MONOTONIC AND CYCLIC SHEAR LOADING RESPONSE OF FINE-GRAINED GOLD TAILINGS

by

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Abstract

The monotonic, cyclic and post-cyclic shear response of gold tailings was investigated using constant-volume direct simple shear test device. The reconstituted gold tailings specimens normally consolidated to vertical effective stress levels ranging from 50 kPa to 400 kPa initially exhibited contractive behaviour followed by a dilative response under monotonic loading, with their shear stiffness and strength increasing with increasing initial effective confining stress. Overconsolidated specimens developed negative excess pore pressures during monotonic shear, with increasing dilative response, shear resistance, and stiffness displayed with increasing overconsolidation ratio (OCR). Overall, the monotonic behaviour of normally consolidated reconstituted gold tailings specimens is similar to the typical monotonic behaviour of normally consolidated clays and low-plastic silts; similarly, the behaviour of overconsolidated reconstituted gold tailings specimens is similar to the typical monotonic behaviour of overconsolidated clays.

During cyclic loading, the tailings exhibited cumulative decrease in effective stress (or increase in equivalent excess pore-water pressure) with increasing number of loading cycles, resulting in progressive degradation of shear stiffness. The cyclic shear resistance increased with increasing OCR. The findings on the cyclic shear response of normally consolidated reconstituted gold tailings are in general agreement with those available published data on the cyclic response of different tailings, obtained from tests carried out on cyclic triaxial (TX) and DSS devices. The CRR of the gold tailings from this study, however, was found to be higher than that observed in Fraser river sand and Quartz rock powder, but in the same range as natural Fraser river silt.
The post-cyclic monotonic shearing response, obtained from DSS tests, carried out on normally consolidated and overconsolidated reconstituted gold tailings specimens was also studied as a part of the current research work. The post-cyclic shear strength of normally and overconsolidated specimens, normalized to the initial effective confining stress, were observed to increase with increasing OCR. The post-cyclic consolidation volume changes experienced by the gold tailings specimens were in agreement with previously published results suggesting that post-cyclic volumetric strains would increase with increasing maximum excess pore water pressure ratio ($r_{u_{\text{max}}}=\Delta u_{\text{max}}/\sigma'_{vc}$) developed during cyclic loading.
Preface

This thesis is written on the outcome of an experimental research program to study the mechanical response of gold mine tailings material. The research was undertaken with funding from Natural Sciences and Engineering Research Council of Canada (NSERC) – Ref.: NSERC STPGP 385905-09: industry collaborator – Golder Associate Ltd.; academic collaborators – Carleton University and University of Alberta.

The following are to be noted with respect to some of the tables and figures presented in the thesis. Table 3.1 is modified from Kim et al. (2011). Part of the Chapter 3 and Figure 3.2 describing the preparation method of the desiccated sample of the gold tailings at the laboratory of Carleton University are extracted and modified from Daliri et al. (2012).

Portion of the text and Figures 4.1 through 4.5 in Chapter 4 are published in Seidalinova & Wijewickreme (2013) of which I am an author. Figure 4.12 is extracted and modified by adding the results from the current study from Wijewickreme et al. (2005). Figure 4.13, 4.14 and 5.6 are obtained and modified by adding data from the current work from Sanin (2010).

I conducted all the experiments, data collection and analysis, as well as the manuscript composition. Dr. Dharma Wijewickreme is the principal investigator of the project. Supervisory author Dr. Dharma Wijewickreme and Dr. Ward Willson were involved in the problem formulation and manuscript edits.
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<tr>
<td>$a_{\text{max}}$</td>
<td>Peak ground acceleration</td>
</tr>
<tr>
<td>$e$</td>
<td>Void ratio</td>
</tr>
<tr>
<td>$e_i$</td>
<td>Initial void ratio (prior consolidation stage)</td>
</tr>
<tr>
<td>$e_c$</td>
<td>Void ratio after consolidation stage</td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>Maximum void ratio</td>
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<tr>
<td>$e_{\text{min}}$</td>
<td>Minimum void ratio</td>
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<tr>
<td>$G_S$</td>
<td>Specific gravity</td>
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<tr>
<td>$K_o$</td>
<td>Coefficient of lateral earth pressure at rest</td>
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<td>$M_w$</td>
<td>Earthquake magnitude</td>
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<tr>
<td>$N_L$</td>
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<td>Mean effective stress</td>
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<td>$r_u$</td>
<td>Pore pressure ratio</td>
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<td>$r_{u-\text{max}}$</td>
<td>Maximum pore pressure ratio</td>
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<tr>
<td>$s_u$</td>
<td>Undrained shear strength</td>
</tr>
<tr>
<td>$u_a$</td>
<td>Pore air pressure</td>
</tr>
<tr>
<td>$u_w$</td>
<td>Pore water pressure</td>
</tr>
<tr>
<td>$\Delta u_r$</td>
<td>Residual excess pore water pressure at the end of cyclic loading</td>
</tr>
<tr>
<td>$(w_i)_{\text{ave}}$</td>
<td>Average initial water content prior to consolidation</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Initial static shear stress ratio</td>
</tr>
<tr>
<td>$\varepsilon_{(v-ps)}$</td>
<td>Post-cyclic volumetric strains</td>
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$\phi_{PT}$ Friction angle at phase transformation

$\phi_{\text{failure}}$ Friction angle at failure

$\phi_{cv}$ Constant volume friction angle

$\gamma$ Shear strain/Unit weight

$\gamma_{\text{max}}$ Maximum shear strain during cyclic loading

$\gamma_d$ Dry unit weight

$\sigma'_{vc}$ Initial vertical effective consolidation stress

$\sigma'_v$ Vertical effective stress

$\tau_{cy}$ Cyclic shear stress amplitude

$\tau_{st}$ Static shear stress bias
**List of Abbreviations**

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<tr>
<td>CPT</td>
<td>Cone penetration test</td>
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<tr>
<td>CRR</td>
<td>Cyclic resistance ratio</td>
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<tr>
<td>CSL</td>
<td>Critical state line</td>
</tr>
<tr>
<td>CSR</td>
<td>Cyclic stress ratio</td>
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<tr>
<td>DSS</td>
<td>Direct simple shear</td>
</tr>
<tr>
<td>FC</td>
<td>Fines content</td>
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<tr>
<td>GWC</td>
<td>Gravimetric water content</td>
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<tr>
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<td>Liquidity index</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid limit</td>
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<tr>
<td>LVDT</td>
<td>Linear Variable Displacement Transducer</td>
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<tr>
<td>MSF</td>
<td>Magnification Scaling Factor</td>
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<td>NGI</td>
<td>Norwegian Geotechnical Institute</td>
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<td>National Research Council</td>
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<td>OCR</td>
<td>Overconsolidation ratio</td>
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<td>Overconsolidation ratio by desiccation</td>
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<td>QSSL</td>
<td>Quasi-steady-state line</td>
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<td>SL</td>
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SS    Steady state/ Simple shear
SSCC  Suction stress characteristic curve
SPT   Standard penetration test
SWCC  Soil-water retention characteristic curve
TT    Thickened tailings
TX    Triaxial
USCS  Unified soil classification system
Acknowledgements

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This Thesis is dedicated to:

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Chapter 1: Introduction

The mining industry is crucially important to the world economy, as it provides raw materials for almost all industrial and household consumer products. However, it is important to note that in typical mining process many tons of waste material is generated to recover a few grams of extracted minerals. This clearly has made mine waste management a key component in a mining operation. Most mine waste materials contain heavy metals, metalloids, radioactivity, acids, and process chemicals; as such, secure disposal of mine waste requires paying due attention to a wide range of physical, chemical, and environmental considerations.

The particle sizes of mine waste may vary from clay to boulder sizes, as they consist of sedimentary, metamorphic or igneous rocks, soils, and loose sediment (Lottermoser, 2010). In this context, two mine waste types that commonly arise from the mining operations are typically classified as: coarse-grained “waste rock” and relatively fine-grained “tailings”.

Tailings are usually transported and placed hydraulically in slurry form during the storage process. Due to the nature of deposition, together with the resulting high degree of saturation and relatively loose densities, the deposited tailings may be susceptible to unacceptable shear deformations during earthquake loading in seismically active areas. This is particularly problematic for tailings dams built using the upstream method of construction.

Surface deposition of thickened tailings is one of the alternative methods to conventional slurry deposition. Herein, prior to deposition, the tailings are dewatered to the extent that they exhibit a
finite yield stress, and are therefore able to form gently sloped stacks. When tailings are
dewatered to point that no particle segregation occurs either during transport or deposition, they
are typically called paste tailings (Cincilla et al., 1997; Martin et al., 2006). The paste tailings as
a material provides enhanced geotechnical performance with a range of positive attributes
including: improved physical properties as a geomaterial, less need to rely on dams for
containment, reduced risk of catastrophic failure of the deposit, increased water recycling within
the mining operation, increased opportunity for progressive closure, and reduced seepage due to
absence of a water cover (Cincilla et al., 1997; Martin et al., 2006; Simms et al., 2007).

The potential for extensive deformation in paste or thickened tailings stacks during static and
earthquake loading condition is a significant design concern. One of the key requirements in the
design of paste tailings deposits is to establish the maximum achievable slope in a paste deposit
while ensuring acceptable performance under anticipated static (i.e., self-weight, operating) and
dynamic (seismic) loading conditions. The knowledge of the stress-strain response of the tailings
material becomes critical in the assessment of the overall stability of a given tailings deposit.

The physical properties of mine waste depend on its mineralogy and geochemistry, deposition
method, mining equipment employed, particle size, and water content. Moreover, soil behaviour
is well-known to be dependent on void ratio, effective confining stress level, particle
microstructure/fabric, anisotropy, stress history, ageing, and intensity and duration of cyclic
loading. These parameters govern shear-induced excess pore water pressure, stiffness, and
strength, in turn controlling the overall response. It is critical to have a solid understanding of the
laboratory element response of paste tailings under static and cyclic (seismic) loading conditions.
Although the response of sands and clays have been studied extensively to date, only limited effort has been placed to study the response of silts/silty soils in general (Boulanger & Idriss, 2007; Bray & Sancio, 2006; Sanin & Wijewickreme, 2006) and for fine-grained tailings in particular (Moriwaki et al., 1982; Poulos et al., 1985; Vick, 1990; Wijewickreme et al., 2005).

Ishihara et al. (1980) investigated the cyclic resistance of several reconstituted different silty tailings to study the consistency characteristics and plasticity. Later, Ishihara et al. (1981) studied cyclic behaviour of undisturbed specimens of tailings obtained from several tailings dams in Japan using triaxial device. Moriwaki et al. (1982) presented results of combined field and laboratory tests on tailings slimes from copper mines. According to the study, the tailings slimes exhibited contractive behaviour under static undrained shearing. The cyclic response of the tailings was examined using triaxial (TX) and direct simple shear (DSS) testing (Moriwaki et al., 1982). Vick (1990) also published a compilation of cyclic resistance ratio (CRR) curves from different mine tailings. McKee et al. (1979), Poulos et al. (1985), Peters & Verdugo (2003) have also investigated cyclic strength characteristics of silty tailings. Peters & Verdugo (2003) showed that the cyclic shear resistance would decrease with increasing fines content, when tested at the same void ratios. They found that the tailings might be susceptible to unacceptable performance during seismic loading leading to failure of tailings storage facilities.

In spite of the above work on tailings the currently available information to understand the response of tailings under thickened conditions is limited. For example, at present, only a limited number of publications about the effect of mechanical overconsolidation ratio (OCR) on the
mechanical behaviour of fine-grained tailings related to cyclic and monotonic response available (Al-Tarhouni et. al., 2011; Kim et al., 2011; Simms et al., 2007; Simms & Grabinky, 2004).

For the above reasons, there is a need to enhance the understanding of the mechanical response of the thickened tailings particularly considering the material type, void ratio, confining stress, particle fabric, etc., and this formed the key impetus for this thesis. With this background, a test program comprising a series of monotonic, cyclic and post-cyclic shear tests was undertaken to study a tailings material derived from a gold tailings material originating from the Bulyanhulu Gold Mine in Northern Tanzania.

The main objectives of the program are:

• assess the effect of stress level on the monotonic response of tailings;
• compare the mechanical response of tailings specimens derived from laboratory deposition/desiccation with those obtained from laboratory reconstitution;
• assess the effect of mechanical overconsolidation (stress-history) on monotonic response;
• compare the observations with those from testing carried out previously on similar fine-grained materials;
• compare the cyclic behaviour of gold tailings with the cyclic behaviour of other tailings materials, and natural sands and silts;
• assess the effect of initial static shear stress on the cyclic resistance ratio (CRR);
• assess the effect of mechanical overconsolidation ratio (OCR) on the cyclic resistance ratio (CRR);
• examine the post-cyclic shear strength characteristics;
• assess the effect of cyclic loading on post-cyclic volumetric strains.

A literature review of previous research on the mechanical behaviour of sands, clays, silts and fine-grained tailings is summarized in Chapter 2. The experimental procedures, description of apparatus and test material used in this study along with sampling method and specimen preparation are discussed in Chapter 3. The experimental results from the main testing program, and discussion of the relevant findings are presented in Chapter 4 and 5, respectively.
Chapter 2: Literature review

Development of new technology in material science required for effective and sustainable mine waste management is one of the most important considerations for the mining industry. Understanding of the mechanical behaviour of tailings to field loading conditions is fundamental for creating meaningful constitutive models for numerical modeling, and in turn, effective geotechnical design of mine tailings facilities. Laboratory testing has an important role to play in this regard, as it helps to understand basic element soil and/or tailings behaviour. The strength parameters, which are typically used in design of mine tailings facilities, landfills, surface foundations, etc., can be obtained during monotonic shearing tests in the laboratory. In addition, structures made of soils, such as foundations, tailings dams, etc. located in seismically active areas are particularly considered to be susceptible to the reduction of strength and stiffness when subjected to cyclic loading.

Within the last 50 years, most of the research was concentrated on the understanding of fundamental behaviour of sands, leading to the development of a detailed approach for engineering design (Youd et al., 2001). Up until recently, only a little attention has been paid to the cyclic behaviour of fine-grained materials. However, after several recent earthquakes where considerable geotechnical failures had been observed in clays and low-plastic silts, significant research has been undertaken to understand the cyclic behaviour of fine-grained soils (Bray et al., 2004; Bray & Sancio, 2006; Sanin & Wijewickreme, 2006). The consequences of liquefaction in fine-grained materials, as reported by Bray & Sancio (2006) and Sanin & Wijewickreme (2006), are different from those occurring in loose saturated sands. Unlike contractive sands, silts undergo cyclic mobility type behaviour (gradual loss of shear strength
and stiffness) then a permanent loss of shear strength. Furthermore, only limited research has been undertaken on fine-grained tailings, although the understanding of the mechanical behaviour or tailings is crucially important for the mining industry.

The main purpose of this chapter is to present information on the methods of tailings deposition, brief observation of the alternative disposal methods, and review of literature of the main aspects of mechanical behaviour of sands, clays, silts and fine-grained tailings. Additionally, discussion of the approaches currently available for the assessment of cyclic shear performance of fine-grained soils is covered.

2.1 Mine waste

Mine waste results from the byproducts of most mineral extraction and represents the largest volumes of wastes generated around the world. According to Crowder (2004), the mining industry in Canada produces approximately 1 million tons of waste rock and 950,000 tons of tailings per day, which in total equals to 700 million tons per year. For example, the production of one ton of copper concentrate results in about 2 to 3 m³ of “dry” overburden waste and 200-400 m³ of tailings. The Highland Valley mine near Kamloops, located in south central British Columbia in Canada, one of the largest copper mines in the world, produces about 160,000 tons of copper and 2,300 tons of molybdenum per year (Dawson, 1994; Hanson, 1992). The typical waste stream of the mine is illustrated on Figure 2.1 to serve as an example of the magnitude of this waste generation.
Mine waste commonly consists of two main waste types: coarse-grained blasted-crushed rock and relatively fine-grained tailings. Waste rock is wall rock material removed to access and mine ore, which may contain low concentration or no desired mineral. Waste rock particle size may vary from fine sands to boulders (Daliri, 2013; Wijewickreme et al., 2010); and they are typically deposited in piles or “dumps”. However, rocks containing an economic concentration of the mineral are sent to the mill for processing. According to Lottermoser (2010), “Tailings are defined as the processing waste from a mill, washery or concentrator that removed the economic metals, minerals, mineral fuels or coal from the mined resource”. Usually the rock is ground to particles less than 100 microns in size by mills for hard rock mining grind with most particles having a diameter greater than 10 microns. This results in tailings particles having predominantly silt size. Typically, tailings are contained by dams constructed using coarser fraction of mine tailings waste (Daliri, 2013; Robinsky, 1999; Vick, 1990).

2.2 Tailings impoundments

There are approximately 3,500 of good-sized tailings impoundments in the world with heights of hundreds of meters and areas of many square kilometers (US EPA, 1994; Davies, 2002; James, 2009). Based on Vick (1990), the surface impoundments are divided into two types: water-retention type dams and raised embankments.
2.2.1 Water-retention type dams

Water-retention type dams for tailings disposal are very similar to conventional water storages structures in appearance, design and construction, and they are built to their full height preceding discharge into the impoundment. These types of dams are reasonable for tailings impoundments with high water storage requirements.

Typical water-retention type dam for tailing storage is illustrated in Figure 2.2. The internal zoning consists of impervious core, drainage zones, appropriate filters, and upstream riprap.
Similar to conventional earth dam technology is used for design of filters, internal seepage control, and slopes (Vick, 1990).

![Diagram of water-retention type dam for tailings storage](image)

Figure 2.2 Water-retention type dam for tailings storage (extracted from Vick, 1990)

### 2.2.2 Upstream, downstream and centerline methods for tailing storage

The majority of tailings impoundments consist of raised embankments, built using either upstream, downstream or centerline methods. The main difference of raised embankments from conventional water-retention type structures is that construction of the embankment lasts throughout the life of the impoundment and is more cost-effective. Raised embankments consist of dikes constructed to contain tailings and wastewater produced. The dikes are raised incrementally depending on the amount of tailings and wastewater. Raised embankment impoundments are usually constructed using upstream, downstream or centerline methods as shown in Figure 2.3 (Vick, 1990). In the upstream method, the dike is constructed in stages, with later stages being placed on top of the tailings themselves. Upstream embankments are most susceptible to liquefaction under seismic loading, and the method is not recommended for areas with high seismic potential (Dobry & Alvarez, 1967; Vick, 1990).
Within the last 100 years there were more than 200 incidents involving unacceptable behaviour of tailings dams. Main reasons of the failures are slope stability, overtopping due to flooding, and seismic activity. One eighths of the incidents was caused by liquefaction of the impounded tailings or tailings used for construction of the impoundments (ICOLD, 2001; WISE, 2009; James, 2009).

2.2.3 Other disposal methods

Tailings are usually transported and placed hydraulically in slurry form during the storage process. Normally, as such, a significant amount of research is going on to improve shear strength characteristics of tailings. Thickened tailings, paste and cake deposition methods are some of the alternative innovative disposal methods based on using dewatered tailings that provide for increased shear strength and deformation characteristics of stored tailings. Some of the above methods are illustrated schematically on Figure 2.4. However, the focus of this thesis will be mainly on tailings deposited by paste tailings deposition method.

2.2.3.1 Slurry tailings

Tailings emanating from the mill normally have very high water contents due to water requirements for operations and transport during mineral processing. Solids concentrations for hard rock mines are usually 50% or lower. Such high water content in slurry tailings may contribute to failure of tailings dams. The slurry tailings, slowly gain strength with time; however, it may still remain susceptible to liquefaction for a considerable time after the deposition. Slurry tailings are usually transported from the mill to the disposal area while in the
turbulent flow regime. During deposition slurry tailings usually undergo grain size segregation with courser particles settling closer to the disposal point, and finer particles settling further down. Slurry tailings deposits normally have slopes less than 1% (Vick, 1990).

### 2.2.3.2 Thickened tailings deposition method

Thickened tailings deposition method was first described by Shields (1974) and later advanced by Robinsky (1979). The method has been widely used since then and is called thickened tailings deposition method. The method is based on dewatering of tailings prior to deposition; and it is an attractive alternative to conventional deposition methods. Thickened tailings are obtained using mechanical compression thickeners or a combination of thickeners and filter presses. High Density Thickened Tailings (HDTT) is dewatered to a point where they have a homogeneous state when deposited from the end of a pipe (Welch, 2003; http://www.tailings.info/disposal/thickened.htm). Thickened tailings typically have high density of about 60% (i.e., moisture content – 40%). There are number of advantages in the method, including increased recycling of process water and improved stability of the deposited tailings. The tailings are dewatered to the state where they exhibit a high yield stress upon deposition usually higher than 20 kPa, but typically not higher than 100 kPa; and they can be deposited in self-supported stacks with gentle slope between 1 to 3.5 degrees (ICOLD and UNEP, 2001; http://www.tailings.info/disposal/thickened.htm). This leads to a decreased total volume of the stack layers and increased resistance of tailings to earthquake loading (Daliri, 2013). Thickened tailings gain shear strength through desiccation and self-weight consolidation after the deposition. Unlike conventional methods, thickened tailings stacks do not necessarily need dam containment. Additionally, there is no overlying layer of water on top of the impoundment,
resulting in seepage from the tailings impoundment; this is usually one of the possible reasons in failure of conventional tailings dams (Crowder, 2004).

2.2.3.3 Paste tailings
Paste tailings have higher degree of thickening with very dense slurry, which are pumped into the storage facility by means of positive displacement pumps. Paste tailings usually have higher beach slope angles of 2-10% compared to High Density Thickened Tailings (Theriault et al., 2003, http://www.tailings.info/disposal/paste.htm). Once the tailings stop flowing, desiccation occurs resulting in crack development. The consequent layer of tailings then fills the cracks providing connection between the two layers and increasing the stability of the structure. Typical tailings deposited by paste tailings deposition method are illustrated on Figure 2.5.

2.2.3.4 Dry stacking of tailings (Filtered tailings)
Filtered wet and dry cake deposition method is based on dewatering tailings to higher degrees than paste tailings deposition method. This type of tailings cannot be transported by pipeline due to its low moisture content, which is typically less than 20%. This low moisture content is achieved by a combination of belt, drum, horizontal and vertical stacked pressure plates and vacuum filtration systems (Davies, 2002; http://www.tailings.info/disposal/drystack.htm). Although, the term “dry cake” or “dry stack” may not be absolutely correct, because the tailings have moisture content slightly below saturation, the terminology has been accepted and used by regulators and designers (Davies & Rice, 2001).
Figure 2.3 Common types of raised embankments (extracted from Vick, 1990)
2.3 General mechanical response of soils

Saturated cohesionless soils, which are stable under drained loading, may be susceptible to excessive deformations under temporary undrained loading when they are subjected to sudden or dynamic loads, such as a shock or an earthquake. This phenomenon of liquefaction is one of the most dramatic causes of damage to structures during earthquakes. Due to liquefaction loose soil tend to contract under the cyclic loading, transferring normal stress from the soil matrix onto the pore water, resulting in a reduction in the effective confining stress within the soil and an
associated loss of strength and stiffness that contributes to deformations of the soil deposit (Idriss & Boulanger, 2008).

Under sloping ground, liquefaction can occur as a result of flow failure; therefore, there is an interest in the static undrained response, related to the potential failure leading to flow slides. Studying the post-cyclic monotonic behaviour allows assessing permanent displacements after cyclic loading by providing post-cyclic shear strength and stiffness of soil and/or post-cyclic volumetric settlements.

### 2.3.1 Mechanical response of sands

Static drained and undrained mechanical behaviour of sands has been studied by many researchers (Casagrande, 1975; Castro, 1969; Roscoe et al., 1963; Vaid & Chern, 1985; Vaid & Thomas, 1995). Typical response of sand during drained monotonic loading in the direct shear apparatus under constant effective confining stress is presented in Figure 2.6. Based on the figure, two different types of volumetric responses have been observed: contractive and dilative. The mobilized friction angle, at which soil starts deforming at constant void ratio, is called the constant volume friction angle ($\phi_{cv}$).
2.3.1.1 Monotonic response

The typical monotonic response of sands can be divided into three types, which are illustrated in Figure 2.7:

- Type 1 response is characterized by reaching a maximum shear strength followed by continuous strain softening. Castro (1969), Casagrande (1975), and Seed (1979) called this brittle response as liquefaction; and Chern (1985) defined it as true liquefaction. It is considered that this type of response leads to flow failure under field conditions.
• Type 2 response exhibits maximum peak shear strength, which is then followed by initial strain-softening and immediate strain-hardening (dilation). Castro (1969) named this typical behaviour as limited liquefaction. The soil starts to soften once it reaches the critical stress ratio. Then, the soil deforms at a constant stress ratio and subsequent dilative response. Ishihara et al. (1975) called the deformation at constant stress ratio as quasi-steady-state (QSS). The term phase transformation (PT) is the point at which the soil changes its behaviour from contractive to dilative, this is also a point of maximum excess pore water pressure; and it can be easily observed on stress path plot (Figure 2.7). Constant volume friction angle ($\phi_{cv}$) was confirmed to be unique for given sand by several researchers (Bishop, 1971; Ishihara et al., 1975; Kuerbis & Vaid, 1988; Negussey et al., 1988; Vaid & Chern, 1985). Additionally, it was found that the friction angle at phase transformation ($\phi_{PT}$) is essentially the same as the constant volume friction angle for given sand (Chern, 1985; Negussey et al., 1988).

• In Type 3 response, the soil exhibits continuously increasing shear resistance with increasing deformation with no strain-softening. Initial increase in excess pore water pressure (contractive response) is then followed by a decrease (dilative response) with increasing strain.

2.3.1.2 Cyclic response

Significant volumetric strains can be developed due to cyclic loading in unsaturated sands; however, most research has been done on saturated sands, which have the potential for generation of excess pore water pressures and associated strength and stiffness degradation under cyclic loading. In order to understand undrained cyclic behaviour of saturated sands, laboratory
cyclic tests have been heavily used. Cyclic loading similar to monotonic undrained loading can be classified into three types of response (Castro, 1969; Vaid & Chern, 1985), illustrated in Figure 2.8 through 2.10.

In the first type of response, which is called “liquefaction”, the soil experiences contractive deformation until reaching the steady state (see Figure 2.8). The second type of response is cyclic mobility with limited liquefaction (see Figure 2.9). The soil having this type of response behaves in a dilative manner, once the stress ratio reaches PT. Larger excess pore water pressures are developed resulting in a state of zero effective stress during the unloading part of the cycle. The third type of response exhibits gradual increase in shear strains along with gradual build-up of pore water pressure with an increase in the number of cycles; however, no strain-softening is observed (Figure 2.10). Many researchers call this type of response “cyclic mobility” (Boulanger & Idriss, 2006; Bray et al., 2004; Vaid & Chern, 1985), and the terminology was used in the current work to describe similar behavioural patterns.
Figure 2.6 Typical shear response of loose and dense sand observed during drained monotonic loading in the direct shear apparatus (Modified from Schofield & Wroth, 1968)

Figure 2.7 Typical undrained monotonic loading response of sand: a) stress-strain response, b) stress path (after Vaid & Chern, 1985)
Figure 2.8 Typical “liquefaction” response during undrained cyclic loading: a) stress-strain response; b) stress path; c) shear strain development (after Vaid & Chern, 1985)
Figure 2.9 Typical “limited liquefaction due to cyclic loading” type of response during undrained cyclic loading: a) stress-strain response; b) stress path; c) shear strain development (after Vaid & Chern, 1985)
Figure 2.10 Typical “cyclic mobility” type of response during undrained cyclic loading: a) stress-strain response; b) stress path; c) shear strain development (after Vaid & Chern, 1985)
2.3.1.2.1 Effect of initial static shear stress bias

There are two possible typical configurations in earthquake geotechnical problems: level ground and/or sloping ground. In order to simulate the condition with no initial shear stresses on the horizontal plane prior level ground conditions, “no static shear stress bias” is commonly used (Seed & Peacock, 1971; Vaid & Finn, 1979). In order to simulate sloping ground conditions, specimens are usually consolidated with an applied static shear stress before subjecting to cyclic loading; these types of tests are typically referred to as cyclic shear tests with “initial static shear stress bias”.

The field and laboratory models of the stress field conditions with and without initial static shear bias are illustrated on Figure 2.11. The presence of initial static shear stress has strong effect on the cyclic resistance of sands. However, in some cases, the presence of initial static shear stress may increase the cyclic resistance to liquefaction (Lee & Seed, 1967; Seed et al., 1975) and decrease the cyclic resistance to liquefaction in others (Casagrande, 1975; Castro et al., 1982; Castro, 1969). The effect of static shear stress on the cyclic resistance of sands is influenced by the initial density, as reported by Vaid & Finn (1979), Vaid & Chern (1985), Seed & Harder (1990), Vaid et al. (2001) and Sriskandakumar (2004). In loose sands, the presence of static bias causes reduction in the cyclic resistance; and in dense (dilative) sands, the presence of static bias increases the cyclic resistance.

Additionally, cyclic resistance of sands may be affected by past liquefaction or pre-shearing (Finn et al., 1970; Ishihara & Okada, 1978; Seed et al., 1977; Suzuki & Toki, 1984; Vaid et al., 1989; Sriskandakumar, 2004). Previously liquefied samples had significantly decreased cyclic
shear resistance compared to virgin samples despite a considerable increase in density following consolidation after the first liquefaction stage (Finn et al., 1970). Sriskandakumar (2004) obtained similar results when investigating Fraser River sand using direct simple shear device. This can possibly be caused by fabric changes. For example, during small pre-shearing the soil fabric may improve, resulting in an increased cyclic resistance of sand to liquefaction during a second cyclic loading phase; whereas during large pre-shearing the soil fabric weakens, causing decrease in cyclic resistance to liquefaction in the following cyclic loading.

2.3.2 Mechanical response of clays

Zergoun & Vaid (1994), Vucetic & Dobry (1991), Chu et al., (2008), Boulanger & Idriss (2006) have extensively investigated cyclic shear response of clays, although ground failures in clay deposits are less common than in saturated sands. The cyclic response of natural Cloverdale clay was studied by Zergoun & Vaid (1994). They used low frequency undrained cyclic loading tests, showing that there is a threshold cyclic stress level, separating the response of clay into two different patterns. Shear strains and pore water pressure reach equilibrium with continuing cyclic loading at low cyclic stress levels. Strain development at higher cyclic stress levels can be divided into three phases: decrease in the rate of strain development per cycle followed by constant rate per cycle; and the final stage, characterized by strain development at increasing rate per cycle, which occurs at a constant effective stress ratio regardless of the cyclic stress level.
Figure 2.11 a) Field and laboratory model cases and b) Stress conditions for an element of soil
Typical cyclic clay behaviour is illustrated in Figure 2.12 (Zergoun & Vaid, 1994). Cyclic loading causes a progressive increase in excess pore water pressure accompanied by increase in shear strains leading to significant ground deformation during earthquake (Idriss & Boulanger, 2008). Cyclic behaviour of clay is different from sand; therefore, to differentiate the term liquefaction in saturated sands the term “cyclic softening” is used for clays. Based on test results on different fine-grained materials obtained using Direct Simple Shear (DSS) and Triaxial devices, a relatively unique relationship between the cyclic resistance and undrained monotonic shear strength was found. The relationship is illustrated on Figure 2.13 (modified from Idriss & Boulanger, 2008). The soil sensitivity, which is defined as the ratio of intact undrained shear strength to the remolded undrained shear strength, has significant influence on possible consequences of cyclic softening. Higher sensitivity is more common for soft and normally consolidated clays with large natural water contents. Such clays are more prone to loss of strength during earthquake loading compared to stiff and overconsolidated clays.

As mentioned earlier, the undrained cyclic strength of clays can be expressed as a function of the soil’s undrained monotonic shear strength, illustrated in Figure 2.13. On the Figure the cyclic stress ratio is expressed as the ratio of cyclic shear stress ($\tau_{cyc}$ in direct simple shear and $q_{cyc}/2$ in triaxial shear) to the undrained shear strength under monotonic loading, $s_u$.

Another important feature that defines the mechanical behaviour of clays is the relationship between monotonic shear strength and consolidation stress history:

$$\frac{s_u}{\sigma'_{vc}} = S \cdot OCR^m$$
where $S$ is the value of $s_u/\sigma'_{vc}$ for OCR=1, and $m$ is the slope of the $s_u/\sigma'_{vc}$ vs. OCR relationship on a log-log plot. This relationship is used for the evaluation of cyclic shear strength in clays (Idriss & Boulanger, 2008).
Figure 2.13 Cyclic stress ratios required to cause failure or some strain criteria versus number of uniform cycles at a frequency of 1 Hz in different clays (after Idriss & Boulanger, 2008)
A procedure to convert the irregular earthquake load to an equivalently damaging load, having simple sinusoidal form, was developed by Seed (1975). Based on the procedure, clay response to earthquake with magnitude $M=7.5$ can be represented by approximately 30 uniform equivalently damaging number of cycles at 65% of peak stress. For sands, the same magnitude can be represented by approximately 15 uniform equivalently damaging cycles. The equivalent number of cycles concept is very important, as it is used for the calculation of magnitude scaling factor (MSF), used in the in-situ liquefaction evaluation procedure and provides a convenient metric for comparing the duration of earthquake motions. The relationships between magnitude scaling factor and earthquake magnitude for sands and clays are presented on Figure 2.14. This concept along with the approach for the estimation of the CRR based on the stress history profile and the estimated values for the normalized CSR are described in Idriss & Boulanger (2008).

Number of tests in diagrams, presented by Andersen (2009), where the number of cycles to failure, defined as a strain criterion, was plotted as a function of average and cyclic shear stresses. Permanent and/or cyclic shear strains at failure were illustrated on the diagrams containing the failure mode. As reported by Andersen (2009), the normalized shear strength increased with the plasticity index, which confirmed similar findings by Guo & Prakash (1999) and Ishihara et al. (1981).
2.3.3 Mechanical response of silts

After recent earthquakes where extensive strains and strength loss had been observed in saturated silts, many authors started studying cyclic shear response of silts (Bray et al., 2004a, 2004b; Hyde et al., 2006; Sanin & Wijewickreme, 2006; Polito et al., 2008). Similar to cyclic response of clays, low-plastic silts are susceptible to cyclic mobility type of response, exhibiting an increase in pore water pressure ratio accompanied by loss in shear resistance and stiffness. Unlike sands, low-plastic silts similar to clays do not experience permanent loss of shear strength (Bray et al., 2004b).

As noted by Sanin (2005), normally consolidated channel-fill Fraser River silt specimens exhibited progressive increase in equivalent excess pore water pressure ratio ($r_u$) along with
degradation of shear modulus in all tests conducted at various cyclic stress ratios (CSR). In this “cyclic mobility” type of response, previously observed in dense sands, the specimens showed contractive response at the beginning of the cyclic loading, followed by dilative response. At higher CSR values specimens exhibited phase transformation at early stages of cyclic loading, reaching 100% of excess pore water pressure within lesser number of cycles compared to those subjected to low CSR values. It was found, that the cyclic resistance ratio is not sensitive to the initial confining stress for channel-fill silt tested at stress levels below 200 kPa (Sanin & Wijewickreme, 2006).

The effect of pre-shearing on cyclic response of Fraser River silt was studied and reported by Sanin (2005). For that, the silt specimens, initially subjected to cyclic loading, were then again re-consolidated and subjected to a second cyclic loading phase. Based on the test results, during the second cyclic loading phase, the samples had rapid drop in effective stress, associated with rise in pore water pressure, and had weaker response compared to the first cyclic loading phase, as presented in Figure 2.15. According to the figure, the specimens reached the liquefaction triggering ($\gamma = 3.75\%$) in a fewer number of loading cycles during the second stage than in the first stage. However, Andersen (2009) observed increase in cyclic shear strength after pre-shearing. The effects of de-structuration of the soil particle structure due to large shear strains during cyclic loading might be the cause in the difference between these contrasting findings.
Hyde et al. (2006) studied the effects of cyclic loading on reconstituted, silt-size powdered limestone using cyclic triaxial testing. They concluded that the initial phase transformation, defined by monotonic shear loading in extension tests, was the boundary between stable and contractive behaviour for isotropically consolidated samples. This boundary in anisotropically consolidated samples was determined by monotonic compression tests. Schematic diagrams explaining the contractive and dilative shear behaviour during undrained monotonic and cyclic loading are illustrated on Figure 2.16.

The post-cyclic recompression and post-cyclic behaviour of silt with respect to the effect of initial anisotropic consolidation were investigated by Hyde et al. (2007). The results were obtained using triaxial testing, which showed that compressibility during post-cyclic
recompression was similar for both isotropic and anisotropic initial stress conditions. During the second cyclic loading, which followed recompression, the cyclic strength increased with increasing anisotropy. Isotropically consolidated specimens had weaker soil structure compared to anisotropically consolidated specimens, similar to the findings, reported by Sanin (2005).

2.3.4 Mechanical response of tailings

Researchers have attempted to determine whether the existing methods for evaluation of unacceptable performance during cyclic loading in soils are applicable to tailings, and if so, how would the dynamic behaviour vary from those, occurring in soils (James, 2009).

Ishihara et al. (1980) studied cyclic behaviour of tailings sand and slimes from gold, copper and lead-zinc mines with different plasticity ranging from non-plastic to those having a plasticity index 28. The specimens were isotropically consolidated to 50 or 100 kPa using the cyclic triaxial device. Based on the test results, the cyclic resistance was noted to be influenced by the sample preparation method, plasticity index, and void ratio. The cyclic resistance of tailings sands was slightly lower than that of the course-grained quartz (Ishihara et al., 1980; James, 2009). Later, the cyclic behaviour of undisturbed and reconstituted tailings from Japan was studied by Ishihara et al. (1981). Reconstituted specimens exhibited weaker cyclic response compared to the undisturbed tailings’ specimens. The cyclic resistance was relatively independent of void ratio and gradation curves, but it increased slightly with plasticity index. Garga & McKay (1984) conducted undrained cyclic triaxial tests on isotropically and anisotropically consolidated specimens of reconstituted hard rock tailings and naturally occurring soils (silt and sand). They confirmed that the tailings cyclic resistance is highly influenced by the
Figure 2.16 Schematic diagram explaining contractive and dilative shear behaviour during undrained monotonic and cyclic loading (after Hyde et al., 2006)
specimen preparation method, and that the naturally occurring soils are more resistant to cyclic loading. Additionally, they concluded that the cyclic resistance tends to increase with increasing principal stress ratio.

The influence of the inclusion of different content of non-plastic fines on the monotonic and cyclic behaviour of copper tailings was investigated by Troncoso (1986) and Troncoso & Verdugo (1985). They reported that the fines content have significant influence on cyclic behaviour of tailings, decreasing the shear modulus and the cyclic resistance with increasing silt content (James, 2009).

The cyclic resistance of fine-grained tailings using direct simple shear testing was studied by Wijewickreme et al. (2005). Undisturbed samples of laterite tailings and copper-gold mine tailings, and both undisturbed and reconstituted samples of copper-gold-zinc tailings were tested. The laterite tailings were either low-plastic (PI = 12) or non-plastic; and the copper-gold and copper-gold-zinc tailings were either non-plastic or low-plastic (PI = 2). The stress-strain and stress path responses of the tailings were similar to the response of silts. The specimens displayed cumulative increase in excess pore water pressure with associated progressive degradation of shear stiffness. The vertical stress did not reach zero, but it dropped significantly. The “cyclic mobility” type of shear response was observed, which is similar to the typical cyclic behaviour of natural silts, clays and dense sands. The comparison between the cyclic stress ratio (CSR) versus number of cycles required to reach γ=3.75% of laterite tailings specimens consolidated to vertical effective stresses of \( \sigma'_{vc} = 100 \) and 200 kPa are presented on Figure 2.17. As shown in the figure, the tailings consolidated to a higher vertical effective stress exhibited
higher cyclic resistance compared to the tailings consolidated to a lower stress. However, tests conducted on copper-gold-zinc tailings indicated that the tailings are insensitive to the initial density condition and confining pressure (see Figure 2.18), which is similar to the clay behaviour observed by Zergoun & Vaid (1994) and Atkinson & Bransby (1978).

The comparison of all the tests by Wijewickreme et al. (2005) is reproduced in Figure 2.19, which indicates that the cyclic resistance of low-plastic copper-gold tailings is noticeably higher than that observed for copper-gold-zinc tailings. Also, the laterite tailings with a relatively high plasticity had weaker cyclic response compared to the copper-gold tailings, which might be due to the higher void ratio of the former compared with the latter.

Kim et al. (2011) compared the mechanical response of mechanically overconsolidated gold fine-grained tailings and the same tailings overconsolidated by desiccation. Based on the comparison, the monotonic strength in both mechanically overconsolidated specimens and specimens overconsolidated by desiccation increased. However, the magnitude of the increase in desiccated specimens was substantially less than in mechanically overconsolidated tailings. Desiccated specimens exhibited a higher degree of strain hardening at higher strains (Kim et al., 2011). The gold tailings specimens overconsolidated by desiccation had less increase in cyclic shear resistance compared to the mechanically overconsolidated specimens. Overall, it was confirmed that desiccation does provide benefits in terms of monotonic and cyclic strength, but not to the same degree as mechanical overconsolidation.
Figure 2.17 Cyclic stress ratio versus number of cycles to reach $\gamma = 3.75\%$ for samples of laterite tailings tested at two confining stress levels (after Wijewickreme et al., 2005)

Figure 2.18 Cyclic stress ratio versus number of cycles to reach $\gamma = 3.75\%$ for samples of copper-gold-zinc tailings tested at different confining stress levels (after Wijewickreme et al., 2005)
Figure 2.19 Comparison of cyclic stress ratio versus number of cycles to reach $\gamma = 3.75\%$ for laterite, copper-gold, and copper-gold-zinc tailings (after Wijewickreme et al., 2005)

2.4 Post-cyclic mechanical response

2.4.1 Post-cyclic monotonic shearing

According to Finn et al. (1994), during seismic analysis of earth structures including tailings storage facilities, one of the main questions is whether the flow slide liquefaction occur or not. In case of no flow slide liquefaction, the next step is to evaluate whether there will be any permanent displacements and whether they are tolerable after cyclic loading. In essence, a three-step analyses approach is undertaken in practice (Byrne et al., 2004): (i) triggering analysis to compute the cyclic stress ratio (CSR) and its comparison with the cyclic resistance ratio (CRR) to identify zones that will liquefy; (ii) conventional limit equilibrium analysis for the assessment of post-liquefaction stability; and (iii) when no flow slide is predicted, simple dynamic analysis to determine permanent deformations triggered by cyclic loading. Coupled effective stress analysis procedures that allow capturing the full shear-strain response are preferred for solving
the boundary-value problems because they take into account pore pressures during the analysis (Finn & Yogendrakumar, 1989; Byrne et al., 2004; Wijewickreme et al., 2005).

Post-cyclic shear strength is one of the main parameters required for limit equilibrium analysis. Some researchers (Seed & Harder, 1990; Stark & Mesri, 1992; Olson & Stark, 2003) have developed relations between liquefied strength $S_u$(LIQ), or liquefied strength ratio $S_u$(LIQ)/$\sigma'_v$ and back-calculated SPT values from field case histories. This type of analysis is considered more suitable, as, unlike laboratory testing, it takes into account void redistributions, water film effects after liquefaction, especially in layered deposits with varying permeability (Kokusho, 2003). Still, laboratory results give a good fundamental understanding of the soils response; and they can be used for confirmation or support of the field-based methods. A constant-volume DSS test has the deformation mode more similar to the field; therefore, it is suitable for studying element response.

2.4.2 Post-cyclic settlements

Another important mechanism that takes place due to the dissipation of shear-induced excess pore water pressures is the overall volume change in the soil mass. Post-liquefaction settlements, caused by the volume changes, may develop during and after earthquake shaking. Bray et al. (2004); Bray & Sancio (2006); Boulanger & Idriss (2006) documented the detrimental consequences that took place due to the settlements, and impacted the performance of structural foundations and linear lifelines (such as buried pipelines, bridges).
The primary factors controlling post-cyclic settlements in sands are cyclic excess pore water pressures and cyclic shear strains (Tokimatsu & Seed, 1987). The potential for volumetric strains have been also linked to the field density. To estimate possible settlements in sands, knowing the field penetration resistance (i.e., standard penetration resistance (SPT) N-value or cone penetration testing (CPT) resistance), and the cyclic stress ratio (CSR) applied by the design earthquake several simplified methods are available (Lee & Albaisa, 1974; Tokimatsu & Seed, 1987; Ishihara & Yoshimine, 1992; Wu, 2002; Zhang et al., 2002).

There is only limited information available about post-cyclic settlements of fine-grained materials. A method for estimation of settlements based on the cyclic excess pore water pressure, plasticity index, and bearing capacity factor of safety was proposed by Yasuhara et al. (2001). Wijewickreme & Sanin (2010) observed that specimens with high equivalent excess pore pressure ratios experienced significant volumetric strains during post-cyclic reconsolidation. Using the correlation between $\varepsilon_{v-ps}$ versus $r_{u-max}$, they proposed a method for estimation of post-cyclic volumetric strains under general loading conditions, which was adopted as a part of current practice by the geotechnical profession in the Greater Vancouver area of the Province of British Columbia, Canada (GVLTF 2007).

2.5 Thesis development and scope of work

The above literature review suggests that, while there are findings from research performed on sands, natural silts, and tailings, the current understanding of the mechanical response of the thickened tailings particularly considering the material type, void ratio, confining stress, particle fabric, etc., is very limited. Clearly, laboratory element testing has a key role to play in
advancing this knowledge. Considering the above, a detailed laboratory element testing research program focused on the monotonic and cyclic shear response of relatively undisturbed and reconstituted gold tailings was undertaken at the University of British Columbia, thus forming the theme of this thesis. The intent was to develop an experimental database for calibration and creation of the input parameters for analytical models on the behaviour of tailings material.

The experimental aspects are presented in Chapter 3. The findings on the following aspects of the mechanical response of gold tailings are presented and discussed in Chapter 4: (i) monotonic loading response of normally consolidated relatively undisturbed and reconstituted gold tailings; (ii) monotonic loading response of overconsolidated reconstituted gold tailings; (iii) cyclic loading response of normally and overconsolidated reconstituted gold tailings; (iv) effects of initial static shear on cyclic loading; and (v) effect of mechanical overconsolidation on cyclic loading. The research outcomes on the post-cyclic behaviour are presented in Chapter 5, which includes: (i) post-cyclic monotonic shear strength response; and (ii) post-cyclic reconsolidation response.
Chapter 3: Experimental aspects

The experimental aspects of the current study to understand the monotonic and cyclic loading response of gold tailings are presented in this chapter. Important considerations such as soil material properties, laboratory specimen preparation procedures, and the description of the test device, are carefully addressed. Finally, the experimental program is outlined.

3.1 Material tested

The tailings materials used for this research originate from a gold mine in Northern Tanzania. The material was initially imported to Canada by the Department of Civil Engineering at Carleton University (CU), Ottawa, Canada, in collaboration with whom the present study at UBC was undertaken. Based on the information available from CU, the material was shipped from the mine in water-filled plastic bags to prevent oxidation of the minerals and acid rock drainage at the pumping water content (38%). The water content, however, had decreased to around 22-25% due to settling during transportation, requiring the tailings with the bleed water produced by settling to re-produce the initial moisture content w=38%. The particle size distribution of tailings derived from the testing conducted at CU are presented on Figure 3.1. The mineralogy of the tailings comprises: silicates 80%, pyrite 11%, calcite 5%, and ankerite 4% (Bryan et al., 2010). The results of chemical analysis of tailings liquid phase as reported by Bryan et al. (2010) indicate the following concentrations of important dissolved ionic species: sodium (394 mg/L), arsenic (95.3 mg/L), copper (126 mg/L), magnesium (2010 mg/L), calcium (7030 mg/L), iron (31100 mg/L).
The thickened tailings deposition under field conditions was simulated by the CU researchers using a pre-fabricated “drying box”, and their procedure is described below for the purpose of information and completion.

The drying box is a steel reinforced Plexiglas chamber with a 0.7 m by 1 m plan area as shown in Figure 3.2 (Daliri et al., 2012). The tailings were sequentially deposited into the drying box at 70% solids content in 5 layers, with layer-thickness ranging from 0.15 m to 0.2 m. The target water contents for the first two layers were 12% and 16%, and 18% for all consequent layers. After deposition of the first soil layer, it was desiccated using two commercial fans until the water content of the soil mass fell below its shrinkage limit. Then, the next fresh layer of gold tailings was deposited over the first desiccated layer; the system was then left for 12 hours allowing for re-saturation of the first layer, but before substantial evaporation. Same procedure was repeated with all the consequent layers. The average degree of saturation of the overall mass varied between 0.90 and 0.95. Once completion of the above process, the sample of this soil deposit for laboratory element testing was obtained by pushing thin-walled tubes (with tube length 665 mm, tube wall-thickness 1.5 mm, tube diameter 74.13 mm) were pushed into this multi-layer deposited tailings mass. Some of the tube samples of gold tailings obtained by CU as per above were subsequently shipped to UBC, and in turn; they were used for the soil element testing conducted as a part of the direct simple shear (DSS) testing in this thesis.
Figure 3.1 Grain-size distribution of gold tailings

Table 3.1 Index parameters of gold tailings (modified from Kim et al., 2011)

<table>
<thead>
<tr>
<th>Index Property</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water content, $w_c$ (%)</td>
<td>18.6 to 25.8</td>
</tr>
<tr>
<td>Liquid limit, LL (%)</td>
<td>22.5</td>
</tr>
<tr>
<td>Plastic limit, PL (%)</td>
<td>20</td>
</tr>
<tr>
<td>Shrinkage limit, SL (%)</td>
<td>18</td>
</tr>
<tr>
<td>Plasticity Index, PI</td>
<td>2.5</td>
</tr>
<tr>
<td>Liquidity Index, LI</td>
<td>$\leq 0$</td>
</tr>
<tr>
<td>% of particles &lt; 0.002mm</td>
<td>25%</td>
</tr>
<tr>
<td>% of particles &gt; 0.075mm</td>
<td>25%</td>
</tr>
<tr>
<td>Unified soil classification</td>
<td>ML</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.89</td>
</tr>
</tbody>
</table>
Only limited tests were conducted on specimens prepared from “undisturbed” tube gold tailings samples; this is because of the lack of availability of sufficient quantity of such samples received from CU. Therefore, most tests in this study were performed on reconstituted specimens prepared from the available quantity of gold tailings (see Section 3.3 for specimen preparation details).

3.2 Direct simple shear test apparatus

Much of the research work to study the cyclic shear behaviour of soils has been undertaken using the cyclic triaxial (CTX) and cyclic direct simple shear (CDSS) devices. The CTX has been more widely used due to its simplicity and common availability together with the ability to subject
specimens along well-defined stress paths (i.e., because of the ability accurately define three principal stresses applied on the specimen).

The CDSS apparatus, alternatively, has been noted as a device that would allow continuous rotation of principle stresses and strains similar to those anticipated under shear wave propagation commonly encountered during earthquake loading. Although, the stresses and strains are known to be not uniform due to the absence of complimentary shear stresses on the vertical boundary, linear and non-linear analyses by Roscoe (1953) and Duncan & Dunlop (1969) as well as experimental observations of Cole (1967), Finn et al. (1978) and DeGroot et al. (1994) indicate that the stress non-uniformities are not significant within the middle-third of the specimen. It has also been noted that the use of a low height to diameter ratio reduces the stress non-uniformities considerably (Airey et al., 1985). The difference between the CTX and CDSS loading modes are presented in Figure 3.3. Taking the above considerations into account, the cyclic direct simple shear apparatus at the University of British Columbia (UBC) was selected for the characterization of the response of gold tailings.

The simple shear apparatus at UBC is of the NGI type (Bjerrum & Landva, 1966), and the system is capable of applying cyclic and monotonic shear loads to the soil specimen. The specimen size used in the apparatus is approximately 20 mm high and 70.4 mm in diameter. The cylindrical soil specimen is surrounded by a reinforced rubber membrane, which constrains the specimen from deforming laterally (radially). As such, the specimen would be in a state of zero lateral strain during consolidation and cyclic loading, which is considered to closely simulate zero horizontal normal strain conditions (on the vertical plane) encountered during “at rest” static
conditions, or typical “free field” conditions under earthquake loading. Simple shear can be performed in undrained or constant volume conditions. In the conventional undrained condition, all drainage in a saturated sample would be suspended. Alternatively, in a constant volume DSS test, the specimen diameter is constrained by the reinforced membrane, and any vertical deformation of the specimen is constrained by clamping the top and bottom loading caps against vertical movement - thereby, keeping the overall volume of the soil skeleton constant during testing. The change in the applied vertical stress in constant volume conditions has been shown equivalent to the excess pore pressure in truly undrained test (Dyvik et al., 1987; Finn et al., 1978). As the mechanics depends on the constant-volume conditions of the soil skeleton regardless of the pore fluid, the approach is considered applicable to soil specimens that are dry as well as the silt-sized tailings from this study.

3.2.1 The loading system

The UBC-DSS apparatus consists of horizontal and vertical loading systems as shown in Figure 3.4. The vertical normal stress, $\sigma_v$, is applied to the specimen by a single acting air piston, located at the bottom of the apparatus; and the shear stress $\tau_h$ for the stress controlled monotonic or cyclic loading is supplied by a double acting frictionless air piston coupled in series with a constant speed motor drive. A smooth transition from stress-controlled to strain-controlled loading can be done if required.

Stress-controlled loading is facilitated by changing the pressure on one of the chambers of the double acting piston (Figure 3.4), while the pressure on the chamber of the piston is held constant. The pressure is supplied by an electro-pneumatic transducer, which is activated by the
voltage signal from the data acquisition and control system. Any prescribed form of cyclic loading can be applied using data acquisition system and computer. Generally, for the cyclic loading, a sinusoidal wave form chosen with certain magnitude and duration of the wave that can be changed during the test, if required. Cyclic or monotonic loading in strain-controlled loading can be applied through constant speed motor. The motor direction and speed can be changed manually as required (Sanín, 2010; Sriskandakumar, 2004).

3.2.2 Data acquisition, control system and measurement resolution

The UBC DSS device is equipped with a high-speed data acquisition and control system, which uses a 12-bit “PCL718” high-speed data acquisition card for signal input and output. The card consists of five A/D input channels and a D/A output channels (Sivathayalan, 2000).

The input channels are used to collect data from the two load cells and three linear variable displacement transducers (LVDTs). One of the channels is used to collect readings from the vertical load, and the second one is for the horizontal load. One LVDT monitors the vertical displacement and the other two are assigned to measure horizontal displacement. In order to get the highest resolution in each displacement range, one of the two LVDTs, monitoring horizontal displacement, is used for measuring small deformations, and the other one for large deformation. All transducers are excited by 5V d.c voltage supply, and the D/A output channels operate from 0 to 5V. Input signals from load cells are amplified by a factor of 1000, following the measurements’ refinement by averaging 60 readings for each data channel. About 500 sets of data per second can be collected by the high-speed data acquisition system.
<table>
<thead>
<tr>
<th>(a) DSS</th>
<th>(b) CTX</th>
<th>Stress status</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Diagram" /></td>
<td><img src="image2" alt="Diagram" /></td>
<td>Initial (static) condition</td>
</tr>
<tr>
<td>√</td>
<td>√</td>
<td>Cyclic loading conditions</td>
</tr>
<tr>
<td><img src="image3" alt="Diagram" /></td>
<td><img src="image4" alt="Diagram" /></td>
<td>Principle stress direction during cyclic loading</td>
</tr>
</tbody>
</table>

Figure 3.3 Comparison of stress conditions in (a) Simple Shear testing – DSS and (b) Triaxial testing – CTX
Figure 3.4 Schematic diagram of UBC simple shear test device (modified from Sriskandakumar, 2004)

The D/A channel controls the electro-pneumatic transducer, regulating the air pressure in one of the chamber of the horizontal loading double acting air piston. The electro-pneumatic transducer is of “SMC IT2051-N33” type, manufactured by SMC PNEUMATICS, which is capable of giving 90 kPa full scale pressure output for a 1000 kPa input pressure. The above carefully selected measurement devices (transducers) and a sophisticated data acquisition system allows obtaining high-resolution measurements. The data reduction program takes into account shaft friction on the horizontal loading ram, stiffness of reinforced membrane, and spring forces from the LVDTs. The resolution of each measurement for a typical 70 mm diameter and 20 mm height sample are presented in Table 3.2
Table 3.2 Measurement resolution of UBC-DSS device

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Resolution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical/Normal stress</td>
<td>± 0.25 kPa</td>
</tr>
<tr>
<td>Horizontal/Shear stress</td>
<td>± 0.25 kPa</td>
</tr>
<tr>
<td>Horizontal/Shear strain</td>
<td></td>
</tr>
<tr>
<td>Small range (&lt;13%)</td>
<td>± 0.01%</td>
</tr>
<tr>
<td>Large range (&gt;13%)</td>
<td>± 0.05%</td>
</tr>
<tr>
<td>Vertical strain</td>
<td>± 0.01%</td>
</tr>
</tbody>
</table>

3.3 DSS Test procedure

As indicated earlier, only limited tests were conducted on specimens prepared from “undisturbed” tube gold tailings samples; this is because of the lack of availability of sufficient quantity of such samples received from CU. Therefore, most tests in this study were performed on specimens reconstituted from the available quantity of gold tailings.

As the first step, the DSS device was assembled for receiving the tailings specimen following extrusion and trimming. Previously boiled in de-aired water for saturation, the porous stone was placed in the bottom pedestal of the device. The reinforced rubber membrane was attached at the bottom, and then sealed with the bottom pedestal using an o-ring. The reinforced DSS membrane was stretched around a split mold with applied vacuum to create a cavity in the DSS device to receive the specimen.
3.3.1 Preparation of specimens from relatively undisturbed tube samples

An upright extruder at the UBC laboratory was used for extruding the tube samples (see Figure 3.5a). The soil in the tube sample was extruded into a stainless steel ring with sharp cutting edge having the same diameter (70-mm) as the sampling tube. The stainless steel ring was positioned right above and in alignment with the axis of the tube during this process. Having the ring around the specimen, allowed careful confinement and securing of the soil specimen against disturbance after extrusion from the tube. The thickness of a given extruded specimen was approximately 40 mm. The specimen secured in the steel ring as per above was trimmed at the top and bottom ends to obtain a specimen height of approximately 20 mm. The soil trimmings were used for water content measurements.

The stainless steel containing the specimen was placed above the cavity of the DSS device, and then the specimen was slightly and uniformly pushed inside the cavity by a plastic plunger at this point. When the specimen is inside the DSS mold, the top cap was lowered to meet with the top surface of the specimen, and seating pressure of ~10 kPa was gently applied using the vertical loading system. Once the specimen was in place as per above, the upper end of the rubber membrane was sealed against the upper end platen using an o-ring. The steps of sample preparation and DSS set-up are illustrated in Figure 3.5.

All the transducers were positioned accordingly and initial outputs were set to zero values prior to beginning of testing. All transducer readings were monitored while removing the split mold, to make sure that no undesirable movements or loads were imparted during this process.
3.3.2 Preparation of reconstituted specimens

The reconstituted specimens were prepared from a saturated slurry as per below, essentially following the procedure previously described by (Sanin, 2010). The tailings soil was mixed thoroughly with de-aired water until a uniform and homogeneous paste was formed. Then, the paste was left under vacuum for at least 24 hours. While under vacuum, the sample was re-mixed regularly to minimize entrapped air bubbles and to obtain saturated homogenous slurry. The material was then carefully spooned to the DSS mold and moisture content measurement was
O-rings were used to protect the DSS specimens from any material loss due to squeezing, when loading to the desired vertical effective stress level in an incremental way.

### 3.4 Test procedure

Once set up for testing as per above, a given specimen was initially consolidated vertically and then subjected to monotonic or cyclic shear loading cycles as described below.

#### 3.4.1 Consolidation phase

The vertical stress on the specimen is increased to the target value required by the test program during this loading phase. The change in height of the specimen with respect to the elapsed time was continuously recorded by data acquisition system. The specimens were allowed to consolidate under the applied stress level until the vertical deformation of the specimen reached a plateau (i.e., sufficient time allowed for completion of primary consolidation).

#### 3.4.2 Static shear bias application phase

For the tests requiring application of initial static shear bias, the desired shear stress was applied incrementally under drained conditions until vertical and shear strains were stabilized. The static shear stress was applied in increments of 2.5 kPa.

#### 3.4.3 Shear loading phase

Following the consolidation phase, the specimen was constrained against vertical deformations by clamping the vertical loading ram (see Figure 3.4); this allowed maintaining constant volume conditions throughout monotonic or/and cyclic loading. Depending on whether the test was
stress or strain controlled, the horizontal shear loading was applied either by means of double acting piston or a constant speed motor respectively. Cyclic tests were conducted under stress-controlled conditions; the shear load was applied in the form of sinusoidal wave with a frequency of 0.1 Hz. For the current study the DSS tests, as mentioned above, were performed under constant volume conditions, which was achieved by controlling the external boundaries. In such tests pore water does not participate in volume control, therefore there is no need to measure pore pressure. The vertical effective stress of the sample at any given time is directly obtained from the measured load at the vertical stress boundary. Taking into account all of the above, the chosen loading frequency of 0.1 Hz is considered suitable. This approach has been in use for constant volume cyclic DSS testing of silty and sandy soils at UBC over the last 25 years.

Monotonic tests were conducted under strain-controlled conditions, where the strain rate of 10% per hour was used. Cyclic shearing was terminated once a limiting shear strain of approximately 3.75% was achieved. In monotonic tests, shearing was continued until reaching approximately 20% shear strain.

### 3.4.4 Post-cyclic reconsolidation

Upon completion of the constant volume cyclic loading described above, some specimens were reconsolidated to the initial confining stresses for assessing potential post-cyclic volumetric deformations. The specimen top cap position was manually returned to an approximately shear strain of zero position prior to reconsolidation, as the specimens had a residual shear strain at the end of monotonic loading.
3.5 Test program

The main objective of the current study was to characterize the fundamental mechanical response of gold tailings and in turn, contribute to the development of analytical tools to model the field problems involving this material. Tests were conducted to examine the following with respect to gold tailings:

- effect of stress level on undrained monotonic shear response of normally consolidated undisturbed and reconstituted tailings in direct simple shear;
- cyclic shear response of normally consolidated reconstituted tailings;
- effect of initial static shear bias on cyclic shear response of normally consolidated reconstituted gold tailings;
- comparison between the undisturbed and reconstituted tailings specimens with respect to the monotonic shear response;
- effect of mechanical overconsolidation on undrained monotonic and cyclic shear response of overconsolidated reconstituted tailings;
- post-cyclic reconsolidation response of normally consolidated reconstituted tailings;
- post-cyclic monotonic shear response of normally consolidated and overconsolidated reconstituted gold tailings.

A summary of the proposed test program describing the test series and the parameters examined are presented in Table 3.3.
Table 3.3 Summary of the proposed test program

<table>
<thead>
<tr>
<th>Test series</th>
<th>Material/ Tests</th>
<th>Nominal $\sigma'_{vc}$ (kPa)</th>
<th>CSR= $\tau_{cy}/\sigma'_{vc}$</th>
<th>Static bias $\alpha= \tau_0/\sigma'_{vc}$</th>
<th>OCR</th>
<th>No. of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>“Undisturbed” gold tailings (NC) and reconstituted gold tailings (NC): Effect of confining stress and fabric</td>
<td>50-400</td>
<td>Monotonic</td>
<td></td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>II</td>
<td>Reconstituted gold tailings (OC): Effect of OCR on monotonic response</td>
<td>50</td>
<td>Monotonic</td>
<td></td>
<td>4-8</td>
<td>2</td>
</tr>
<tr>
<td>III</td>
<td>Reconstituted gold tailings (NC): Effect of static bias on cyclic response</td>
<td>50</td>
<td>0.1-0.7</td>
<td>0.05-0.1</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>IV</td>
<td>Reconstituted gold tailings (OC): Effect of OCR on cyclic response and post-consolidation response</td>
<td>50</td>
<td>0.1-0.7</td>
<td>0</td>
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Chapter 4: Monotonic and cyclic response of gold tailings

A detailed examination of the DSS test results from the current study is presented in this chapter. As indicated previously, the work was conducted with the main objective of characterizing the monotonic and cyclic response of gold tailings for creating a database and contributing to the development of analytical modeling tools. The experimental findings on the effect of stress level and specimen preparation on monotonic response of normally consolidated relatively undisturbed and reconstituted specimens, effect of mechanical overconsolidation on monotonic response of overconsolidated reconstituted specimens are discussed in the first part of the chapter. Next, cyclic shear loading response of normally consolidated reconstituted gold tailings, followed by the discussion on the effect of initial static shear bias, mechanical overconsolidation on cyclic response of reconstituted gold tailings specimens. Data from monotonic shear tests is presented first followed by the data derived from cyclic tests. All the results from monotonic and cyclic tests are then compared with those from other similar geomaterials to obtain a better understanding of the mechanical behaviour of gold tailings in general. The key results from the testing are summarized along with the applied test parameters in Table 4.1.

4.1 Monotonic shear loading response

Results of constant volume monotonic direct simple shear tests performed on relatively undisturbed and reconstituted specimens of gold tailings as described in Chapter 3 of this Thesis are summarized in this Section. The main objective of this investigation was to study: (i) the influence of initial effective confining stress by consolidating specimens to different vertical consolidation stress levels; (ii) the effect of specimen preparation; and (iii) the effect of mechanical overconsolidation.
Table 4.1 Test parameters and summary results

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<th>Test series</th>
<th>Test ID</th>
<th>(w_i)ave</th>
<th>e_i</th>
<th>OCR</th>
<th>σ'_vc (kPa)</th>
<th>e_c</th>
<th>Cyclic tests</th>
<th>PCC*</th>
<th>ε_v-pc (%)</th>
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*PCC: Post cyclic consolidation test data
4.1.1 The effect of initial effective confining stress on monotonic response of normally consolidated reconstituted gold tailings

The tests conducted on specimens initially consolidated to different effective vertical confining stress ($\sigma'_vc$) levels, between 50 to 400 kPa, are examined herein. All the tests were conducted on reconstituted specimens were normally consolidated prior to shearing. However, the initial stress history of the tests conducted on specimens directly extruded from tube soil samples received from Carleton University could not be well defined. As mentioned in Chapter 3, the relatively undisturbed samples from Carleton University were prepared by simulating thickened tailings deposition method. The tailings paste was placed layer-by-layer, and then it was partially desiccated; the latter process caused the sample to desiccate and experience some overconsolidation. Since the degree of desiccation of these samples is unavailable, the specimens directly prepared from tube samples deposited at Carleton University are referred to as “relatively undisturbed”. In spite of these limitations, it was decided to compare the monotonic behaviour of relatively undisturbed specimens directly prepared from tube samples deposited at Carleton University with those prepared by reconstitution.

The effect of vertical effective stress on stress-strain response, excess pore pressure generation, and stress path derived from constant volume monotonic DSS tests on reconstituted specimens of gold tailings normally consolidated to varying confining stress levels are presented in Figure 4.1. As may be noted from Figures 4.1b and 4.1c, the specimens initially deformed in a contractive manner, and then followed by a dilative response. The phase transformation occurred at the same mobilized shear stress ratio ($\tau/\sigma'_vc$) for all reconstituted specimens, although the tests were conducted at different vertical effective stresses. This observation is in accord with the
observations made by Negussey et al. (1988) for coarse-grained soils and Sanin & Wijewickreme (2006) for low-plastic natural silts. Figure 4.1a also demonstrates that, as expected, the stiffness and strength of the specimens increase with increasing initial confining stress level.

The above stress-strain, excess pore pressure, and stress path responses were normalized with respect to their initial effective vertical consolidation stress, $\sigma'_{vc}$, and the results are shown in Figure 4.2. The normalized behaviour shows that stress-paths follow a consistent pattern and fall within a narrow range. This stress history normalizability is similar to the typical response of normally consolidated clays and low-plastic silts noted by Atkinson & Bransby (1978) and Sanin & Wijewickreme (2006).

The results from monotonic tests on relatively undisturbed normally consolidated gold tailings are presented together with those obtained from tests conducted on reconstituted normally consolidated specimens on Figure 4.3. As noted in Table 4.1, initial void ratios for the reconstituted specimens are in the order of 0.61-0.77. This is significantly higher than the void ratios for undisturbed specimens that fall in the range of 0.56-0.58. In spite of this difference, it was considered that it would be of value to superimpose and compare the basic behavioural characteristics. All reconstituted specimens displayed a more contractive response than those exhibited by the undisturbed specimens tested at the same stress levels. Clearly, the response of the relatively undisturbed specimens is significantly stiffer and stronger than the response of the reconstituted specimens. Leroueil & Hight (2003) and Wijewickreme & Sanin (2008) observed similar trend when studying difference in mechanical response between normally consolidated reconstituted and naturally deposited soils. However, the difference in the response between
Figure 4.1 Constant volume monotonic DSS tests on reconstituted normally consolidated specimens of gold tailings at varying confining stress levels: a) shear stress-strain response, b) excess pore water pressure-strain response, c) stress paths; \( \sigma'_{vc}=50 \text{ kPa} \), \( \sigma'_{vc}=100 \text{ kPa} \), \( \sigma'_{vc}=200 \text{ kPa} \) and \( \sigma'_{vc}=400 \text{ kPa} \)
Figure 4.2 Constant volume monotonic DSS tests on reconstituted normally consolidated specimens of gold tailings at varying confining stress levels: a) normalized shear stress-strain response, b) excess pore water pressure ratio-strain response, c) normalized stress paths; $\sigma_{vc}' = 50$ kPa, 100 kPa, 200 kPa and 400 kPa
Figure 4.3 Constant volume monotonic DSS tests on undisturbed and reconstituted normally consolidated specimens of gold tailings at varying confining stress levels: a) shear stress-strain response, b) excess pore water pressure-strain response, c) stress paths; $\sigma'_{vc} = 50$ kPa, 100 kPa, 200 kPa and 400 kPa
relatively undisturbed and reconstituted gold tailings, used for this study, might be due to partial desiccation of relatively undisturbed samples during sample preparation at Carleton University, lower void ratios in the relatively undisturbed specimens and/or different specimen preparation methods (fabric). As such, firm conclusions cannot be made without further experimentation.

The results of undrained monotonic tests conducted on mechanically overconsolidated reconstituted specimens of the same gold tailings material are presented in Figure 4.4. Figure 4.5 presents the same results re-plotted after normalizing. As may be noted, the shear strength and stiffness of the material is noted to increase with increasing OCR. The reconstituted specimens with OCR = 4 and 8 after 10% shear strain show strain-softening behaviour, which is typical for lightly overconsolidated clays Mitchell & Soga (2005).

The overconsolidated specimens show a dilative tendency under shearing compared to the counterpart normally consolidated specimens, as indicated by the increase in $\sigma'/\sigma'_{vc}$ in Figure 4.4c. This tendency continues until the peak resistance is reached. Then, the initial increase is followed by a decrease in shear resistance (Figure 4.4c).

The pore pressure versus strain curves for specimens with OCR = 4 and 8 show decrease in $\Delta u/\sigma'_{vc}$ (negative pore pressures) followed by an increase. The higher the OCR, the more negative the pore pressures and the smaller the final value of decrease in $\Delta u/\sigma'_{vc}$. Malek (1987), and Ladd & Edgers (1972) have observed similar behaviour when studying monotonic shearing response of overconsolidated clays.
Figure 4.4 Constant volume monotonic DSS tests on reconstituted mechanically overconsolidated specimens of gold tailing at varying confining stress levels: a) shear stress-strain response, b) excess pore water pressure-strain response, c) stress paths; OCR=1, 4 and 8
Figure 4.5 Constant volume monotonic DSS tests on reconstituted mechanically overconsolidated specimens of gold tailings a) normalized shear stress-strain response, b) excess pore water pressure ratio-strain response, c) normalized stress paths; OCR=1, 4 and 8
The normalized shear strength and excess pore pressure ratio values (Figures 4.5a and 4.5b) for reconstituted specimens at different OCR are similar to those observed by Kim et al. (2011), when testing the same reconstituted material using DSS apparatus at Carleton University. It is however, important to note that the loading rate used at Carleton University was 20% per hour compared to the shearing rate used for this study, which was 10% per hour.

### 4.2 Cyclic shear response

The results from the detailed experimental investigation to assess the cyclic shear response of gold tailings are summarized in the subsequent section. The main objectives of the analysis are as follows: (i) examine cyclic behaviour of normally consolidated reconstituted gold tailings specimens; (ii) compare the findings with the previously published information on the cyclic behaviour of similar fine-grained tailings and soil materials; (iii) assess the effect of the initial static shear bias on the cyclic behaviour of normally consolidated reconstituted gold tailings specimens (i.e., simulating typical slopes encountered in deposits formed by thickened tailings method); (iv) assess the effect of mechanical overconsolidation on the cyclic behaviour of normally consolidated reconstituted gold tailings.

#### 4.2.1 Cyclic shear response of normally consolidated reconstituted tailings

The results of constant volume cyclic direct simple shear tests performed on normally consolidated reconstituted specimens of gold tailings are summarized in this subsection.
4.2.1.1 General stress-strain, stress-path and pore water pressure response

The typical stress path, stress-strain, pore pressure ratio and shear strain development during cyclic loading of normally consolidated reconstituted tailings are shown in Figure 4.6; the data obtained from the DSS test carried out on the reconstituted gold tailings. The specimen experiences a cumulative increase in equivalent excess pore-water pressure with associated

Figure 4.6 The response of reconstituted gold tailings in constant-volume cyclic DSS loading (sample BT016OC1: $\sigma'_v = 50$ kPa, CSR=0.16, OCR=1): a) stress-strain response; b) stress path response; c) shear strain versus number of cycles; d) equivalent excess pore pressure ratio versus number of cycles of normally consolidated reconstituted gold tailings
progressive degradation of shear stiffness. Unlike typical cyclic response of loose or medium dense sand behaviour, the vertical effective stress does not reach zero state. However, it drops significantly to a minimum value close to zero. This cyclic mobility type stress-strain response is similar in form to undrained cyclic shear responses observed from cyclic shear tests on natural clayey soils and natural low plastic silts (Wijewickreme & Sanin, 2004; Zergoun & Vaid, 1994), dense reconstituted sand (Kammerer, 2002; Sriskandakumar, 2004) and other fine-grained tailings (Wijewickreme et al. 2005). The stress-strain and stress path responses for reconstituted normally consolidated specimens of gold tailings with different CSR values are presented in Figures 4.7 through 4.10. The variation of excess pore water pressure \( r_u = \Delta u / \sigma'_{vc} \) vs. number of loading cycles for the cyclic tests conducted with \( \sigma'_{vc} = 50 \text{ kPa} \) at different cyclic stress ratios are presented in Figure 4.11. All the specimens exhibit gradual increase of \( \Delta u \) or the rate of generation of equivalent excess pore water pressure ratio \( r_u \) with increasing applied CSR. For example, the specimen with CSR=0.35 developed \( r_u = 81\% \) in 1 cycle; whereas, the specimen tested at the same level of vertical effective stress with CSR=0.26 only reached \( r_u = 71\% \) even
after almost 3 cycles. The specimen with CSR=0.16 reached \( r_{ui} = 92\% \) after the application of 21 number of loading cycles, while the specimen with CSR=0.35 reached the same level of excess pore water pressure ratio after 200 number of cyclic loading.

Figure 4.8 Constant volume cyclic simple shear DSS test on reconstituted specimen of normally consolidated gold tailings: stress strain and stress path curves; \( \sigma'_{vc} = 50 \) kPa; CSR=0.16

Figure 4.9 Constant volume cyclic simple shear DSS test on reconstituted specimen of normally consolidated gold tailings: stress strain and stress path curves; \( \sigma'_{vc} = 50 \) kPa; CSR=0.26
Figure 4.10 Constant volume cyclic simple shear DSS test on reconstituted specimen of normally consolidated gold tailings: stress strain and stress path curves; $\sigma'_v=50$ kPa; CSR=0.35

Generally, it can be stated that all the specimens initially show cumulative contractive response, and with increasing number of load cycles, the specimens display cumulative increase in excess pore water pressure with associated progressive degradation of shear stiffness. In a given cycle, the shear stiffness experiences its transient minimum when the applied shear stress is close to zero (when the effective stress is minimum). It is important to note that the shear response gradually changed from contractive to dilative (or experienced phase transformation) during the “loading” (or increase in shear stress) phases. Then the specimens were deforming in a contractive manner during the “unloading” phases (or decrease in shear stress) suggesting significant plastic volumetric strains during unloading, particularly in specimens that had experienced phase transformation.
Figure 4.11 Constant volume cyclic simple shear DSS tests on reconstituted specimen of reconstituted normally consolidated gold tailings: excess pore water pressure vs. number of cycles for different CSR values; $\sigma_{vc}' = 50$ kPa

4.2.1.2 Cyclic shear resistance of normally consolidated reconstituted gold tailings

There are many factors influencing the cyclic shear resistance of a soil; namely: the confining pressure, density, initial static shear stress bias, and specimen preparation methods, etc. (Mitchell & Soga, 2005; Seed, 1979; Seed & Harder, 1990).

The cyclic shear resistance is often expressed using the parameter cyclic resistance ratio (CRR), where CRR is defined as $\tau_{cy}/\sigma_{vc}'$ in DSS tests and $(\sigma_{d})_{cy}/2 \sigma_{vc}'$ in triaxial tests (Note: $\tau_{cy}$ is the cyclic shear stress in DSS test and $(\sigma_{d})_{cy}$ is the cyclic deviator stress in triaxial tests). Herein, the CRR is defined with respect to the instance when the single-amplitude cyclic shear strain ($\gamma$) in a DSS specimen has reached $\gamma=3.75\%$. This is equivalent to reaching $\varepsilon = 2.5\%$ single-amplitude axial strain in a triaxial soil specimen. This definition of the limiting shear strain was suggested by the U.S. National Research Council (NRC 1985) for assessment of the cyclic shear resistance of sands, and it has been adopted in previous studies of cyclic behaviour of soils at UBC.
Figure 4.12 Cyclic shear resistance to reach shear strain $\gamma = 3.75\%$ in sample included in this study in comparison with the results on cyclic response of similar fine-grained tailings published by different research groups. SA, single amplitude; DA, double amplitude; $\gamma$, shear strain; $\varepsilon_a$, axial strain in cyclic triaxial tests.
The change in applied cyclic stress ratio (CSR) versus number of cycles required to reach limiting shear strain in cyclic shear tests on normally consolidated reconstituted gold tailings specimens consolidated to vertical effective stress of $\sigma'_{vc} = 50$ kPa is presented in Figure 4.12. Additionally, the previously published cyclic shear resistance values for other types of tailings derived from both cyclic triaxial (CTX) and DSS testing as compiled by Wijewickreme et al. (2005) are superimposed on the same figure. As may be noted, compilations for most tailings presented on the plot have been extracted from Vick (1990).

As noted by others, the cyclic shear resistance derived from tailings tested using the cyclic triaxial device (CTX) (see Figure 4.12) is considerably less compared to the strength degradation of the tailings derived from DSS. This is mainly attributed to the different mode of loading between DSS and triaxial tests (Donaghe & Gilbert, 1983; Finn et al., 1978; Roscoe, 1970; Seed & Peacock, 1971; Wijewickreme et al. 2005). The discrepancy between CTX and DSS exists and

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Figure 4.13 Grain-size distribution of Fraser River sand, Fraser River silt and Quartz (extracted from Sanin, 2010)
Figure 4.14 CRR to reach $\gamma = 3.75\%$ in reconstituted normally consolidated specimens this study and CRR of natural fine-grained and coarse-grained materials reported by others. SA, single amplitude; DA, double amplitude; $\gamma$, shear strain; $\epsilon_a$, axial strain in cyclic triaxial tests

it is discussed in Wijewickreme et al. (2005). The DSS data from the current study fall in the lower part of the plot compared to the results obtained from cyclic triaxial tests derived by other researches.

Moreover, the results for reconstituted gold tailings from this study were also compared with the CRR obtained for some other soils – i.e., low plastic natural Fraser River silt, quartz rock powder, and Fraser River sand by Sanin & Wijewickreme (2006), Sanin (2010), and
Sriskandakumar (2004). The comparison is presented on the Figure 4.13, and the corresponding grain-size distributions for the soils are presented on the Figure 4.14. As may be noted from Figure 4.13, the cyclic resistance curve for the gold tailings lies above the cyclic resistance curves for Fraser River sand and Quartz, which is in accord with the previous observations. It is of interest to note that the CRR curve for the gold tailings studied herein is in the same range as that for natural Fraser river silt.

4.2.2 The effect of initial static shear loading on cyclic shear response of normally consolidated reconstituted tailings

As indicated in Table 4.1, the effect of static shear stress bias was investigated for two different values of $\alpha$, $(\tau_o/\sigma'_v)$: 0.05 and 0.1. The cyclic loading stress-strain, stress path response and shear strain and equivalent excess pore pressure ratio versus number of cycles at CSR = 0.15 with initial static bias of 0.1, are presented on the Figure 4.15. Since the amplitude of the initial static shear stress is not larger than the cyclic shear stress amplitude, shear stress reversal can be observed, which is shifted more towards positive direction as illustrated on the Figure 4.15b. The induced pore water pressure increases rapidly in the first several cycles, then it increases slower, but it never reaches 1. The build-up of equivalent excess pore water pressure with the increasing number of cycles could result in significant cyclic shear strains even at moderate levels of cyclic loading. The shear strain is also shifted in the direction of the static shear stress; and it increases quite progressively with each subsequent cycle. The deformation progresses due to cyclic mobility, but without transient states of zero effective stress, similar to the cyclic response of the gold tailings without static bias presented above. The cyclic loading stress-strain and stress path
Figure 4.15 Typical response of reconstituted gold tailings in constant volume cyclic DSS loading with initial static shear loading (sample BT015B01: \(\sigma'_{vc}=50\,\text{kPa},\,\text{CSR}=0.15,\,\alpha=0.1\)):

- a) stress-strain response;
- b) stress path response;
- c) shear strain versus number of cycles;
- d) equivalent excess pore pressure ratio versus number of cycles

The variation of excess pore water pressure (\(r_u=\Delta u/\sigma'_{vc}\)) vs. number of loading cycles for the cyclic tests conducted at \(\sigma'_{vc}=50\,\text{kPa}\) with different initial static shear bias at cyclic stress ratios 0.05 and 0.1 are presented in Figures 4.22 and 4.23 respectively. All the specimens show gradual increase of \(\Delta u\) with increasing applied CSR, which is similar to the specimens sheared cyclically without initial static shear bias. Figure 4.22 shows responses of specimens...
Figure 4.16 Constant volume cyclic simple shear DSS test with initial static shear loading on normally consolidated reconstituted gold tailings specimen: stress-strain and stress path curves; \( \sigma'_{vc} = 50 \text{ kPa}, \) CSR=0.15, \( \alpha = 0.05 \)

Figure 4.17 Constant volume cyclic simple shear DSS test with initial static shear loading on normally consolidated reconstituted gold tailings specimen: stress-strain and stress path curves; \( \sigma'_{vc} = 50 \text{ kPa}, \) CSR=0.25, \( \alpha = 0.05 \)
sheared cyclically at $\alpha=0.05$. Based on the Figure, the specimen with CSR=0.35 developed $r_u=80\%$ in 0.3 cycle; whereas, the specimen tested at the same level of vertical effective stress with CSR=0.25 reached approximately the same level of excess pore pressure ratio after about 2 cycles. The specimen with CSR=0.15 reached $r_u=93\%$ after the application of 19 number of loading cycles.

Figure 4.23 shows excess pore water pressure ratios for the specimens tested at $\alpha=0.01$. Apparently, for the specimen with CSR=0.30, it took less than 1 cycle to reach 91% of excess pore pressure ratio; whereas, for the specimen tested at CSR=0.15 only 83% of excess pore pressure ratio was reached after approximately 11 cycles; and finally, for the specimen tested at CSR=0.10, 74 cycles needed to be applied to reach 92% of excess pore pressure ratio.

Figure 4.24 shows excess pore water pressure ratios for the specimens tested at different initial static shear bias levels with $\alpha = 0$, 0.05 and 0.1 at approximately the same CSR level. For the specimen tested at $\alpha = 0$, or without static shear bias, the excess pore water pressure level rises to almost 67% in approximately 3 loading cycles; whereas, the specimen tested at $\alpha = 0.05$ reached 83% of excess pore pressure ratio within around 3 cycles; and in the specimen with $\alpha = 0.1$ the excess pore pressure ratio went up to 91% after only 1.5 number of loading cycles.
Figure 4.18 Constant volume cyclic simple shear DSS test with initial static shear loading on normally consolidated reconstituted gold tailings specimen: stress-strain and stress path curves; $\sigma'_{ve}=50$ kPa, CSR=0.35, $\alpha=0.05$

Figure 4.19 Constant volume cyclic simple shear DSS test with initial static shear loading on normally consolidated reconstituted gold tailings specimen: stress-strain and stress path curves; $\sigma'_{ve}=50$ kPa, CSR=0.10, $\alpha=0.1$
Figure 4.20 Constant volume cyclic simple shear DSS test with initial static shear loading on normally consolidated reconstituted gold tailings specimen: stress-strain and stress path curves; $\sigma'_{vc}=50$ kPa, CSR=0.15, $\alpha=0.1$ (Please note, Figure 4.20 is identical to Figure 4.15 and is repeated for easy comparison)

Figure 4.21 Constant volume cyclic simple shear DSS test with initial static shear loading on normally consolidated reconstituted gold tailings specimen: stress-strain and stress path curves; $\sigma'_{vc}=50$ kPa, CSR=0.30, $\alpha=0.1$
Figure 4.22 Constant volume cyclic simple shear DSS tests on reconstituted specimens of normally consolidated gold tailings with initial static shear bias: excess pore pressure vs. number of loading cycles for different CSR values: $\alpha =0.05, \sigma'_{vc}=50 \text{ kPa}$

![Diagram](image1)

Figure 4.23 Constant volume cyclic simple shear DSS tests on reconstituted specimens of normally consolidated gold tailings with initial static shear bias: excess pore pressure vs. number of loading cycles for different CSR values: $\alpha =0.1, \sigma'_{vc}=50 \text{ kPa}$

![Diagram](image2)
Figure 4.24 Constant volume cyclic simple shear DSS tests on reconstituted specimens of normally consolidated gold tailings with initial static shear bias: excess pore pressure vs. number of loading cycles for different $\alpha$ values, at approximately the same CSR level: $\sigma'_v c = 50$ kPa

The effect of the initial static shear stress on the cyclic response of the gold tailings is shown on the Figure 4.25. Clearly, the presence of the initial static shear stress has a strong effect on the cyclic shear resistance of the material. For example, for the gold tailings without initial static shear stress tested at CSR = 0.20 a shear strain of 3.75% was reached after about 9 cycles of loading; whereas, for the specimens tested with initial static shear stress $\alpha=0.05$ at the same CSR value it takes about 5 cycles to reach a shear strain of 3.75%. The specimens tested with initial static shear stress $\alpha = 0.1$, the shear strain of 3.75% was reached after approximately 2 cycles. All the specimens exhibit progressive degradation in cyclic shear strength with increasing level of initial static bias.
Figure 4.25 Cyclic resistance ratio to reach $\gamma=3.75\%$ from constant volume cyclic DSS tests on reconstituted gold tailings at varying initial static shear stress bias conditions

4.2.3 The effect of mechanical overconsolidation on cyclic shear response of overconsolidated reconstituted tailings

The results of constant volume cyclic direct simple shear tests performed on overconsolidated reconstituted specimens of gold tailings are summarized in this subsection.
4.2.3.1 General stress-strain, stress path and pore water pressure response

To study the effect of mechanical overconsolidation on the cyclic behaviour of the gold tailings, the specimens were overconsolidated to OCR values of 4 and 8; in essence, the specimens were initially normally consolidated to 200 kPa and 400 kPa vertical effective stress respectively, and then unloaded to 50 kPa vertical effective stress. Then, the consolidated specimens were cyclically loaded at similar or comparable CSR levels. The stress-strain and stress path behaviour of overconsolidated gold tailings with OCR=4 and 8 is presented in Figures 4.26 through 4.31. As shown in the figures, the CRR level is clearly observed to increase with increasing OCR.

Despite of the overconsolidation effect, the degradation of shear stiffness due to the development of pore pressures is still prevalent. Therefore, if sufficient number of cycles at relatively high cyclic stresses is applied, large shear strains during the cyclic loading are still possible. Generally, for the initially OC specimens, the mechanism of shear strain development is of cyclic mobility type, which is similar in form to those observed in normally consolidated gold tailings specimens. The overconsolidated specimens show phase transformation from contractive to dilative from the very first cycle. The dilative tendency becomes more significantly pronounced with increase in OCR and CSR.

The variation of excess pore water pressure ($r_u = \Delta u / \sigma'_{vc}$) versus number of loading cycles for the cyclic tests conducted on specimens overconsolidated to OCR = 4 and 8 are presented on Figures 4.32 and 4.33 respectively. All the specimens show gradual increase of $\Delta u$ with increasing applied CSR. Figure 4.32 shows responses of specimens overconsolidated to OCR=4 and sheared cyclically. Based on the figure, the specimen with CSR=0.55 developed $r_u=78\%$ in 0.5
cycle; whereas, the specimen tested at CSR=0.24 reached approximately the same level of excess pore pressure ratio after about 50 cycles. The specimen with CSR=0.2 reached \( r_u = 67\% \) after the application of 100 cycles; and the specimen tested at CSR=0.42 reached 57\% of excess pore pressure ratio after being exposed to only 5 cycles.
Figure 4.26 Constant volume cyclic simple shear DSS test on overconsolidated reconstituted gold tailings specimen: stress-strain and stress path curves; OCR=4, CSR=0.24

Figure 4.27 Constant volume cyclic simple shear DSS test on overconsolidated reconstituted gold tailings specimen: stress-strain and stress path curves; OCR=4, CSR=0.42
Figure 4.28 Constant volume cyclic simple shear DSS test on overconsolidated reconstituted gold tailings specimen: stress-strain and stress path curves; OCR=4, CSR=0.55

Figure 4.29 Constant volume cyclic simple shear DSS test on overconsolidated reconstituted gold tailings specimen: stress-strain and stress path curves; OCR=8, CSR=0.35
Figure 4.30 Constant volume cyclic simple shear DSS test on overconsolidated reconstituted gold tailings specimen: stress-strain and stress path curves; OCR=8, CSR=0.48

Figure 4.31 Constant volume cyclic simple shear DSS test on overconsolidated reconstituted gold tailings specimen: stress-strain and stress path curves; OCR=8, CSR=0.65
Figure 4.33 presents excess pore pressure ratio vs. shear strain response for specimens initially overconsolidated to OCR=8. Based on the figure, it took only 0.5 cycles to reach 79% of excess pore pressure ratio when tested at CSR=0.65; while, for the specimen tested at CSR=0.35 the same level of excess pore pressure ratio was reached after 58 number of cycles. The specimen tested at CSR=0.48 reached 53% of excess pore pressure ratio within only 2.5 number of cycles; whereas, the specimen tested at CSR=0.2 reached only 15% of excess pore pressure ratio even after application of 200 number of cycles.

The excess pore water pressure ratios vs. shear strain curves for the specimens tested at approximately the same CSR levels mechanically overconsolidated to different OCR values are presented on Figure 4.34. According to the figure, in normally consolidated specimen the excess pore pressure ratio reached 67% in 2.5 cycles; in the overconsolidated specimen with OCR=4, 82% of excess pore pressure ratio was achieved after 48 cycles; and only 15% of excess pore pressure ratio was reached even after 200 number of cycles in specimen with OCR=8. This implies that the higher the OCR, the less excess pore pressure level is developed.
Figure 4.32 Constant volume cyclic simple shear DSS test on reconstituted specimens of overconsolidated gold tailings: excess pore water pressure vs. number of loading cycles for different CSR values; OCR=4

Figure 4.33 Constant volume cyclic simple shear DSS test on reconstituted specimens of overconsolidated gold tailings: excess pore water pressure vs. number of loading cycles for different CSR values; OCR=8
Figure 4.34 Constant volume cyclic simple shear DSS test on reconstituted specimens of overconsolidated gold tailings: excess pore water pressure vs. number of loading cycles for different OCR values at approximately the same CSR level; OCR=1, 4 and 8

4.2.3.2 Cyclic resistance ratio of overconsolidated reconstituted gold tailings

The variation of cyclic strength obtained for the gold tailings specimens with OCR = 1, 4 and 8 are presented on the Figure 4.35. Clearly, the CRR increases considerably with increasing level of overconsolidation. Similar findings were reported by Andersen (2009) and Sanin & Wijewickreme (2006) when analyzing different types of soils. For instance, when tested at CSR=0.30, the specimens overconsolidated to OCR = 8 reach limiting shear strain at about 70 number of cycles; at the same CSR level gold tailings specimens with OCR=4 reached unacceptable strains after approximately 12 number of cycles; and overconsolidated specimens with OCR=1 reached unacceptable strains within around 2 number of cycles. Considering all of the above, there is strong dependency of the cyclic shear strength of the gold tailings material on OCR.
Figure 4.35 Cyclic resistance ratio to reach limiting shear strain value from constant volume cyclic DSS tests on reconstituted gold tailings with OCR=1, 4 and 8
4.3 Summary and principal findings

The monotonic and cyclic shear response of fine-grained gold tailings was examined using the constant-volume cyclic direct simple shear (DSS) test device. The device is considered to more closely simulate field conditions during earthquake loading compared to cyclic triaxial apparatus (CTX). The results of the current study will contribute to the improvement in understanding of the mechanical cyclic response of fine-grained tailings, and provide useful input parameters for further numerical simulation of the material as well as design of tailings storage facilities.

Under normally consolidated monotonic constant-volume shear loading, the reconstituted gold tailings specimens deformed initially in a contractive manner followed by a dilative response. Specimens consolidated to vertical effective stress levels from 50 kPa to 400 kPa exhibited identical mobilized shear stress ratios at phase transformation. When normalized, the stress paths followed a consistent pattern and fell within a narrow range, suggesting that the observed monotonic behaviour is similar to that noted previously for normally consolidated clays.

Monotonic shear response of overconsolidated gold tailings specimens, as expected, indicated increasing shear resistance and stiffness with increasing OCR. Reconstituted specimens with OCR = 4 and 8 displayed strain - hardening monotonic shear loading response, followed by the strain - softening after reaching ~10% shear strain. Overconsolidated specimens developed negative excess pore pressures during shearing, suggesting increased dilative response with increasing OCR. This behaviour is similar to those typically observed for overconsolidated clays.
The data from cyclic tests provides valuable initial information on the stress-history and initial sloping ground condition effects on the cyclic behaviour of the gold tailings. Typically, fine-grained tailings show a cumulative decrease in effective stress (or increase in equivalent excess pore-water pressure) with increasing number of loading cycles, which causes progressive degradation of shear stiffness. Similar cyclic mobility type of response has been observed by others during previously conducted research on natural clays, low-plastic silts, similar fine-grained tailings, and reconstituted dense sands.

Based on the specimens tested at different initial static shear stresses, it was observed that cyclic resistance ratio (CRR) (i.e., CSR to reach number of cycles to reach $\gamma = 3.75\%$) would decrease with increasing level of initial static shear stress bias. The tests on overconsolidated gold tailings specimens, as expected, indicated that the CRR and shear stiffness would increase with increasing OCR.

The results from this work on normally consolidated reconstituted specimens were also compared to the available published data on the cyclic response of different tailings, obtained from tests carried out on cyclic triaxial (TX) and DSS devices. The comparison indicates that the findings from this research work are in good agreement with those from past work. The CRR of the gold tailings material used in this study was found to be higher than that observed in Fraser river sand and Quartz rock powder, but in the same range as Fraser river silt.
Chapter 5: Post-cyclic response of gold tailings

The assessment of post-cyclic shear strength and post-cyclic reconsolidation strains are useful for stability and deformation evaluations as well as performance of existing structures founded on liquefiable soil masses. Seed & Harder (1990), Stark & Mesri (1992) and Olson & Stark (2002) have developed correlations between liquefied strength $S_u$(LIQ), or liquefied strength ratio $S_u$(LIQ)/$\sigma'_{vc}$, and in-situ standard penetration values computed by back-analyses of field case histories. Such back-analysis for estimation of $S_u$(LIQ) has been considered more suitable, since laboratory testing is not able to simulate the void redistributions, or water film effects, that take place after liquefaction, particularly in layered deposits with contrasting permeability (Kokusho, 2003).

With this in mind, and taking into account the current lack of understanding and limited information available on the subject, it was considered of value to study the post-cyclic behaviour of gold tailings in the laboratory as part of the current research program. In particular, the post-cyclic shear strength of normally consolidated and overconsolidated reconstituted gold tailings is examined. The post-cyclic reconsolidation volumetric settlements, due to dissipation of excess pore water pressures developed during cyclic loading, are presented and, the current findings are compared with previously published results on other soil materials. The testing procedure of the post-cyclic reconsolidation is presented in Section 3.3.
5.1 Experimental observations

The results of constant volume post-cyclic monotonic direct simple shear tests performed on normally consolidated and overconsolidated reconstituted specimens of gold tailings and post-cyclic reconsolidation tests of normally consolidated reconstituted specimens of the same material are summarized in this subsection.

5.1.1 Post-cyclic monotonic shear strength response

The response of reconstituted overconsolidated gold tailings specimens with OCR = 1, 4, and 8 preliminary cyclically loaded to different CSR values observed under post-cyclic undrained monotonic shearing (conducted on specimens after reaching $\gamma = 3.75\%$ strain criteria during cyclic loading) are plotted in Figures 5.1 through 5.3. The post-cyclic maximum undrained shear strength ($S_{u\text{-PC}}$) was estimated from the available post-cyclic monotonic DSS test data for the gold fine-grained tailings. The $S_{u\text{-PC}}$ thus obtained for all the DSS tests were then normalized with respect to the initial vertical effective consolidation stress ($\sigma'_{vc}$) to obtain the post-cyclic maximum shear strength ratio ($S_{u\text{-PC}}/\sigma'_{vc}$). The corresponding undrained post-cyclic shear strength ($S_{u\text{-PC}}$), initial vertical effective consolidation stress ($\sigma'_{vc}$), post-cyclic shear strength ratio ($S_{u\text{-PC}}/\sigma'_{vc}$), cyclic stress ratios (CSR), and overconsolidation ratios (OCR) are summarized in Table 5.1.

The post-cyclic shear stress-strain response has a very low initial shear stiffness - typical of soils that developed significant excess pore water pressures during cyclic loading. With increasing shear strain level, the specimens clearly exhibited dilative response along with increasing shear stiffness; with further shearing, the shear stiffness drops until shear strength seems to reach a
peak (or a plateau) at relatively large strains. It is noted that measured stresses at large strains may be subject to error due to potential high stress and strain non-uniformities within DSS specimens. The early part of the post-cyclic stress strain response, with initially very low and subsequent build-up of shear stiffness, is also similar to those reported by Vaid and Sivathayalan (1997) from their simple shear tests on water-pluviated sands under simple shear loading.
Figure 5.1 Post-cyclic monotonic shear response of overconsolidated reconstituted gold tailings specimens with OCR = 1 (initial consolidation stress conditions under cyclic loading are given in legend) – (a) stress-strain response; (b) excess pore water pressure response; and (c) stress path response.
Figure 5.2 Post-cyclic monotonic shear responses of overconsolidated reconstituted gold tailings specimens with OCR = 4 (initial consolidation stress conditions under cyclic loading are given in legend) – (a) stress-strain response; (b) excess pore water pressure response; and (c) stress path response.
Figure 5.3 Post-cyclic monotonic shear responses of overconsolidated reconstituted gold tailings specimens with OCR = 8 (initial consolidation stress conditions under cyclic loading are given in legend) – (a) stress-strain response; (b) excess pore water pressure response; and (c) stress path response.
Figure 5.4 presents a plot of $S_u/\sigma_v'$ vs. the corresponding overconsolidation ratio (OCR). A clear trend of increasing post-cyclic shear strength ratio with increase in OCR (or increasing density) can be noted. The post-cyclic shear strength ratios for normally consolidated gold tailings specimens ranges from 0.4 to 1.1; for overconsolidated specimens with OCR = 4 the range for post-cyclic shear strength varies between 0.6 to 0.8; and for overconsolidated specimens with OCR=8 the $S_u/\sigma_v'$ varies between from 1.2 to 1.9. Except for the $S_u/\sigma_v' = 1.1$ for OCR that might be considered as an outlier, general increase in $S_u/\sigma_v'$ with OCR can be noted. This behaviour is in alignment with liquefied strength ratio ($S_u$-LIQ) vs. normalized
penetration resistance (i.e., packing density) correlations suggested by Olson & Stark (2002), based on back analysis of 33 case histories. Wijewickreme et al. (2005) observed a similar trend, when studying post-cyclic shear strength of fine-grained tailings under different initial consolidation void ratios. However, further testing is required to make any firm conclusions.

The post-cyclic data obtained from laboratory testing should be used with caution. Laboratory post-cyclic shear strength may be higher than the post-cyclic strength obtained from the field due

Table 5.1 Summary of post-cyclic undrained shear strength of normally and overconsolidated reconstituted gold tailings

<table>
<thead>
<tr>
<th>OCR</th>
<th>CSR=$\tau_{\text{cyc}}/\sigma_v'$</th>
<th>$S_{u-PC}$ (kPa)</th>
<th>$\sigma_v'$ (kPa)</th>
<th>$S_{u-PC}/\sigma_v'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1</td>
<td>19.2</td>
<td>52.1</td>
<td>0.4</td>
</tr>
<tr>
<td>1</td>
<td>0.16</td>
<td>28.7</td>
<td>45.3</td>
<td>0.6</td>
</tr>
<tr>
<td>1</td>
<td>0.26</td>
<td>43.2</td>
<td>40.0</td>
<td>1.1</td>
</tr>
<tr>
<td>1</td>
<td>0.35</td>
<td>20.7</td>
<td>47.8</td>
<td>0.4</td>
</tr>
<tr>
<td>4</td>
<td>0.2</td>
<td>33.5</td>
<td>50.9</td>
<td>0.7</td>
</tr>
<tr>
<td>4</td>
<td>0.24</td>
<td>31.2</td>
<td>50.4</td>
<td>0.6</td>
</tr>
<tr>
<td>4</td>
<td>0.42</td>
<td>42.8</td>
<td>50.8</td>
<td>0.8</td>
</tr>
<tr>
<td>4</td>
<td>0.55</td>
<td>37.6</td>
<td>51.1</td>
<td>0.7</td>
</tr>
<tr>
<td>8</td>
<td>0.2</td>
<td>100.1</td>
<td>51.4</td>
<td>1.9</td>
</tr>
<tr>
<td>8</td>
<td>0.35</td>
<td>60.2</td>
<td>48.8</td>
<td>1.2</td>
</tr>
<tr>
<td>8</td>
<td>0.48</td>
<td>73.5</td>
<td>50.8</td>
<td>1.4</td>
</tr>
<tr>
<td>8</td>
<td>0.65</td>
<td>66.3</td>
<td>51.0</td>
<td>1.3</td>
</tr>
</tbody>
</table>
to number of different factors. In the laboratory, the drainage conditions can be controlled; and the specimens are usually tested under undrained conditions. In the field, however, based on the observations of field flow failure, drainage may occur. As a result, the shear strength mobilized in the field may not be undrained (Fiegel & Kutter, 1994; Olson & Stark, 2002; Stark & Mesri, 1992). In literature, the term “liquefied shear strength” is often used to describe the shear strength that is mobilized during liquefaction flow failure. However, drainage, pore-water pressure redistribution, soil mixing and other potential effects should be taken into account, while interpreting the test results. Laboratory shear tests often indicate dilation at large strains; however, in the field the dilation observed in the laboratory may not occur (Olson & Stark, 2002; Yoshimine et al., 1999).

5.1.2 Post-cyclic reconsolidation response

Another key mechanism of earthquake-induced deformations is the overall volume change ($\varepsilon_{v-ps}$) in the soil mass that takes place due to the dissipation of shear-induced excess pore water pressures. Wijewickreme & Sanin (2010) proposed a relationship between post-cyclic volumetric strain and maximum excess pore water pressure ratio ($r_{u-max}$) based on their tests on Fraser River silt, Fraser River sand and Quartz powder. A reasonably consistent trend with respect to $r_{u-max}$ was obtained using data from tests on soil specimens with different overconsolidation history, particle fabric, and initial void ratio. It was noted that the results for Fraser River sand were located lower in the graph compared to the results for other soils. Wijewickreme & Sanin (2010) noted that the value of $\varepsilon_{v-ps}$ gradually increases with increasing $r_{u-max}$. The approach has been adopted as a part of current practice by the geotechnical profession in the Greater Vancouver area of the Province of British Columbia, Canada (GVLTTF 2007).
Limited number of tests was carried out to study the post-cyclic consolidation characteristics of gold tailings herein. Preliminary results are compared with previously observed post-cyclic settlements of low plastic Fraser River silt, Fraser River sand and Quartz powder. The observations on the behavioural patterns of different materials are considered useful. Section 3.3 describes the testing procedure of the post-cyclic reconsolidation.

Typical post-cyclic reconsolidation volumetric strain versus time response of normally consolidated reconstituted gold tailings previously cyclically loaded to different CSR levels are presented on Figure 5.5. The four curves presented on one plot correspond to post-cyclic consolidation tests carried out on soil specimens that have developed different levels of maximum excess pore water pressure ratio ($r_{u\text{-max}}=\Delta u_{\text{max}}/\sigma'_{\text{vc}}$). Based on the information on the plot, it appears that the level of $r_{u\text{-max}}$ has some influence on the post-cyclic volumetric strains (i.e., post-cyclic consolidation settlements).

The results from the present study are superimposed on the $\varepsilon_{\text{v-ps}}$ versus $r_{u\text{-max}}$ relationship proposed by Wijewickreme & Sanin (2010) in Figure 5.5. It can be noted that the results for gold tailings lie within the range of the band suggested by Wijewickreme & Sanin (2010).
Figure 5.5 Typical volumetric strain ($\varepsilon_{v,pc}$) versus time characteristics of normally consolidated reconstituted gold tailings during post-cyclic consolidation.

![Graph showing typical volumetric strain versus time characteristics](image)

CSR = 0.35, $r_{u\text{-max}} = 0.81$
CSR = 0.26, $r_{u\text{-max}} = 0.71$
CSR = 0.10, $r_{u\text{-max}} = 0.95$
CSR = 0.16, $r_{u\text{-max}} = 0.92$

Figure 5.6 Post-cyclic volumetric strain ($\varepsilon_{v,pc}$) versus maximum cyclic excess pore water pressure ratio ($r_{u\text{-max}}$) during cyclic loading for gold tailings (extracted from Sanin (2010)).

![Graph showing post-cyclic volumetric strain versus maximum pore pressure ratio](image)

- Natural FRSilt 100 kPa (Sanin, 2010)
- Natural FRSilt 85 kPa (Sanin, 2010)
- Reconstituted FRSilt Sanin, 2010)
- Quartz rock powder (Sanin, 2010)
- FR sand (Sanin, 2010)
- Tailings from this study:
  - Reconstituted gold tailings

Maximum Pore pressure ratio during cyclic loading, $(\Delta u/\sigma'_{vo})_{\text{max}}$
5.2 Summary and principal findings

The post-cyclic monotonic shear and consolidation response of gold tailings were examined using constant volume cyclic direct simple shear (DSS) testing. All tests were conducted on normally consolidated and overconsolidated specimens prepared by reconstitution. The comparison of maximum post-cyclic undrained shear strength ratio of normally consolidated and overconsolidated gold tailings specimens was carried out to study the effect of overconsolidation ratio. A clear trend of increasing post-cyclic shear strength ratio ($S_{u-PC}/\sigma'_{vc}$) with increase in OCR (or increasing density) was noted. However, as discussed above, the results should be interpreted taking into account the difference between laboratory and field post-cyclic conditions. Additionally, the post-cyclic consolidation volume changes experienced by the gold tailings specimens, subjected to cyclic loading at different cyclic stress ratios (CSRs) and different values of cyclic shear strain based only on limited number of tests, were evaluated. Based on the results, the post-cyclic reconsolidation volumetric strains increase with the maximum pore water pressure generated during cyclic loading. The results are then superimposed with previously published by Wijewickreme & Sanin (2010) results for Fraser River silt, Fraser River sand and Quartz rock powder. The results derived for the gold tailings studied herein are in agreement with previously published results on post-cyclic volumetric strains.
Chapter 6: Summary and conclusions

This thesis presents an experimental investigation on the constant-volume shear response of low-plastic fine-grained gold tailings. The monotonic and cyclic shear as well as post-cyclic response of the tailings was assessed using the direct simple shear (DSS) device with the main objective of developing an experimental database for calibration of analytical models and generation of the input parameters for the modeling of the response of paste tailings deposits. In particular, the research was focused on understanding the effects of the following factors on the monotonic and cyclic loading response of the gold tailings: initial vertical effective stress, effect of initial static shear bias, and mechanical overconsolidation. Additionally, a comparison was made between the monotonic response of tailings specimens: (i) derived from tube samples extracted from a desiccated laboratory soil deposit; and (ii) prepared by direct laboratory reconstitution of disturbed bulk soil samples. Post-cyclic behaviour of the gold tailings material, considering post-cyclic volumetric strains and post-cyclic shear strength, was also studied. The key contributions, conclusions and recommendations originating from this research are summarized below.

6.1 Monotonic shear response of fine-grained gold tailings

Under monotonic constant - volume DSS loading, reconstituted normally consolidated gold tailings specimens initially exhibited contractive behaviour followed by a dilative response. Tests conducted on specimens initially consolidated to vertical effective stress levels ranging from 50 kPa to 400 kPa exhibited identical mobilized shear stress ratios at the instance of phase transformation. The monotonic shear stiffness and strength of the gold tailings specimens increased with increasing initial confining stress level. The stress paths for these tests, when normalized to the initial effective confining stress $\sigma'_{vc}$, followed a consistent pattern and fell
within a narrow range. This stress history normalizability is similar to the typical response of normally consolidated clays and low-plastic silts.

Relatively undisturbed and reconstituted specimens showed significantly different shearing behaviour during normally consolidated monotonic shear tests. Reconstituted specimens exhibited more contractive response compared to the undisturbed specimens tested at the same stress level although the starting void ratios for the reconstituted specimens were higher than the void ratios for relatively undisturbed specimens. The undisturbed specimens had noticeably stiffer and stronger response than reconstituted specimens. The difference between the responses might be attributed to the partial desiccation during sample preparation at Carleton University, lower void ratios in the relatively undisturbed specimens and/or different specimen preparation methods (fabric).

Monotonic shearing of overconsolidated reconstituted gold tailings specimens, as expected, indicated increasing shear resistance and stiffness with increasing OCR. Overconsolidated reconstituted specimens showed strain–hardening followed by the strain-softening after reaching about 10% shear strain. Overconsolidated specimens developed negative excess pore pressures during shearing, suggesting increased dilative response with increasing OCR. The behaviour is similar to the typical monotonic behaviour of overconsolidated clays.

6.2 Cyclic loading response of normally and overconsolidated reconstituted gold tailings

Strain-softening, accompanied by loss of shear strength, was not observed regardless of the applied cyclic stress ratio or the decrease in effective stress experienced (i.e., the level of
equivalent excess pore-water pressure generated) during cyclic loading. This behaviour suggests that gold tailings is unlikely to experience flow failure as typically observed in loose saturated sand deposits subjected to cyclic loading.

Under constant-volume cyclic DSS loading, gold tailings exhibited a cumulative decrease in effective stress (or increase in equivalent excess pore-water pressure) with increasing number of loading cycles, which suggested progressive degradation of shear stiffness. Specimens tested under more severe cyclic stress ratio levels reached $\gamma = 3.75\%$ in a fewer number of cycles than those subjected to low CSR values.

The cyclic tests with initial static shear stress bias conducted on gold tailings, simulating sloping ground condition, indicate decrease in cyclic resistance ratio (CRR) (i.e. CSR to reach number of cycles to reach $\gamma = 3.75\%$) with increasing level of initial static shear stress bias. The tests on overconsolidated gold tailings specimens indicated that the CRR and shear stiffness would increase with increasing OCR.

The results from this work on normally consolidated reconstituted specimens were also compared to the available published data on the cyclic response of different tailings, obtained from tests carried out on cyclic triaxial (TX) and DSS devices. The comparison indicates that the findings from this research work are in good agreement with those from past works. The CRR of the gold tailings material used in this study was found to be higher than that observed in Fraser River sand and Quartz rock powder, but in the same range as Fraser River silt.
The difference between the CRR observed for different materials may be attributable to the differences in the mineralogy, chemical composition, and particle shapes of the material types (in addition to void ratio and confining stress effect). For example, chemical composition of tailings would likely have been impacted by the processing of mine ore. There is opportunity for higher angularity in particle shapes in tailings material (that arises from the mechanical crushing and processing of mine ore) compared to Fraser River sand that is naturally deposited. Further investigations to study the chemical composition, mineralogy, and microstructure will be required to confirm the merits of these considerations.

6.3 Post–cyclic response of gold tailings

The post-cyclic monotonic shearing response, obtained from DSS tests, conducted on normally consolidated and overconsolidated reconstituted gold tailings specimens, was studied. The post-cyclic shear strengths of normally consolidated and overconsolidated specimens were normalized to the initial effective confining stress \( \sigma'_{vc} \) to obtain post-cyclic strengths ratio \( (S_{u-PC}/\sigma'_{vc}) \). The post-cyclic shear strength ratio was noted to increase with increasing OCR (or increasing density). However, the values of post-cyclic shear strengths derived from constant volume laboratory tests should be interpreted with caution since these results may be affected by the drainage conditions that prevail under field conditions.

The post-cyclic consolidation volume changes experienced by the gold tailings specimens, subjected to cyclic loading at different cyclic stress ratios (CSRs) and different values of cyclic shear strain based only on limited number of tests, were evaluated. Based on the results, the post-
cyclic reconsolidation volumetric strains increase with the maximum pore water pressure generated during cyclic loading. The results were superimposed with previously published by Wijewickreme & Sanin (2010) results for Fraser River silt, Fraser River sand and Quartz rock powder. The results derived for the gold tailings studied herein are in agreement with previously published results on post-cyclic volumetric strains.

6.4 Recommendations for future research

In addition to the findings presented above, the work also led to identification of additional research tasks for future research to advance the current understanding of the mechanical response of tailings:

- The findings presented in this thesis were based on tests carried out on low-plastic fine-grained gold tailings obtained from Bulyanhulu gold mine, located in Northern Tanzania. Increasing and improving the database by carrying out similar work on tailings from different mines is recommended. As mentioned earlier, the mechanical behaviour of tailings might be significantly influenced by type of mine, depositional method, particle size, porosity, particle shape, etc.

- There is a need to systematically compare the results from mechanically overconsolidated gold tailings specimens from the current program with those from “undisturbed” samples obtained from the laboratory gold tailings deposit prepared at Carleton University (as noted earlier, this work could not be undertaken as a part of the present scope of work due to the unavailability of “undisturbed” samples obtained from the laboratory gold tailings deposit prepared at Carleton University before completion of the present research work).
• The effect of particle fabric on the mechanical behaviour of tailings is an important consideration. As such, detailed studies should be undertaken to study the effect of fabric, particle orientation, particle contacts, porosity, chemical composition, etc., on the mechanical behaviour. This will require the use of state-of-the-art microscopic observational techniques (e.g., Scanning Electron Microscopy, X-ray Computer Tomography), with high-resolution 2D and 3D imaging.

• Additional correlations of the laboratory observations can be done with data from field testing (e.g., cone penetration testing), so that the response of a given gold tailings under monotonic and cyclic loading can be confidently predicted using data from field testing.

• The data from the current work was obtained from tests conducted using the direct simple shear (DSS) device, in which “$K_0$” conditions are assumed. Measuring the lateral stresses on DSS specimens during testing should improve the understanding of the response of tailings to monotonic and cyclic loading.
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