the intrinsic ICL to the Sedimentation Compression Line (SCL), representing the normalised in-situ state of a clay, to illustrate the effect of in-situ structure. The SCL consistently lies above the ICL for normally consolidated clays; a measure of the enhanced resistance of a naturally deposited clay over a reconstituted one resulting from differences in fabric and bonding (i.e. structure). The influence of structure on the compressibility or stiffness of clays was first recognised by Terzaghi (1941) and later confirmed by Skempton (1944). At pressures in excess of 1000 kPa the ICL and SCL tend to converge as the natural material is de-structured.

The ICL for three natural clays covering a wide range of liquid limits and pressures was found to be represented with sufficient accuracy by the following equation:

$$I_v = 2.45 - 1.285\log\sigma_v^{'} + 0.015\left(\log\sigma_v^{'}\right)^3 \qquad \textbf{Eq. 4-11}$$

where $\sigma_v^{'}$ = vertical effective pressure in kPa

It was also found that for soils lying above the A-Line on the Casagrande chart, I_v could be correlated to the void ratio at liquid limit using:

$$I_v = \frac{e - 0.109 - 0.679e_{LL} + 0.089(e_{LL})^2 - 0.016(e_{LL})^3}{0.256e_{LL} - 0.04} \qquad \textbf{Eq. 4-12}$$

where e_{LL} = void ratio at the liquid limit

or by substituting $G_s = 2.7$ as Schofield and Wroth did,

$$I_v = \frac{e - 0.109 + 0.018LL - 64.9\cdot10^{-6}LL^2 + 0.315\cdot10^{-6}LL^3}{0.007LL - 0.04} \qquad \textbf{Eq. 4-13}$$

Eq. 4-13 only holds for clays with $26 < LL < 160$. Statistical analysis has shown that equally good correlations are achieved using plasticity index for high plasticity clays, but that errors become significant in low plasticity clays. According to Burland the normalisation did not fit soils lying below the A-Line on the Casagrande chart well.

Normalising the Compression Behaviour of Gold Tailings

In the previous section two methods of normalising the compression behaviour of natural clays were reviewed. These methods employ either the liquidity index or the void index, and propose simple correlations with the liquid and plastic limit through which the compression behaviour of a clay may be modelled. The question remains whether similar techniques can be applied to silty materials, such as tailings, which generally lie below the A-Line on the Casagrande chart. If successful, information on the state of density and bulk stiffness in a tailings profile, will be extremely useful, for example in calculating storage volumes, etc.

Figure 4-5 reproduces the compression data of the five tailings samples considered in this study, the properties of which are summarised in Table 4-11. In Figure 4-6 the compression data is geometrically extrapolated in a similar fashion to the extrapolation of the CSL by Schofield and Wroth. The equations for these extrapolated lines are, in fact, the normal consolidation lines for each sample, expressed in terms of the parameters λ and N

Table 4-11: Atterberg limits and critical state compression parameters for Mizpah and Pay Dam tailings.

Description	LL	e_{LL}	p'_{LL}	PL	e_{PL}	p'_{PL}	PI	CU	λ	N
Mizpah										
Whole tailings	29	0.790	71	22	0.600	1350	7	27.5	0.066	1.082
Pond Fine	43	1.183	14	32	0.880	400	11	10.7	0.1053	1.512
Pond Coarse	28	0.764	5	22	0.601	320	6	25	0.0447	0.856
Pay Dam										
Penstock Fine	56	1.534	27	39	1.069	400	17	4.9	0.1760	2.126
Coarse	29	0.792	35	22	0.601	600	7	21.5	0.0685	1.040

as given in Table 4-11. There is some convergence of the extrapolated NCL's to an unique focus point, at Ω_{NCL} = (20 000 ; 0.41). Also shown on Figure 4-6 is the location of the liquid and plastic limits of each tailings sample. The liquid limits are rather loosely scattered around a mean normal effective stress of 20 kPa, and the plastic limits around 430 kPa. Normalisation using the liquidity index relies on the uniqueness of these pressures.

Using the Ω_{NCL}-focus together with the 20 kPa and 430 kPa liquid limit and plastic limit stresses, the compression curves of the tailings should be normalised in the Liquidity Index, I_L (Schofield & Wroth, 1968). Alternatively the method proposed by Burland (1990), using the Void Index, I_V, can be used, which relies on the log-linear relationship of the compression curves. The application of these two techniques on the tailings samples are summarised in Table 4-12. The poor fit of the test data to the normalised lines at low stresses is due to sample preparation disturbance and bedding effects.

Table 4-12: Normalisation of compression data.

Normalisation		**Equation of normalised line**	**Figure**
Liquidity Index	$I_L = \dfrac{w - PL}{LL - PL}$	$I_L = 2 - \ln(p')/3$ $w = 100e/G_s$	Figure 4-7
Void Index (e_{100} & e_{1000})	$I_V = \dfrac{e - e_{100}}{e_{100} - e_{1000}}$	$I_V = 2 - \ln p'/2.303$	Figure 4-9
Void Index (N & λ)	$I_v = \dfrac{e - N}{\lambda}$	$I_v = -\ln p'$	Figure 4-11

The I_L- normalisation shows some scatter of the data around the normalised NCL with equation:

$$I_L = 2 - \ln(p')/3 \qquad \textbf{Eq. 4-14}$$

The significance of the constants in this equation can be explained as follows:

- The slope of the line, 1/3, indicates that the confinement stress at plastic limit is three natural log-cycles larger than at the liquid limit for gold tailings, i.e. or $p'_{PL} = 20p'_{LL}$. Schofield and Wroth (1968) found $p'_{PL} = 100p'_{LL}$ for natural clays.
- The intercept, 2, calibrates the normalised NCL so that the liquidity index is equal to zero at the liquid limit and 1 at the plastic limit.

The scatter is a result of the fact that the plastic limit and especially the liquid limit states are not closely gathered around the confinement stresses of p'_{LL} = 20 kPa and p'_{PL} = 430 kPa. Nevertheless, Figure 4-8 illustrates the use of Eqs. 4-4 and 4-14 for predicting the compression of the tailings studied. These equations fit both the Pay Dam and Mizpah fine tailings data quite well, but an error, in void ratio, of up to 0.08 results for the low plasticity coarse samples and the whole tailings mix.

Normalisation using the void index after Burland (1990), Figure 4-9 and Figure 4-11, results in a much better fit compared with the liquidity index, Figure 4-7. The normalisation can be done either by expressing I_V in terms of e_{100} and e_{1000} (as was done by Burland for one-dimensional compression), or by expressing I_V in terms of the critical state parameters N and λ, which is more appropriate for isotropic compression. The success of the I_V-normalisation confirms the log-linear nature of the compression behaviour of the tailings and does not include the effects of the Atterberg limits as does the I_L-normalisation.

The equations of the normalised lines in Figure 4-9 and Figure 4-11 are:

$$I_v = f(e_{100} \;\&\; e_{1000}): \qquad I_V = 2 - \ln p'/2.303 \qquad \textbf{Eq. 4-15}$$

$$I_v = f(N \;\&\; \lambda): \qquad I_V = -\ln p' \qquad \textbf{Eq. 4-16}$$

To predict the compression behaviour using Eqs. 4-15 and 4-16, correlations have to be found between the e_{100}, e_{1000}, N and λ, and the Atterberg limits for example. Figure 4-10 and Figure 4-12 use the following equations to this effect:

- In terms of e_{100} and e_{1000}

$$e_{100} = 0.647\ln\left(\frac{LL - PL}{2.24}\right) \qquad \textbf{Eq. 4-17}$$

$$e_{1000} = 0.53\ln\left(\frac{LL}{10}\right) \qquad \textbf{Eq. 4-18}$$

- In terms of N and λ

$$\lambda = \frac{PI - 1.44}{88.5} \quad \textbf{Eq. 4-19}$$

$$N = \frac{PI + 2.3}{9} \quad \textbf{Eq. 4-20}$$

Eqs. 4-17 and 4-18, Figure 4-10, predict the compression data of the tailings as a function of the Atterberg limits to an accuracy of 0.02 in void ratio, ignoring the effects of sample disturbance at low stress levels. It is concluded, therefore, that the density and stiffness of in-situ tailings can be accurately predicted as a function of the confinement stress and some fundamental parameters, such as the Atterberg limits. For these predictions, the normalisation technique proposed by Burland is recommended together with the equations above.

Influence of Composition on Tailings Compression

For the range of stresses considered in this study, 20 to 500 kPa, comparison between fine and coarse tailings samples indicates that in virgin compression:

Mizpah pond tailings: $e_{fine} \approx 1.5 \cdot e_{coarse}$ **Eq. 4-21**

Pay Dam penstock tailings: $e_{fine} \approx 2 \cdot e_{coarse}$ **Eq. 4-22**

These ratios are for reconstituted samples under controlled laboratory isotropic load conditions. Measurements on undisturbed samples from two locations at the Pay Dam penstock are shown in Table 4-13.

Stress levels in Table 4-13 are based on the measured unit weight of the material. Pre-consolidation pressures in the order of 100 to 150 kPa were estimated by comparing triaxial consolidation tests on reconstituted specimens with tests on undisturbed specimens (Figure 3-80, Chapter 3). The source of these pressures is most likely pore water suctions as a result of previous desiccation of the deposit. Unfortunately, suctions could not be measured directly at the time of this investigation to verify these values. In addition to suction effects, the processes of reclamation and sampling would have altered the in-situ stress state. Thus, void ratios quoted in Table 3-12 and Table 4-13 are based on direct measurements on undisturbed samples in the laboratory, but the stresses could well be as high as 150 kPa due to suctions or even lower than quoted values as a result of stress relief. However, data points representing the values in Table 4-13 plot close to the reconstituted compression curves from triaxial tests as shown in Figure 4-13.

Table 4-13: In-situ void ratios at Pay Dam penstock from undisturbed samples.

Pay Dam		**Fine**			**Coarse**		
1. Undisturbed samples for triaxial tests.							
Depth below surface	m	4.3			3.0		
Void ratio		1.39	1.46	1.41	0.77	0.81	0.76
Average void ratio		1.42			0.78		
Vertical effective stress σ'_v	kPa	75			55		
Overconsolidation ratio (measured)		2			2		
[6] $K_o = 0.44 + 0.2(PI/100)$		0.47			0.51		
$p' = \sigma'_v(1+2K_o)/3$	kPa	49			37		
$e_{fine} : e_{coarse}$		**1.8**					
2. Independent set of undisturbed samples.							
Depth below surface	m	4.3			2.0		
Degree of saturation	%	99			61		
Void ratio		1.488			0.873		
Vertical effective stress σ'_v	kPa	75			33		
$p' = \sigma'_v(1+2K_o)/3$	kPa	49			22		
$e_{fine} : e_{coarse}$		**1.7**					

An interesting feature of Figure 4-13 is that, irrespective of stress level, the void ratios of the fine samples are nearly twice that of the coarse samples. It is concluded that layers of coarse and fine material in a tailings profile, will vary significantly in density or void ratio following soil-forming processes and consolidation. Section 4.3.3, however, will show that they have exactly the same undrained shear strength.

In Section 4.2.3 is was illustrated that the tailings considered in this study are structurally dominated by a matrix of fines or slimes. In fact, the finest sample (90% slimes, PI = 17) contained up to 40% clay minerals and the coarsest sample (50% slimes, PI = 6) at least 10% clay minerals (Table 4-3). The fine sample can be expected to have low bulk density (1 Mg/m^3 in-situ) and high compressibility due to its high slimes content. If a small percentage of tailings sand (quartz grains > 63 μm) is added to this sample[7], the density will increase slightly. An added bulky grain occupies its volume in the sample with a density of 2.63 Mg/m^3, in the fines matrix with a density of approximately 1 Mg/m^3. At some stage, with the

[6] The recommendations of Massarsch (1979) were strictly for normally consolidated clays but are used here as a rough estimate.

[7] Adding coarse particles, is the opposite of what was discussed in Section 4.2.3, however, the basic principles still apply.

addition of more sand, a maximum density will be reached, where fines fill the voids between coarser particles. At this stage, however the sands may still be coated with a thin layer of fines, in which case the mechanical behaviour may yet be influenced by the fines. With the addition of more sand, the density will decrease again with a lack of fines to completely fill void spaces between the coarse particles. The mechanical behaviour of such a sample, however, still depends on whether there is clean contact between coarse particles or a coating of fines. At the extreme end of adding more sand, both the density and mechanical behaviour of the sample will be controlled by the sand fraction. Tailings collected from the pond areas are more dense and less compressible the coarser they are, and vice versa. In this material the fines control both the structure (density) and mechanical behaviour (compressibility and shear strength).

4.3.2 Properties of Gold Tailings as a Function of the Density State

This section briefly discusses some relevant properties of gold tailings as a function of the reconstituted normally consolidated density state. Combined figures of stiffness and consolidation data, measured during the triaxial compression tests, are summarised in Table 4-14.

Table 4-14: Results of isotropic compression tests (stress levels: 5 - 500 kPa).

Description	Parameter	Units	Figure
Stiffness			
Coefficient of Compressibility	m_{vi}	m^2/MN	Figure 4-14
Bulk Stiffness	K	MPa	Figure 4-15
Drainage Properties			
Coefficient of Consolidation	c_{vi}	m^2/year	Figure 4-16

Stiffness

The bulk stiffness of a specimen can be expressed either through the Coefficient of Compressibility, m_{vi}, or its inverse, the Bulk Modulus, K. These parameters are calculated for each increment of compression loading and are dependent on the magnitude of the stress increment over which they are calculated. In this study load increments were doubled each time.

The Coefficient of Compressibility can be calculated from the compression data using,

$$m_{vi} = \frac{1000\delta e}{(1+e_o)\delta p'} \quad \textbf{Eq. 4-23}$$

where m_{vi} = coefficient of compressibility in m^2/MN or 1/MPa

e_o = void ratio at the start of an increment

δe = incremental change in void ratio

$\delta p'$ = incremental change in mean normal effective stress in kPa

The Bulk Modulus is then simply the inverse of the coefficient of compressibility, or

$$K = \frac{1}{m_{vi}} \qquad \textbf{Eq. 4-24}$$

To determine the nature of the change in stiffness with increased confinement pressure, care should be taken with the measured initial values at very low stress levels. The void ratio measurements at the start of a compression test were found to lie consistently below the theoretical normal consolidation line. This is especially apparent on the Void Index normalised plots, Figure 4-7 and Figure 4-9. The slightly overconsolidated state of the specimens was probably the result of slight disturbances during specimen preparation at such high void ratios and the granular nature of the silty tailings.

Figure 4-17 and Figure 4-18 re-plots the experimental stiffness data as symbols, and compare these to curves of stiffness, based on the NCL's of the respective samples. Note the deviation of the measured points at low stress. The shape of the normally consolidated stiffness curves suggests that stiffness increases proportionally to the isotropic confinement stress as,

$$m_{vi} = \frac{A}{p'} \quad \text{or} \quad K = \frac{p'}{A} \qquad \textbf{Eq. 4-25}$$

where A = a curve fitting constant.

Best fit values for *A* with respect to the NCL-data are:

- Mizpah whole tailings $A = 39$
- Mizpah pond fine tailings $A = 55$
- Mizpah pond coarse tailings $A = 28$
- Pay Dam penstock fine tailings $A = 79$
- Pay Dam penstock coarse tailings $A = 42$

Empirical correlations between *A* and fundamental parameters such as the Atterberg limits may be sought. However, the stiffness parameters can just as well be calculated from the slope of NCL, which in itself is a function of the Atterberg limits.

Loading conditions in a tailings impoundment are often assumed to be one-dimensional for material in the pond area, or when overburden pressures overcome desiccation effects with depth in the beach and daywall areas. The Constrained Modulus, appropriate for one-dimensional load conditions, can be expressed as a function of the Bulk Modulus and Poisson's ratio, see Eq. 4-29.

For one-dimensional loading,

$$m_v = \frac{1}{M} \qquad \textbf{Eq. 4-26}$$

where m_v = coefficient of compressibility for one-dimensional loading
M = constrained modulus not to be confused with, M, the CSSM parameter

From the theory of elasticity,

$$\frac{m_{vi}}{m_v} = \frac{E'(1-\nu')}{(1+\nu')(1-2\nu')} \cdot \frac{3(1-2\nu')}{E'} \qquad \textbf{Eq. 4-27}$$

where E' = Young's Modulus for of effective stresses
ν' = Poisson's ratio for effective stresses

Thus,

$$\frac{M}{K} = \frac{3(1-\nu')}{1+\nu'} \qquad \textbf{Eq. 4-28}$$

or assuming $\nu' = 0.33$,

$$M = \frac{3K}{2} \qquad \textbf{Eq. 4-29}$$

Drainage Properties

An attempt was made at determining the drainage characteristics of the tailings during triaxial consolidation stages. Rust (1996) illustrated that piezocone pore pressure dissipation data in tailings did not fit well with consolidation theory derived for clay materials. It will be shown here that even under well controlled conditions in the triaxial apparatus, tailings consolidation deviates significantly from standard one-dimensional consolidation theory. However, by fitting theoretical curve data to tailings consolidation data at 50% pore pressure dissipation a fair estimate of the consolidation characteristics can be made.

Consolidation is the process by which pore water is dissipated through the porous skeleton of a soil following a change in the state of effective stress. Theories of consolidation fall into two main categories. The first, associated with the names of Terzaghi (1923) and Rendulic (1936) are known as unlinked approaches, where it is assumed that the total stress remains constant everywhere so that consolidation strains are caused only by the change of pore water volume. The second is the coupled Biot theory, in which the continuing interaction between soil skeleton and pore water is included in the formulation. This leads, in general, to more complex equations for solution (Biot, 1941).

Biot's equation governing general three-dimensional pore pressure variation can be written as (Gibson & Lumb, 1953),

$$c \cdot \nabla^2 u_e = \frac{\partial u_e}{\partial t} - \frac{1}{3}\frac{\partial \sigma_{kk}}{\partial t} \qquad \textbf{Eq. 4-30}$$

where c = a constant

u_e = excess pore pressure

t = time

σ_{kk} = sum of the total normal stresses

$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$ is the LaPlacian operator

The corresponding Terzaghi-Rendulic equation is,

$$c \cdot \nabla^2 u_e = \frac{\partial u_e}{\partial t} \qquad \textbf{Eq. 4-31}$$

Biot's theory reduces to the Terzaghi equation when:

- The mean normal total stress is time independent.
- The displacement field is irrotational (Sills, 1975), for example during a constant load isotropic triaxial one-dimensional consolidation cycle.

The theory of small strain one-dimensional consolidation as proposed by Terzaghi in 1923 relies on a number of key assumptions:

- The soil is considered to be homogeneous, isotropic and fully saturated.
- The principal of effective stress is valid.
- Darcy's law is valid.
- Soil grains and pore water are incompressible.
- Displacements of the soil and flow of pore water are one-dimensional.
- The coefficients of compressibility, m_v, and consolidation, c_v, remain constant.

- The self weight of the material is ignored.
- Only infinitely small strains are considered.

Under these assumptions Eq. 4-31 can be written as,

$$c_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t} \qquad \textbf{Eq. 4-32}$$

where c_v = coefficient of vertical consolidation

z = depth in the dissipating layer

Eq. 4-32 may be solved analytically for appropriate boundary conditions by the method of separation of variables as described in detail by Taylor (1948). The solution emerges as a Fourier series giving the local degree of consolidation at a depth of z and time t, $U_t(z)$ by:

$$U_t(z) = 1 - \sum_{m=0}^{\infty}\left[\frac{2}{M}\sin(M \cdot Z)EXP\left(-M^2 \cdot T_v\right)\right] \qquad \textbf{Eq. 4-33}$$

where $M = \frac{1}{2}\pi(2m+1)$

$Z = z/H$

$T_v = c_v \cdot t/H^2$

H = length of the shortest drainage path.

Eq. 4-33 was subsequently used as a theoretical model of one-dimensional[8] triaxial consolidation. There are two recognised methods, based on Eq. 4-33, for calculating the coefficient of consolidation from consolidation stages in the triaxial: the square-root-of-time method and the logarithm-of-time method.

Taylor's square-root-of-time method (1948), requires an estimate for the time to the end of consolidation (at least 95% dissipation of excess pore pressure), or t_{100} from the volume change vs. square-root-of-time curve, Figure 4-19a. The value of c_{vi} can then be calculated from

$$c_{vi} = \frac{\pi D^2}{\lambda \cdot t_{100}} \qquad \textbf{Eq. 4-34}$$

where c_{vi} = coefficient of triaxial isotropic consolidation

D = specimen diameter

λ = constant depending on drainage boundary conditions

[8] During consolidation in the triaxial pore pressures were measured at the base with drainage taking place vertically upwards (no side drains) through the top cap to the volume change burette and a back pressure system.

For vertical drainage from the top of the specimen only, and for a specimen height to diameter ratio of $L/D = 2$, $\lambda = 1$ (Head, 1984 Vol.3).

The second method uses the theoretical time factors, $T_{50} = 0.379$ and $T_{90} = 1.031$, for pore pressure dissipation measured at the base of a specimen, draining through the top only, as illustrated by curve B on Figure 4-19b. The value of c_{vi} may be calculated from either of the following equations,

$$c_{vi} = \frac{T_{50} \cdot H^2}{t_{50}} \quad \textbf{Eq. 4-35}$$

or

$$c_{vi} = \frac{T_{90} \cdot H^2}{t_{90}} \quad \textbf{Eq. 4-36}$$

where t_{50} = time for 50% dissipation of excess pore pressures measured at the base
t_{90} = time for 90% dissipation of excess pore pressures measured at the base

Figure 4-20 compares pore pressure dissipation rates measured at the base of triaxial specimens, under approximately 300 kPa isotropic confinement pressure, to theoretical consolidation curves. The theoretical curves were derived using the following procedures:

- **Best Fit**: Equation 4-33 was solved for c_{vi} by numerical iteration, until the least squares error between the measured data and the theoretical curve was a minimum.
- $\boldsymbol{t_{50}}$: The time for 50% dissipation of the base excess pore pressures was used together with $T_{50} = 0.379$ (appropriate for the triaxial boundary conditions) to calculate c_{vi}.
- $\boldsymbol{t_{100}}$: The time for 100% consolidation was estimated using volume change vs. square-root-of-time data, and c_{vi} calculated with Eq. 4-34.

The shape of the tailings dissipation curves on Figure 4-20 deviates significantly from the shape of the theoretical curves. The tailings consolidate more quickly early on as remarked by Rust (1996), but is slower to consolidate near the end, compared with theoretical predictions. Consequently, the method using t_{100} to balance the measured data with the theoretical curves, gives inaccurate estimates of the coefficient of consolidation. This is clearly shown by the values of $c_{vi(t100)}$ at the bottom of Figure 4-20. However, the t_{50} method, which balances the two curves near the middle, results in as good an estimate of c_{vi} as can be expected, see values of $c_{vi(t50)}$ at the bottom of Figure 4-20. Reasons for the tailings consolidation curves to deviate from the shape of the theoretical curves include:

- The fast draining nature of silty tailings compared with the slow draining clays for which the original assumptions were made. For example Darcy's law is only valid under laminar flow conditions, etc.

- Close inspection of the consolidation curves presented in Chapter 3 shows a significant proportion of creep during the consolidation process. Volume changes were taking place up to 24 hours, whereas the excess pore pressures dissipated within about an hour. Creep within the soil skeleton invalidates the incompressible medium assumption.
- The permeability of especially the coarse samples are relatively high compared with the permeability of the triaxial drainage system including the porous stone, filter paper and top-cap drainage lead. Inertia of the measurement equipment may have had an influence on the rate of pore pressure dissipation and volume change.

The best fit procedure was subsequently used in Figure 4-21 to calculate values of the coefficient of consolidation for the tailings shown in Figure 4-16.

Rowe (1959) proposed a method of adjusting the coefficient of consolidation under isotropic load, c_{vi}, to give equivalent values for one-dimensional loading, c_v, for clays using a multiplying factor

$$f_{cv} = \frac{1}{1 - B(1 - A)(1 - K_o)} \qquad \textbf{Eq. 4-37}$$

where A & B = the Skempton (1954) pore pressure parameters
K_o = coefficient of earth pressure at rest

For normally consolidated clays, the pore pressure parameter A is likely to lie between 0.5 and 1 (Head, 1984 Vol. 3); according to Bishop and Henkel (1962) A = 0.47 for a sandy clay and A = 0.08 for a loose sand. K_o is approximately 0.5 for the range of plasticities measured in tailings, according to the work of Massarsch (1979). If B is close to unity and taking A as 0.3, $f_{cv} \approx 1.5$. Using this relationship together with the stiffness values from Eq. 4-29, the vertical permeability can be calculated using,

$$k_v = c_v \cdot m_v \cdot \gamma_w \qquad \textbf{Eq. 4-38}$$

Permeabilities for the tailings calculated using Eq. 4-38, are represented as the solid lines in Figure 4-22. This figure shows a rapid initial reduction of 2 to 3 times the permeability up to a confinement stress of 100 kPa approximately, but thereafter, becomes fairly constant at the following approximate values:

- Mizpah Whole Tailings 10 m/year
- Mizpah Pond Fine 5 m/year
- Mizpah Pond Coarse 15 m/year
- Pay Dam Penstock Fine 1.5 m/year

- Pay Dam Penstock Coarse 2.5 m/year

Tailings literature seem to agree that permeability can best be predicted using Hazen's formula or

$$k = c \cdot D_{10}^2 \qquad \textbf{Eq. 4-39}$$

Using the permeabilities in Figure 4-22 together with the effective particle diameters of the respective samples, D_{10}, Hazen's constant becomes 1.45. Mittal and Morgenstern (1975) proposed a value of 1 for tailings sands. Hazen's formula would under predict permeabilities at low stress levels by a factor of up to 3 or 4 compared with the values in Figure 4-22.

4.3.3 Shear Strength of Gold Tailings

It has been shown, for samples recovered from the pond areas of impoundments, that there are significant differences in:

- *Composition*: Between typical fine and coarse deposited layers there are significant differences in grading, especially in the median particle size or D_{50}, but not as much in the upper and lower limits of D_{10} and D_{90}. Different grades consist of varying percentages of bulky tailings sands and plate-like fines or slimes. The fundamental properties of these sands and slimes differ significantly with respect to mineralogy, particle shape and surface texture.
- *Compressibility State*: The density, compressibility and consolidation properties of tailings are highly dependent on grade, but seems to be controlled by the slimes fraction

It does, however, appear that the specific gravity remains constant for different grades recovered from the same location. A study of the literature on gold tailings shear behaviour in Chapter 2 also indicated that the shear strength properties of gold tailings are independent of grade, with zero effective cohesion and an effective angle of internal friction, ϕ', of 34° on average. The discussion that follows will show that not only the strength parameters, but also the absolute undrained shear strength, are independent of tailings grade.

Table 4-15 summarises the relevant shear and critical state parameters of the five gold tailings samples studied. The parameters were derived using best fit procedures on the undrained triaxial shear data of Chapter 3. The final selection of the failure state was based on a combination of the following aspects:

- Shape of the triaxial tests paths including deviator stress (q'), mean normal effective stress (p') and excess pore pressure (u_e) response with shear strain (ε_q).

Table 4-15: Stress paths and properties derived from undrained triaxial shear.

Dam	Description	$^{9}\lambda$	N	Γ	M	ϕ'	Figure
Mizpah	**Reconstituted - (Whole tailings, delivery pulp)**						
	Whole Tailings	0.066	1.082	1.025	1.400	34.5	Figure 4-23 Figure 4-24
	Reconstituted - (Pond fine and coarse layers)						
	Pond Fines	0.105	1.512	1.442	1.400	34.5	Figure 4-25
	Pond Coarse	0.045	0.856	0.826	1.350	34.5	Figure 4-26
Pay Dam	**Reconstituted - (Pond fine and coarse layers)**						
	Penstock Fines	0.176	2.126	1.990	1.270	31.5	Figure 4-27
	Penstock Coarse	0.068	1.040	0.980	1.270	31.5	Figure 4-28
	Undisturbed - (Pond fine and coarse layers)						
	Penstock Fines	0.162	2.053	1.935	1.370	34.0	Figure 4-29
	Penstock Coarse	0.080	1.134	1.070	1.460	36.0	Figure 4-30

- Maximum stress ratio, $\eta = q'/p'$.
- Goodness of fit of the failure envelope to the Mohr's circles of failure using the following relationship between M and ϕ':

$$M = \frac{6\sin\phi'}{3 - \sin\phi'} \qquad \textbf{Eq. 4-40}$$

Eq. 4-40 holds for triaxial compression on purely frictional materials (Atkinson & Bransby, 1978).

Some observations on the behaviour of the tailings during undrained triaxial shear are listed below:

(a) Referring to Figures 4-23, 4-25 and 4-27 the stress paths for reconstituted specimens were all characteristic of normally consolidated strain hardening behaviour to failure. The mean normal effective stress decreased with increasing deviator stress resulting in the characteristic ellipse-shaped stress paths often used in CSSM.

(b) Only the undisturbed specimens at low confinement stress (p' = 50 kPa) showed lightly overconsolidated behaviour, confirming the assumption of a pre-consolidation stress of approximately 150 kPa for the in-situ material. This is clearly shown on Figure 4-29,

[9] The critical state parameters are given in terms of void ratio in the compression plane.

where both the fine and coarse specimens, but especially the coarse specimen, show near vertical stress paths to the CSL in the stress plane.

(c) No structural collapse or strain softening was evident in either reconstituted specimens prepared from a slurry, or in the undisturbed specimens. In other words, in Figures 4-23, 4-25, 4-27 and 4-29, there are no cases where the deviator stress increased up to a certain level, and then decreased with continued shear.

(d) Except for the Pay Dam fine tailings (fine specimens in Figures 4-27 and 4-29), which are the finest material of all, stress paths displayed phase transfer dilation at failure (Ishihara et al., 1975) and strain hardened with continued post failure shear. Post failure dilatency was especially prominent in the coarser grades shown in Figures 4-23 and 4-25. Pay Dam fine tailings reached an ultimate state or critical state at failure with no change in effective stress or pore pressure with continued shear.

(e) In each case where two specimens of different grades, but originating from the same sampling location, were tested, the stress paths were almost exactly the same in the stress plane. This observation holds for both reconstituted (Figures 4-25, 4-27) and undisturbed (Figure 4-29) samples. As a result, layers of fine and coarse composition from the same location had the same undrained shear strength, which is adequately described by a single effective shear strength parameter of $M \approx 1.36$ or $\phi' \approx 34°$. Collectively the variation in ϕ' was between 34 ± 2° - a remarkable fact, considering the variations in grading, density, stiffness etc.

The observation of approximately constant undrained shear strength for fine and coarse samples can be explained with respect to differences in void ratio and drainage properties. The method of preparing specimens from a slurry at constant virgin confinement stress, results in widely different densities or void ratios for the fine and coarse samples respectively. For example at a confinement stress of 5 kPa the finer tailings exist at a void ratio of 1.7 compared with 0.8 for coarse tailings - a factor of two difference. These diverse density states exist throughout the consolidation stages and during undrained shear. Similar states were also measured on the undisturbed samples, with void ratios in the fine tailings approximately 1.5 compared with 0.8 in the coarse tailings, a ratio of nearly 1.9 (Table 4-13). If new specimens were to be prepared, where fine and coarse tailings have the same density at the same confinement pressure, it would only be possible by pre-consolidating the fine tailings. In this case the fine specimen would be much stronger than the coarse specimen. This principal is also well illustrated by the State Boundary Surfaces (SBS's) for the Pay Dam penstock fine and coarse tailings. Figure 4-31 shows the SBS's assuming Cam-Clay as a behavioural model and using the parameters derived in Table

4-15. The use of Cam-Clay theory is only for illustrative purposes, other behavioural models could just as well have been used. On the stress plane (q' vs. p' - Figure 4-32), there is hardly any difference between the two SBS's and CSL's, hence the similar shear strengths at any given confinement stress. However, in the compression plane (e vs. p' - Figure 4-32) there are major differences between the fine and coarse sample densities. In preparing the graphs, void ratios were chosen to correspond with the values measured during the triaxial consolidation tests, which are representative of void ratios and densities under confinement pressures of 50 up to 400 kPa. It is theoretically possible to overlap the two SBS's on the compression plane, either at extremely high confinement pressures in the fine tailings, or extremely low densities in the coarse tailings. However neither of these would be practically feasible.

Although the undrained shear strength of tailings appear to be independent of grade following similar soil-forming histories, field loading conditions are not necessarily fully drained (no build-up of excess pore pressure) or fully undrained (no change in pore water volume). Under field loading the higher permeability coarse layers could mobilise greater partially drained shear strengths compared with the lower permeability finer grades. In the extreme, the coarse material will mobilise the full drained shear strength, whereas the fine material will mobilise the much lower undrained shear strength, depending on the rate of shear. In fact, Eq. 4-45 indicates that the drained shear strength would be almost three times as high as the undrained shear strength, provided the same initial isotropic confinement stress.

The stress paths in Figures 4-25, 4-27 and 4-29 show that,

$$q'_{f(undrained)} \approx 0.65 p'_o \qquad \textbf{Eq. 4-41}$$

From the equation of the CSL,

$$q'_f = \mathrm{M} \cdot p'_f \qquad \textbf{Eq. 4-42}$$

and restrictions of drained triaxial compression,

$$q'_f = 3\left(p'_f - p'_o\right) \qquad \textbf{Eq. 4-43}$$

it can be shown that,

$$q'_{f(drained)} \approx 1.85 p'_o \qquad \textbf{Eq. 4-44}$$

Eq. 4-41 and Eq. 4-44 can be combined to show that,

$$q'_{f(drained)} \approx 2.85q'_{f(undrained)} \qquad \textbf{Eq. 4-45}$$

Another source of differing shear strengths between fine and coarse tailings results from their respective post critical state stress paths. Under controlled undrained shear, fine tailings reach an ultimate critical state at constant strength with continued shear strain. On the other hand, coarse tailings are likely to undergo phase transfer dilation after reaching the critical state and mobilise greater shear strengths with continued dilation, even under fully undrained loading.

It is concluded that following similar sub-aqueous sedimentation-consolidation histories, tailings of different grades will exist in an impoundment at varying density states, but so that their specific gravity and undrained shear strengths are virtually the same. However, the quick draining properties of the coarser material coupled with post failure stress-dilatency can result in higher strengths being mobilised in these materials. No tests were performed on undisturbed samples from the daywall or beach areas to confirm these observations for sub-aerial deposition. However, isotropic suction from desiccation should not alter the state of the material apart from increasing the density and stress level in both the fine and coarse layers.

It has to be emphasised here that the samples were all collected from the pond areas of impoundments, representing a selection of fine and coarse layers from these sites. Evidence from electron micrographs and behaviour of these samples under laboratory test conditions suggest that state was controlled by the fine slimes rather than the coarse sands. With higher fines content, densities were lower and the material was more compressible, and vice versa.

4.3.4 Structure

It has not been a principal objective of this study to define the structure of deposited tailings. However, based on visual observations of undisturbed specimens under the electron microscope and the results of a number of laboratory tests, some general remarks are in order.

A packing of uniform spheres can range in void ratio between e_{max} = 0.9 and e_{min} = 0.35, which are typical of the limiting void ratios in cohesionless, single grained, rounded sands (Sowers, 1979). Soils with angular bulky grains usually have a somewhat smaller range with e_{max} = 0.85 to e_{min} = 0.45. Flattened or elongated particles do not form simple cohesionless structures, slabs may bridge voids resulting in an open high void ratio packing, or be wedged tightly and parallel in a stable mass with low void ratio. In such materials the maximum and minimum void ratios probably have little significance. Flaky particles, such as clay minerals, may similarly form either an open haphazard packing or an oriented dense fabric, depending on the nature of the forces between the particles and the soil-forming environment.

With an heterogeneous arrangement of bulky grains and flaky fines such as tailings, the packing arrangement or fabric would be highly dependent on the amount and orientation of the fines. If there is a deficiency of fines the structure may be dominated by a skeleton of bulky particles and the fines will only serve to fill voids between these grains. On the other hand even with a deficiency of fines the mechanical behaviour can still be determined by layers of fines between the coarse particles. Nevertheless, with addition of fines the void ratio will decrease as more and more fines are filling the voids within the skeleton, resulting in a lowering of the normal consolidation and critical state lines in the compression plane. This is clearly illustrated by Papageorgiou et al. (1999) who found the steady state line for a coarse sandy tailings to lie above samples of the same material with some addition of fines. Such a coarse structure, dominated by the bulky tailings sands, can only be expected to exist on the upper beaches of an impoundment or when using cyclone underflow for embankment construction. At some stage, as the fines content increases, the bulky grains will be pushed apart and start to float in a "sea of fines". As soon as this happens the density reduces with increasing fines, resulting in higher void ratios and a raising of the normal consolidation and critical state lines in the compression plane. Tailings considered in this study show a deficiency of bulky particles, even for the coarser samples, so that the structure is determined and controlled by the randomly orientated fines rather than the bulky sands. This is entirely consistent with the measured high void ratios, the fact that the state lines of the fine samples are located above that of the coarse samples and with visual evidence from electron micrographs of undisturbed specimens.

According to Sowers (1979) a honeycomb structure can develop when cohesionless fine sand or silt particles settle in still water. Because of their small size they settle slowly and wedge between each other without having the opportunity to roll into more stable positions.

Schiffman et al. (1986) found that cohesionless sands and silts, deposited sub-aqueously at low to moderate relative densities, exhibit peak undrained shear strength behaviour consistent with a meta-stable structure. Similar observations have also been made by Lucia et al. (1981) and Troncoso (1986) on tailings and silty sand hydraulic fill, respectively. Such a meta-stable structure can also develop when damp, fine sand is dumped into a fill or pile without densification or when a specimen is prepared by wet-tamping techniques.

Papageorgiou et al. (1999) succeeded in inducing liquefaction behaviour in tailings only by using wet tamping techniques to prepare triaxial specimens. The honeycomb structure is usually able to support static loads by arching, with little distortion before de-structuring commences. Upon de-structuring under compression, shear or dynamic loading, the meta-stable structure may collapse with excessive deformation and possibly liquefaction.

It is possible for tailings to form with an open meta-stable structure when settling in the still pond of an impoundment. Undisturbed samples recovered from the Pay Dam penstock area did not indicate structurally permitted states under consolidation, or collapse during undrained shear. However, these samples were subject to the effects of drying prior to sampling, resulting in a lightly overconsolidated state, which could have destroyed any structure. Material on the beach and daywall areas cannot be expected to exist with an open structure as flow across the beach allows for horizontal movement and rolling of particles into a stable packing. In addition, the effects of desiccation on sub-aerial beaches result in an overconsolidated state.

It is a well known fact that gold tailings slurries are flocculated when leaving the reduction works. The degree of flocculation depends on the concentration and nature of the fines in the tailings. Inter-particle forces between the fine particles in a flock produce strong bonds trapping considerable free water in the structure with resulting high void ratios. Although a flocculated sediment is highly compressible under static loading the strong inter-particle bonds are able to resist displacement under vibration loading.

A number of potential bonding agents are present in the tailings particles and process water including silica, metal oxides and calcium carbonate. However, no evidence of cementation bonding by such elements could be found on the micrographs of undisturbed samples or detected during consolidation and shear of undisturbed triaxial specimens. The tailings behave rather like a loose but stable material that strain hardens to failure. On reaching the critical state the material may be subject to phase transfer dilation depending on the concentration of fines.

Precipitation on exposed surfaces often results in the formation of a white crystalline surface crust. This crust is so localised in nature that it constitutes only a thin surface skin, without influencing the strength of the material. However, it may affect the evaporation potential from the surface and possibly serve as a discontinuity in permeability.

4.4 CHARACTERISATION BY PIEZOCONE

Piezocone soundings were performed at both sampling locations, Pay Dam as well as at the Mizpah cross-section. The results of these soundings are correlated in this section with the findings of the laboratory test program to evaluate existing interpretation procedures and to extend these methods where applicable. The piezocone has enjoyed worldwide recognition as a valuable characterisation tool in tailings, especially for defining the tailings profile and establishing the internal seepage regime. The discussion will show that piezocone penetration data can also be used to estimate the state of strength and stiffness in a tailings impoundment.

Figure 4-33 and Figure 4-34 summarise the piezocone test results on Mizpah and Pay Dam. A phreatic surface or water table was only encountered on the Pay Dam at a depth of approximately 16.4 m below surface. The ambient pore pressure build-up below the water table was approximately 7 kPa per meter depth, indicating a slight vertical downwards seepage gradient. This is consistent with seepage losses from an adjacent, active, tailings impoundment, which has been a source of many problems for the mine.

4.4.1 Soils Identification

Throughout the development of the piezocone a number of soils identification systems have been proposed. Probably the best known of these include the Jones and Rust identification chart, Figure 4-35a (Jones & Rust, 1982; 1983), and the Robertson and Campanella charts, Figure 4-36 (Robertson, 1990). Both these systems provide similar results and are based on normalised data resulting from cone resistance, sleeve friction and pore pressure measurements under saturated conditions. They are designed for the pore pressure sensor located directly behind the cone tip. The Jones and Rust system has been modified slightly by the author by extending the material definitions in the soft to very soft clay ranges in accordance with the Robertson and Campanella system, Figure 4-35b. This modified Jones and Rust system will be used to compare the piezocone derived profiles with actual profiles.

The exposed profiles at the Pay Dam beach and penstock sites provided excellent opportunities to evaluate the performance of the piezocone soils identification system. However, these sites have been exposed to drying, and although layers of the finer tailings are saturated (S = 99% - Table 4-13), the coarse layers are not (S = 61% - Table 4-13).

Figure 4-37 and Figure 4-39 illustrate direct comparisons between the in-situ profiles and piezocone penetration data for both sites on Pay Dam, together with the profiles derived using the modified Jones and Rust identification system, Figure 4-35b. For the purpose of tailings identification the soil descriptions have been changed from clay, silt, silty-sand and sand to fine silt, silt, coarse silt and fine sand, which are more practical descriptions for tailings. Figure 4-38 and Figure 4-40 show the penetration data represented on the identification chart itself.

Due to the stiffness of the system, the piezocone pore pressure sensor is able to react instantly to the different pore pressure responses in fine and coarse layers (Lunne et al., 1997). In the first meter or so on Figure 4-37 and Figure 4-39 the penetrometer moved through material that is dry to slightly moist and subsequently registered very little pore pressure response. Penetration through such dry material can lead to de-saturation of the pore pressure filter element. However, by using glycerine to saturate the piezocone filter element, the problem of de-saturation was eliminated. As the in-situ moisture content increased with depth, and thus the level of saturation[10], fine layers started to generate positive excess pore pressure and coarse layers, either no response or negative excess pore pressure.

Due to thin individual layers in the profiles, the measured cone resistances and pore pressures are not likely to be fully developed. Both measurements are influenced by a finite zone of material above and below the sensors, the size of which ranges between 2 or 3 to 10 and even 20 cone diameters, depending on material stiffness (Lunne et al., 1997). As the probe nears a coarse stiff layer from within a fine soft layer, for example, the tip resistance will "feel" the stiffness of the coarse layer before it enters and continues sensing the soft layer, once it has penetrated the coarse layer for some depth. Cone resistance is much more prone to these effects than pore pressure, because of the small localised filter element and quick reaction time of the pore pressure measuring system. Ideally a layer should be thick enough so that a measurement develops its full potential, indicated by a plateau or constant value near mid depth, which gradually changes as the next layer is approached. With the highly layered tailings profile, neither cone resistance or pore pressure measurements have the opportunity to fully develop. This is especially evident in the very sharp drop-off in excess pore pressure at depths of 2.3, 2.5, 2.9, 3.1, 3.3, 4.4 and 4.5 meters in Figure 4-37.

With such a highly layered profile as well as the fact that measurements were taken, in this case, mostly above the phreatic surface, it is surprising how well the soils identification chart predicts the field profiles in Figure 4-37 and Figure 4-39. The piezocone, therefore, should

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[10] Note that for both these holes the water table is located below 16m, however at approximately 4 m below surface the fine layers are 99% saturated and the coarse layers 60% saturated.

be considered an effective tool for the in-situ stratification of tailings profiles, given that rapid continuous profiles can be extracted at small cost compared with methods of sampling.

4.4.2 Pore Pressure Dissipation

The piezocone today is recognised as probably the most reliable site investigation tool for determining the in-situ seepage regime in a tailings impoundment (Rust et al., 1984; East et al., 1988a; Van der Berg, 1995; Rust et al., 1995; Rust, 1996; Wagener et al., 1997). A full discussion of its use in this regard does not fall within the scope of this thesis. However, some comments will be made regarding estimates of the in-situ coefficient of consolidation based on pore pressure dissipation tests.

Torstensson (1975) concluded that, based on the theories of cavity expansion, the coefficient of consolidation should be interpreted at 50% dissipation using the following equation,

$$c = \frac{T_{50} \cdot r_o^2}{t_{50}} \qquad \textbf{Eq. 4-46}$$

where c = coefficient of consolidation

T_{50} = normalised time factor for 50% dissipation of excess pore pressures

r_o = cone diameter

t_{50} = time for 50% dissipation of excess pore pressures.

The value of the time factor T_{50}, is derived from theoretical solutions to the consolidation problem, similar to Eq. 4-33, and is dependent on the rigidity index of the material or

$$T_v = f\left(I_r = \frac{G}{s_u}\right) \qquad \textbf{Eq. 4-47}$$

where I_r = rigidity index

G = shear stiffness

s_u = undrained shear strength.

Rust (1996) suggested using a value of T_{50} = 3.74 based on the work of Randolph and Wroth (1979) for cavity expansion solutions to consolidation around driven piles. This value is appropriate for the filter element located directly behind the cone tip and for a rigidity index of I_r = 100, which he believed to be typical for tailings in general.

Piezocone dissipation tests, and hence estimates of the coefficient of consolidation in a tailings impoundment, are practically possible only in layers of the finest tailings. Dissipation of excess pore pressure is so quick in the coarser layers that dissipation tests can not be recorded.

Since pore pressure dissipation is assumed to be horizontal or normal to the axis of the penetrometer in a layer of infinite extent, solution to Eq. 4-46 gives an estimate of the horizontal coefficient of consolidation or c_h. However, in a highly layered tailings profile, with thin successive layers of differing drainage properties, drainage paths may deviate significantly from horizontal. For example, if the penetrometer is stopped for a dissipation test within a layer of fine tailings, but near the interface with a coarse layer, then dissipation can be predominantly vertical in the direction of the coarse layer. Similarly, where discontinuities in the layering exists such as pre-existing surface cracks, drainage patterns may also be altered. These conditions could lead to misinterpretation of in-situ c_v values from piezocone dissipation tests.

Figure 4-41 shows estimates of the coefficient of consolidation based on piezocone dissipation data from Mizpah and Pay Dam. Also shown are the results of the triaxial consolidation tests with respect to the vertical coefficient of consolidation or c_v on reconstituted fine and coarse tailings from the same dams. The piezocone data lies well within the boundaries defined by the triaxial data on fine and coarse tailings from both dams. However, in-situ values, which are representative of the finer tailings only, are consistently higher than laboratory estimates on the same material. The reason for this lies probably in the fact that drainage boundary conditions around the penetrometer are much more complicated than suggested by the assumptions of cavity expansion theory. It is quite possible that there is a significant percentage of vertical drainage given the thin layer thickness and proximity to coarser free draining layers.

4.4.3 Shear Strength and Stiffness

Soil strength interpretation of cone resistance is usually expressed in the form of Eq. 4-48.

$$q_c = N_c \cdot s_u + \sigma_o \qquad \textbf{Eq. 4-48}$$

where q_c = measured cone resistance

N_c = a theoretical cone factor similar to bearing capacity factors

s_u = undrained shear strength based on triaxial data

σ_o = in-situ total confinement pressure, either σ_{vo}, σ_{ho} or σ_{mean} (p'_o)

Theoretical Solutions: Cavity Expansion Theory

According to Lunne et al. (1997) theoretical solutions for N_c can be grouped under the following classes:

- Classical bearing capacity theory: Terzaghi (1943).
- Cavity expansion theory: Spherical - Meyerhof (1951) or Cylindrical - Baligh (1975).
- Cavity expansion theory combined with conservation of energy: Vesic (1975).
- Analytical and numerical modelling using linear and non-linear stress-strain theories: Ladanyi (1967).
- Strain path theory: Baligh (1985).

In fine grained soils, penetration is generally assumed undrained or constant volume. Theoretical solutions for undrained penetration based on cavity expansion theory take the generalised form of,

$$N_c = \frac{4}{3}\left[1 + \ln\left(\frac{\text{Stiffness}}{\text{Strength}}\right)\right] + \text{Constant} \qquad \textbf{Eq. 4-49}$$

In this equation the stiffness to strength ratio or Rigidity Index, I_r, can be expressed as

$$I_r = \frac{G_u}{s_u} \qquad \textbf{Eq. 4-50}$$

where G_u = undrained shear stiffness

Values for the constant in Eq.4-49 vary for cylindrical and spherical cavity expansion theory, as well as on the choice of the stress-strain constitutive relationship. Typically the constant ranges between 1 and 10.

Teh (1987) improved the basic model using an elastic perfectly plastic strain path approach, and showed that undrained penetration is influenced by material shear strength (s_u), in-situ stress state (σ'_{vo} & K_o), relative stiffness ($I_r = G_u/s_u$) and cone roughness (α), so that,

$$N_c = 0.19 + 2.64\ln(I_r) - \frac{\sigma'_{vo}}{s_u}(1 - K_o) + 2\alpha \qquad \textbf{Eq. 4-51}$$

where α = roughness coefficient , rough (1), smooth (0), but 0.5 is commonly used.

In view of the fact that an undrained response is only likely under saturated conditions, the cavity expansion method was subsequently assessed using CPTU data from the Pay Dam penstock and Mizpah beach-pond interface locations. The purpose of this exercise was to determine how sensitive cone resistance is to differences in stiffness (rigidity Index) between fine and coarse layers in an impoundment. In other words, could the upper and lower bound cone values in a typical tailings profile (Figure 4-33 & Figure 4-34) be credited to differences in stiffness between fine and coarse layers. A key feature of this exercise was the assumption of a constant effective angle of friction, $\phi' = 34°$, as was established with the undrained triaxial shear tests on reconstituted and undisturbed fine as well as coarse tailings samples from both locations.

The procedure was developed around the following arguments and assumptions:

(a) The first step was to select a range of mean normal effective confinement pressures, p'_o, representative of in-situ conditions in a typical tailings impoundment, in this case 0 to 400 kPa.

(b) Corresponding in-situ void ratios were then calculated using the normalisation technique proposed by Burland (1990) as set out in Section 4.3.1 of this chapter. To this extent

$$e = I_v \cdot (e_{100} - e_{1000}) + e_{100} \qquad \textbf{Eq. 4-52}$$

where I_v = void index so that

$$I_v = 2 - \frac{\ln(p'_o)}{2.303} \qquad \textbf{Eq. 4-53}$$

e_{100} & e_{1000} = void ratios at 100 and 1000 kPa confinement pressures

$$e_{100} = 0.647 \ln\left(\frac{PI}{2.24}\right) \qquad \textbf{Eq. 4-54}$$

$$e_{1000} = 0.53 \ln\left(\frac{LL}{10}\right) \qquad \textbf{Eq. 4-55}$$

(c) Assuming saturated conditions, the unit weight of the tailings was calculated using

$$\gamma_{sat} = \frac{G_s + e}{1 + e}\gamma_w \qquad \textbf{Eq. 4-56}$$

(d) It was then assumed that sedimentation in a large pool under self-weight loading results in a normally consolidated one-dimensional stress state so that the effective overburden pressure is given by

$$\sigma'_{vo} = \frac{3p'_o}{1 + 2K_o} \qquad \textbf{Eq. 4-57}$$

where K_o = coefficient of earth pressure at rest

K_o can be estimated using either the relationship proposed by Jaky (1944) or Massarsch (1979)

Jaky: $K_o = 1 - \sin(\phi')$ **Eq. 4-58**

Massarsch: $K_o = 0.44 + 0.2\dfrac{PI}{100}$ **Eq. 4-59**

Both these formulations resulted in approximately the same value of K_o = 0.45.

(e) To calculate the total overburden pressure the ambient pore pressure distribution, u_o, was taken from CPTU dissipation data and used in the equation,

$$\sigma_{vo} = \sigma'_{vo} + u_o$$ **Eq. 4-60**

In the case of Pay Dam a phreatic surface was encountered at a depth of 16.4m, but no free water table existed in the Mizpah profiles.

(f) It then became possible to calculate the depth, h, corresponding to each initial p'_o increment through,

$$h = \frac{\sigma_{vo}}{\gamma_{sat}}$$ **Eq. 4-61**

(g) The undrained shear strength at each depth increment was calculated assuming constant effective shear strength, ϕ' = 34°, together with the observation made in Section 4.3.3 that

$$q'_{f(undrained)} \approx 0.65 p'_o$$ **Eq. 4-62**

or

$$s_u = \tfrac{1}{2} q'_f = 0.325 p'_o$$ **Eq. 4-63**

(h) In Section 4.3.1 it was shown that the bulk stiffness of the tailings can be expressed as a function of the confinement stress with

$$K' = \frac{p'_o}{A}$$ **Eq. 4-64**

The values for A depend on the grading properties of the material and range between 40 and 80.

(i) The undrained shear stiffness was estimated from the theory of elasticity as

$$G_u = G' = \frac{3K'}{2}\frac{(1-2\nu')}{(1+\nu')}$$ **Eq. 4-65**

where ν = Poisson's ratio, assumed to be 0.33 for the tailings.

(j) Finally it was possible to calculate an equivalent value for the Rigidity Index

$$I_r = \frac{G_u}{s_u}$$ **Eq. 4-66**

The above procedure was repeated first using material properties for the fine tailings, and then for the coarse tailings, but with constant strength parameters (c' = 0; ϕ' = 34°) in both cases. Fitting predicted cone resistance values to the original CPTU data for both the Mizpah and Pay Dam sites resulted in,

$$N_c = \frac{4}{3}\left[1 + \ln(I_r)\right] + 10$$ **Eq. 4-67**

A direct comparison of Eq. 4-67 with the CPTU data is shown in Figure 4-42.

The cavity expansion method was used to study the stiffness dependency (strength assumed constant) of cone resistance in tailings. This was done in an attempt to account for the large differences in measured cone resistance, that typically follow upper and lower bound trends with depth in tailings. The stiffness ratios calculated for the tailings were very low compared with the suggested value of I_r = 100 by Rust (1996). For the fine tailings I_r ranged between 15 and 20 and in the coarse tailings it doubled to 30 to 40. Cavity expansion theory showed that cone resistance is little affected by the difference in stiffness between fine and coarse tailings. Even increasing I_r an order of magnitude, as was done on Figure 4-42, still did not make a significant difference. Cavity expansion theory predicted the lower bound penetration resistances in tailings well, but failed to account for the upper bound measurements as shown in Figure 4-42.

Effective Stress Interpretation

An effective stress method has been developed by Senneset et al. (1982; 1988), Senneset and Janbu (1985) and Sandven et al. (1988). In this method an empirical bearing capacity formula in terms of effective stress can be expressed as,

$$q_c - \sigma_{vo} = N_m\left(\sigma'_{vo} + a\right)$$ **Eq. 4-68**

where σ_{vo} = total vertical overburden pressure

σ'_{vo} = effective vertical overburden pressure

a = attraction coefficient

N_m = a bearing capacity factor for cone penetration so that,

$$N_m = \frac{N_q - 1}{1 + N_u \cdot B_q} \qquad \textbf{Eq. 4-69}$$

N_q = bearing capacity factor so that,

$$N_q = \tan^2\left(45 + \tfrac{\phi'}{2}\right)\exp[(\pi - 2\beta)\tan\phi'] \qquad \textbf{Eq. 4-70}$$

β = angle of plastification

N_u = bearing capacity factor, or,

$$N_u \approx 6\tan\phi'(1 - \tan\phi') \qquad \textbf{Eq. 4-71}$$

B_q = normalised excess pore pressure generated or,

$$B_q = \frac{u_e}{q_c - \sigma_{vo}} \qquad \textbf{Eq. 4-72}$$

u_e = excess pore pressure measured immediately behind the cone with,

$$u_e = u_t - u_o \qquad \textbf{Eq. 4-73}$$

u_t = measured pore pressure

u_o = ambient pore pressure from dissipation data

The attraction coefficient is aimed at modelling the effects of overconsolidation, desiccation, cementation or any such attractive forces between particles. An estimate of the soil attraction value can be made based on the shape of the q_t vs. σ'_{vo} plot, from triaxial tests or from general experience. Typical values according to Senneset et al. (1989) are:

- 0-10 for soft clays and silts as well as loose sands,
- 10-20 for medium stiff clay/silt and medium dense sand,
- 20-50 for stiff clays/silt and dense sand,
- >50 for hard, stiff and overconsolidated or cemented soils.

The angle of plastification, β, expresses an idealised geometry for the generated failure zones around the advancing cone, and according to Senneset et al. (1990), is difficult to assess, both experimentally and theoretically. However, Senneset and his co-workers argue that β depends on soil properties such as compressibility and stress history, plasticity and sensitivity. Sandven et al. (1988) presented values of β found from experimental correlations between laboratory determined tan(ϕ') and CPTU values, as shown in Table 4-16.

The effective stress method is associated with large degrees of uncertainties and should be viewed as highly empirical and approximate. Nevertheless, ignoring the effects of surface

Table 4-16: Tentative values of the angle of plastification in various soil types after Sandven et al. (1988).

Soil Type	Tentative β-value
Dense sands, overconsolidated silts, high plasticity clays, stiff overconsolidated clays.	-20° to -10°
Medium sands and silts, sensitive clays, soft clays.	-5° to +5°
Loose silts, clayey silts	+10° to +20°

desiccation, a = 0, and using an angle of plastification of 20° in the fine tailings and 10° in the coarse tailings, the method was compared with field data in Figure 4-43. The normalised pore pressure parameter, B_q, was found to be in the order of 0.4 for the fine tailings and zero in the coarse tailings, and should account to some degree for drainage conditions.

Maximum measured cone resistances in the coarser layers were well predicted by this method, which serves as a type of upper boundary. Measurements in the fine tailings layers were overestimated and must be influenced by the fact that layer thickness does not allow full development of excess pore pressures, as well as the low absolute stiffness ratios in these materials.

State Parameter Approach

Been and Jefferies (1985) suggested that the state parameter, ψ, correlated well with large strain engineering parameters from triaxial tests. The state parameter defines the vertical separation of the current state of void ratio to the equivalent critical state void ratio, at the same mean normal effective stress. Been et al. (1986; 1987) developed a procedure for estimating the state parameter in sand from cone penetration tests based on calibration chamber tests. The procedure as adopted here is:

- Define the steady state or critical state line for the material based on laboratory triaxial tests - refer to Table 4-15.
- Normalised state parameters, m and κ are then determined from Figure 4-44 as a function of the slope of the steady state line, or λ_{ss}, see Table 4-17.

Table 4-17: Normalised state parameters for Mizpah and Pay Dam tailings.

Tailings Dam	Description	λ_{ss}	m	κ
Mizpah	Pond fine tailings	0.105	10.46	14.19
	Pond coarse tailings	0.045	11.19	22.90
Pay Dam	Penstock fine tailings	0.170	10.04	12.02
	Penstock coarse tailings	0.075	10.75	16.51

- The state parameter follows from the void ratio at depth minus the equivalent steady state void ratio or

$$\psi = e - e_{ss} \qquad \textbf{Eq. 4-74}$$

where e = current void ratio

e_{ss} = the equivalent void ratio on the steady state line

- The relationship between the measured cone resistance and the state parameter follows from,

$$\frac{q_c - p_o}{p_o^{'}} = \kappa \cdot \exp(-m \cdot \psi) \qquad \textbf{Eq. 4-75}$$

The result of the state parameter approach as a predictive tool in gold tailings is illustrated in Figure 4-45.

The state parameter approach relies on empirical correlations with the compressibility of the material and is not directly influenced by differences in strength. It should therefore be able to differentiate between the more compressible fine tailings compared with the stiffer coarse tailings. As with cavity expansion, measurements in the soft fine tailings were well predicted. In the coarser layers the state parameter gave a better response as a function of the reduced state parameter values in these layers, but still could not account for the upper bound measurements.

Using the known properties of the fine and coarse tailings, the state parameter for isotropic normally consolidated fine tailings range between +0.07 and +0.15 and for the coarse tailings between +0.02 and +0.1. The full range, therefore, lies approximately between +0.02 and +1.5. If the state parameter is calculated directly from field measurements using Eq. 4-75 and average values for κ and m as in Figure 4-46 are taken, the state parameter lies between 0.0 and +0.2 indicative of contractile normal to lightly overconsolidated states. However, in many of the coarser layers the state parameter was negative, indicating dilative states.

Conclusions

The preceding discussion applies well known strength interpretation methods to CPTU data in tailings with varying degrees of success. However, these results can be interpreted with regard to the composition and state of this material in a typical impoundment:

- Sub-aqueous deposition leads to the formation of a loose/soft normally consolidated sediment. This type of material should contract during drained shear or generate positive excess pore pressures during undrained shear, and is not expected to exhibit peak-strength behaviour in the absence of collapsible structure. Desiccation under sub-aqueous conditions can build in some overconsolidation on the beach areas, but limited evidence from this study suggests pre-consolidation pressures in the order of 150 to 200 kPa that will soon be destroyed with saturation and overburden loading.
- Layers of the finest composition of tailings, are expected to shear in an undrained manner during cone penetration at the standard rate of 2 cm/s. Bugno and McNeilson (1984) proposed that an undrained response is likely for material with permeability below 3 m/yr and that a fully drained response can be expected when the permeability exceeds 3000 m/yr. The fine tailings ranged in permeability between 1.5 and 15 m/yr suggesting undrained penetration. The quick and positive response in pore pressure during penetration through these layers, Figure 4-33 and Figure 4-34, strengthens this assumption. Under these conditions cavity expansion theory and the state parameters approach are able to predict penetration resistance as a function of strength and stiffness with sufficient accuracy for practical purposes.
- Layers of a coarser composition can allow considerable dissipation of excess pore pressure with permeabilities between 5 and 20 m/yr, and generate a much higher partially drained shear strength during penetration. The virtual absence of excess pore pressure in these layers, Figure 4-33 and Figure 4-34, is evidence to this fact. In some coarse layers negative excess pore pressures are measured - evidence of dilation. Dilation in granular soils results typically from an "overconsolidated" or dry-of-critical state, or more likely in this case, phase transfer dilation once the stress path reaches the critical state. Undrained triaxial shear of the coarser tailings all show contractile behaviour until the equivalent critical state strength is reached. With continued shear, phase transfer dilation becomes prominent in the coarse tailings. With the large strain field imposed by an advancing cone such dilation would increase the measured cone resistance and generate negative excess pore pressures. The effective stress approach with its B_q-parameter accounts to some extent for this dilatency.

It is concluded, therefore, that penetration resistance in a typical saturated tailings deposit is governed, in fine layers, chiefly by undrained strength and stiffness, but in coarse layers, more by stress dilatency and partial drainage. Lower bound cone resistances are well predicted by cavity expansion and state parameter methods based on the strength and stiffness properties of these layers. The difference between lower and upper bound measurements serves as an indication of the variance in fine and coarse layers in the tailings profile and the effects of partial saturation, partial drainage and post failure stress-dilatency in the coarser layers.

Figure 4-33 is duplicated as Figure 4-47 with the lower and upper bound measurements highlighted. Of these boundaries the lines of minimum cone resistance are fairly constant throughout the cross-section. At a depth of 10 m the minimum cone resistance is approximately 1 MPa for the daywall, lower beach and beach-pond interface locations. These measurements are consistent with the properties of the fine tailings examined in this study and would be well represented by a cavity expansion or state parameter model. At the middle beach and upper beach locations the minimum resistance increases to 2 and 3 MPa respectively at 10 m depth. Probable reasons for this increase include higher stiffness as a result of densification due to desiccation, and a general increase in the coarseness of the material deposited in these areas under sub-aerial conditions. Both these properties would also increase the effects of partial drainage and phase transfer dilation, thus increasing upper bound cone resistances as well. Throughout the cross section the profiles remain highly layered with a mixture of "weak" and "strong" layers of fine and coarse composition.

Figure 4-47 shows a gradual decrease in cone resistance from the upper beach to the pond area, consistent with a general decrease in grade towards the decant facilities. However, penetration resistances measured at the daywall location are comparable to the measurements at the beach-pond interface, rather than at the upper beach locations. This results from paddock system of daywall construction. The whole tailings slurry delivered is used to fill the daywall paddies along flow paths parallel to the wall. At a central low point between two discharge stations, the slurry passes through the daywall into the night area. The idea is that the coarser material will be deposited on the daywall and that the fines will decant to the nightpan. Figure 4-47 suggests that a significant amount of fines, comparable to the pond areas is deposited on the daywall and that the wall itself must be less competent than the upper beach, with respect to both strength and permeability.

The results of this study indicate that materials deposited sub-aqueously in a tailings impoundment can be expected to exist in a contractile normal consolidated state without any collapsible structure, but rather with the potential to dilate post critical state, especially in the coarser material. The question remains why large scale liquefaction has been observed to occur during some failures of gold tailings impoundments, notably during the Merriespruit disaster. Possible mechanisms for this can include the following:

- Loose saturated and uncompacted tailings can generate excess pore pressure during cycles of stress reversal. The effects become cumulative during a seismic event and can initiate liquefaction (Vick, 1983). However, the high permeability of coarse tailings should prevent undrained conditions during cyclic loading so that excess pore pressures dissipate as fast as they are generated. Coarse tailings should, therefore, have very low liquefaction susceptibility.

- Dilative overconsolidated coarse tailings can be contractile at very low strains, similar to overconsolidated sands as shown by Vesic and Clough (1968). This initial contractile state can generate small positive excess pore pressures if not fully drained. Following Been et al. (1987; 1988) a flow type failure may be the result under static load provided a significant trigger mechanism, such as a slope instability.
- Collapsible fabric can also result in a liquefaction type failure as was illustrated by Papageorgiou et al. (1997; 1999) on triaxial specimens prepared by wet-tamping techniques.