VOLUME CHANGE AND PERMEABILITY OF MIXTURES OF WASTE ROCK AND FINE TAILINGS

by

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ABSTRACT

Current methods of mine waste disposal have resulted in environmental problems on a global scale. In an effort to circumvent problems inherent to segregated disposal of waste rock and tailings, this thesis examines an alternative mine waste disposal technique where waste rock and tailings are combined as a homogenous mixture. One type of waste rock and one type of tailings were examined individually and as mixtures for volume change, permeability, and soil-water characteristics. Laboratory investigations included compressibility testing in a large diameter consolidation cell, hydraulic conductivity by constant head test alternated with static loading, and soil-water characteristic curve by pressure plate test. Tailings rheology was examined in a cup and bob rotational viscometer. A two-year meso-scale column study of self-weight consolidation was also conducted.

The behaviour of mixtures of waste rock and tailings is dependent on mixture design, and mixtures may therefore be designed for specific geotechnical properties. Design variables examined included mixture ratio and initial tailings solids content. Mixture ratio was found to have an optimum value where the tailings slurry just fills the voids between rock particles. Tailings solids content was found to have limits for mixing defined by rheological yield stress. Mixtures had compressive strains similar to waste rock alone and much lower than tailings alone. The finding was attributed to the presence of a load bearing waste rock skeleton. The hydraulic conductivity of mixtures was similar to tailings and much lower than waste rock. The finding was attributed to the presence of a fine-grained tailings matrix. While waste rock alone was observed to
remain unsaturated, mixtures remained saturated for long periods of time under conditions of free drainage without access to water.

Mixture particle structure was conceptualized through the use of a particle model and described quantitatively by waste rock skeleton void ratio and tailings matrix void ratio. Methods for relating mixture compressibility and hydraulic conductivity to structure are proposed. While exhaustive testing of the range of possible mixtures and behaviours was not possible, this thesis has provided a fundamental theoretical basis for understanding mixture design and behaviour with respect to mixture particle structure.
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<tr>
<td>%fines CIP</td>
<td>percentage CIP tailings passing the 0.425 mm sieve by mass</td>
</tr>
<tr>
<td>%fines mixture</td>
<td>percentage of mixture passing the 0.425 mm sieve by mass</td>
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<tr>
<td>%fines rock</td>
<td>percentage of waste rock passing the 0.425 mm sieve by mass</td>
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<tr>
<td>A</td>
<td>area of specimen</td>
</tr>
<tr>
<td>a</td>
<td>area of standpipe</td>
</tr>
<tr>
<td>A</td>
<td>fitting constant for hydraulic conductivity regression</td>
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<tr>
<td>AEV</td>
<td>Air Entry Value</td>
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<td>c&lt;sub&gt;v&lt;/sub&gt;</td>
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<td>HALT</td>
<td>Hydro-Active Limestone Treatment System</td>
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<td>high density polyethylene</td>
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<td>k</td>
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<td>m&lt;sub&gt;v&lt;/sub&gt;</td>
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<tr>
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<tr>
<td>n&lt;sub&gt;b&lt;/sub&gt;</td>
<td>porosity of crushed rock</td>
</tr>
<tr>
<td>n&lt;sub&gt;large&lt;/sub&gt;</td>
<td>porosity of large component</td>
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n_r  porosity of waste rock skeleton
n_{r0} porosity of initial waste rock skeleton
n_s  porosity of sand
n_{small} porosity of small component
n_t  porosity of tailings matrix
P    Pulp Density or mass solids divided by mass total for tailings slurry
p', q triaxial stress parameters
r    radius
R    mixture ratio, waste rock to tailings by dry mass
R^2  regression coefficient (Coefficient of Determination)
SWCC soil-water characteristic curve
T_s  surface tension
t    time
u    pore-water pressure
V    volume total
v    Poission's Ratio
V_a  volume air
v_c  clay specific volume
V_c  volume of clay particles
v_g  granular specific volume
V_g  volume of granular particles
V_r  volume of waste rock solids
V_s  volume solids
V_t  volume of tailings solids
V_v  volume voids
V_w  volume water
w    gravimetric water content, mass water divided by mass solids
\sigma_v' vertical effective stress
\sigma_{v_r} vertical effective stress carried by waste rock portion of mixture
\sigma_{v_t} vertical effective stress carried by tailings portion of mixture
\alpha_r waste rock participation factor
\eta_C Casson apparent viscosity
\rho_r  density of waste rock solids
\rho_t  density of the tailings solids
\tau_C Casson yield stress,
T    tortuosity
T_s tortuosity of sand
\tau  shear stress
y    weight % of large constituent in a mixture
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CHAPTER ONE. BACKGROUND

Chapter One reviews the problems associated with conventional mine waste disposal practices, introduces the concept of mixed disposal, and presents the research objectives of this thesis.

1.1 Introduction

The current thesis investigates an alternative mine waste disposal technique with the potential to address problems associated with standard practices for mine waste disposal. Chapter One presents an overview of the major problems associated with standard mine waste disposal methods, as well as common strategies to deal with them.

The use of mineral resources is necessary to the operation of human society. As a consequence of the distribution of mineral resources within the earth's crust, the extraction of mineral resources produces large quantities of waste materials. The majority of material that is moved and processed during mining operations is re-deposited at the mine site as waste. The two major waste streams include waste rock and tailings, which are discussed below. The strip ratio of waste rock to ore for surface metal mining operations may be higher than ten to one. In addition, the amount of metal recoverable from a high-grade ore may be in the order of grams per tonne. The product minerals that are taken off-site for use represent a minute quantity relative to the mass and volume of waste rock and tailings produced by the mining process. Experience has shown that conventional mine waste disposal techniques result in predictable problems.
Waste rock is most commonly disposed by end dumping out of haul trucks in large piles. Waste rock piles are highly porous and typically promote weathering and the formation of acid rock drainage. Tailings are often disposed by pumping them in slurry form to impoundments. Slurried tailings in impoundments remain loose and unconsolidated, and are prone to catastrophic liquefaction failures. The problems of acid rock drainage and liquefaction failure are inherent to the physical structure of waste rock piles and tailings in impoundments, respectively.

1.2 Waste Rock

Waste rock is rock that is not milled, but is simply moved to get at an ore body. Waste rock at surface mines is typically blasted, loaded into trucks, and then dumped in large piles. Rock particle sizes greater than one meter in diameter are common. Angular waste rock particles disposed by end dumping have open void spaces that allow rapid infiltration of water and free flow of air. The idea that waste rock can be dumped and safely ignored has resulted in the wide-spread environmental problem of acid rock drainage (ARD).

1.2.1 Acid Rock Drainage

Waste rock can weather rapidly upon exposure to water and oxygen. If waste rock contains sulphide minerals it may react with water and oxygen in the presence of naturally occurring bacteria to produce an effluent called “acid rock drainage” (ARD). There have been many studies of the chemistry of ARD (e.g. Shum and Lavkulich 1999, Puura et al. 1999, Williams and Smith 2000, Schippers et al. 2000). Sample chemical
reactions for the oxidation of iron pyrite (FeS₂), a common metal sulphide, are shown in Equations [1.1] – [1.4] (after Singer and Strum 1970).

[1.1] FeS₂(S) + 7/2O₂(aq) + H₂O = Fe²⁺, Fe³⁺ + 2SO₄²⁻ + 2H⁺
[1.2] Fe²⁺ + 1/4O₂(aq) + H⁺ = Fe³⁺ + 1/2 H₂O
[1.3] Fe³⁺ + 3H₂O = Fe(OH)₃(s) + 3H⁺
[1.4] FeS₂(s) + 14 Fe³⁺ + 8H₂O = 15 Fe²⁺ + 2SO₄²⁻ + 16 H⁺

The production of H⁺ lowers the pH of drainage effluents and allows metals that were initially part of the rock as metal sulphides to dissolve into solution. The rate of reaction is dependent on pH, but more importantly on the presence and action of bacteria such as *Thiobacillus ferrooxidans*, which can accelerate the rate-determining step of Fe²⁺ oxidation by a factor of 10⁶ (Singer and Strum 1970). ARD becomes an environmental problem when natural forms of neutralization such as carbonate minerals are not available in sufficient quantities to neutralize the drainage. The metal rich, low pH effluent is highly toxic to life downstream of the waste rock pile, e.g. Barry et al. (2000), Soucek et al. (2001).

Currently, there is no proven technology that can stop the ARD reaction in an existing waste rock pile. Methods of dealing with ARD include application of neutralizing materials, bactericides, the use of oxygen and/or water barrier soil covers, relocation, isolation and segregation, and also the treatment of effluents by neutralization or
attenuation. A sample of methods investigated for preventing or reducing ARD are summarized in Table 1.1. Sample case histories of ARD problems include Porterfield et al. (2003), Aykol et al. (2003), and Waters and O’Kane (2003).
Table 1.1 Sample strategies for management of ARD.

<table>
<thead>
<tr>
<th>Method</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cover Systems</strong></td>
<td></td>
</tr>
<tr>
<td>Soil / water / wood bark</td>
<td>Yanful et al. 2000 and Yanful and Orlandea 2000</td>
</tr>
<tr>
<td>soil as oxygen barrier</td>
<td>Lundgren 2001</td>
</tr>
<tr>
<td>soil as oxygen and water barrier</td>
<td>O’kane et al. 1998</td>
</tr>
<tr>
<td>clay slurry</td>
<td>Shang 1997</td>
</tr>
<tr>
<td>wood waste</td>
<td>Germain et al. 2000, 2003</td>
</tr>
<tr>
<td>soil as store and release system</td>
<td>Williams et al. 1997</td>
</tr>
<tr>
<td>de-inking residues</td>
<td>Cabral et al. 1997</td>
</tr>
<tr>
<td>pulp and paper</td>
<td>Cabral 2000</td>
</tr>
<tr>
<td>run of mine material</td>
<td>O’kane et al. 2000</td>
</tr>
<tr>
<td>clay-sand composite</td>
<td>Woyshner and Yanful 1995</td>
</tr>
<tr>
<td>soil with capillary break</td>
<td>Nicholson et al. 1989</td>
</tr>
<tr>
<td>multi layer till/sand/gravel</td>
<td>Simms and Yanful 1999</td>
</tr>
<tr>
<td>desulphurized tailings</td>
<td>Sjoberg Dobchuk et al. 2003</td>
</tr>
<tr>
<td><strong>Relocation</strong></td>
<td></td>
</tr>
<tr>
<td>submergence</td>
<td>Morin and Hutt 2001</td>
</tr>
<tr>
<td><strong>Bactericides</strong></td>
<td></td>
</tr>
<tr>
<td>sodium dodecyl sulphate</td>
<td>Schippers et al. 1998</td>
</tr>
<tr>
<td>thiocyanate</td>
<td>Olson et al. 2003</td>
</tr>
<tr>
<td><strong>Addition of Acid Buffering Materials</strong></td>
<td></td>
</tr>
<tr>
<td>calcium bentonite, concrete</td>
<td>Schippers et al. (1998)</td>
</tr>
<tr>
<td>limestone</td>
<td>Yanful et al. (2000) and Yanful and Orlandea (2000)</td>
</tr>
<tr>
<td>phosphate rock</td>
<td></td>
</tr>
<tr>
<td>acid neutralizing rock</td>
<td>Morin and Hutt 2000</td>
</tr>
<tr>
<td>lime kiln fines</td>
<td>Lapakko et al. 2000</td>
</tr>
</tbody>
</table>
The addition of acid buffering materials is generally effective until consumed (eg. Yanful et al. 2000). The amount of potentially acid forming rock in a waste rock deposit is routinely millions of tonnes it is unlikely that an equivalent quantity of neutralizing material will be economically available. Studies reviewing performance of soil cover systems show mixed results. Woyshner et al. (1997) predicted an eventual decline in ARD production from wastes covered with compacted clay. Timms and Bennett (2000) concluded that a compacted clay cover over wastes at Rum Jungle temporarily reduced oxidation by one third to one half, but that rates of oxidation were increasing after 15 years. Wilson et al. (2003b) reviewed several case histories and concluded that the performance of barrier and multi-layer soil covers does not warrant the high cost of construction.

Two waste cover methods stand out as having the potential to reduce ARD. Williams et al. (2003a) concluded that a “store and release” cover was highly effective in a semi-arid climate over a seven year period, although the significance of long term erosion of the cover remains an issue. Yanful et al. (2000) concluded that water covers were effective for reducing ARD compared to soil and woodbark covers. Water saturated materials can significantly reduce the formation of ARD by limiting the flow of oxygen to advective flow by percolation and to diffusive flow through saturated media (eg. Schuring et al. 1997). While soil covers may act as effective barriers to oxygen (Lundgren 2000) or water in the short term, the long term performance and integrity of soil cover systems is questionable due to effects of climate, vegetation, burrowing animals, erosion, etc.
Soil covers may act to both prevent and mitigate ARD. What is more common than cover systems is the use of treatment systems for on-going ARD problems. Waters et al. (2003) reviewed existing methods of ARD treatment including passive treatment systems such as oxic and anoxic limestone drains, limestone diversion wells, Pyrolusite® limestone beds, aerobic and anaerobic wetlands, reducing and alkalinity producing systems, permeable reactive barriers, slag leach beds, and gas redox and displacement systems. Active approaches discussed include pH control/precipitation, adsorption and absorption, ion-exchange, electrochemical concentration, biological mediation/redox control, flocculation and filtration, and crystallisation. Waters et al. (2003) also reviewed the price and efficiency of reagents used for water quality control, and also application systems such as conventional dry powder liquid mixing and dosing systems, pulsed carbonate reactors, Neutra-Mill Technology, Hydro-Active Limestone Treatment System (HALT), and Aquafix. Passive treatment technologies were viewed as a post-closure technology with limited capacity and service life that requires on-going maintenance. Active treatments required continuous or semi-continuous application to correct water quality and may be engineered to deal with almost any effluent type and loading. Waters et al. (2003) concluded that the most widely used and low cost method for treatment of ARD effluents is pH adjustment with cost effective neutralization chemicals such as limestone.

The ARD reaction depends, as with any chemical reaction, on the availability of reactants. The availability of sulphide minerals for reaction depends on waste rock mineralogy, particle size and state of oxidation. The distinction between ore and waste
rock is an economic one and waste rock can contain significant quantities of metal sulphides. Sulphur reducing bacteria are widely distributed, even underground (e.g. Schrenk et al. 1998). Water and oxygen are available from the atmosphere, but must move into and through the waste rock pile to react with sulphide minerals. Water acts as both a reactant and as a transport mechanism for ARD. Consequently, waste rock hydrology is critical for predicting the rate and quantity of ARD (Swanson et al. 2000). Waste rock hydrology is complicated by unsaturated conditions in the waste rock pile, where hydraulic conductivity varies with particle size distribution, particle packing arrangements, and matric suctions, or negative pore-water pressures. The production of ARD is dependent on climate, but also on the effect of waste rock pile structure relative to the transport of water and oxygen.

1.2.2 Structure

Smaller scale studies attempting to quantify the flow of water through waste rock have demonstrated the complexity of unsaturated flow in randomly placed waste rock. Stockwell et al. (2003) were unable to measure a field based soil-water characteristic curve for randomly placed waste rock due to spatial variability and hysteresis of waste rock properties in a 12 m pile. Tracer studies by Eriksson et al. (1997) in laboratory columns (0.8 m diameter, 2 m height) did not indicate significant preferential flow in waste rock. Li (2000) concluded that infiltration is heterogeneous for unsaturated flow, and is homogenous or ubiquitous for flow channelling for waste rock in columns (1.5 m diameter, 2 m height). Studies of waste rock placed randomly in an intermediate scale pile (8 m square, 5 m high) indicated flow occurring within the fine-grained matrix, within larger pores and within matrix-free areas (Nichol et al. 2000, Nichol et al. 2003).
Nichol et al. (2003) observed that application of water at a constant rate to the surface of a pile of randomly placed waste rock resulted in a complex discharge hydrograph that was not easily correlated to the rate of application. Tracer testing with the same pile indicated several spatially distinct flow pathways with varying wetting front velocities, average water velocities and dispersive properties (Nichol et al 2003). Marcoline et al. (2003) concluded that internal structure, rather than surface treatments, appears to dominate flow processes (for an 8 m square by 5 m high pile). The structure of randomly placed waste rock is complex and influences the storage and release of both water and oxidation products.

Larger-scale studies of waste rock piles indicate that macro-structure within the waste rock piles influences waste rock hydrology. Herasymuik (1996) characterized a waste rock pile constructed by end dumping in a semi-arid climate. The internal structure of the pile consisted of dipping, interbedded coarse and fine layers with a zone of coarse material at the base of the pile. Herasymuik (1996) proposed a conceptual model (herein called the Herasymuik model) that accounts for the flow and storage of water within waste rock piles constructed by end dumping. According to the Herasymuik model, the flow of liquid water occurs in layers of finer material under negative pore-water pressures, and the flow of water vapour (gas flow) occurs in layers of coarser material with open pore spaces. The waste rock pile was formed by end dumping rock in a dry condition. The pile remained dry except for the upper 15 m of the pile, which were wetted by infiltration. Unlike past observations of waste rock piles, Herasymuik’s investigation was comprehensive, and included sampling and logging of test pits on
transects through the pile. Each layer was logged with respect to mineralogic components, general grain size range, texture, structure, condition of interparticle spaces (open or infilled with fine matrix), state of oxidation and weathering, color, strike, and dip. In situ measurements included water content, matric suction, temperature, and relative humidity. Select samples were tested in the laboratory for water content, particle size distribution, soil-water characteristic curves, paste pH, specific gravity, and also saturated hydraulic conductivity (Herasymuik 1996).

Research supporting the Herasymuik model includes work by Newman (1999), who demonstrated that preferential water flow occurs primarily in finer materials for vertically oriented layers of coarse and fine waste rock above the water table. Newman (1999) also found that layers with finer particle size distributions in the top of a waste rock pile were relatively wet and oxidized, while layers with coarser particle size distributions were found to be dry and unoxidized. Stromberg and Banwart (1999) and Hollings et al. (2001) also found that smaller waste rock particles weather faster than larger ones. Fines et al. (2003b) investigated two waste rock piles in different climates and observed the same internal dipping structures as observed by Herasymuik (1996). Wilson et al. (2000), and more recently Fala et al. (2003) have numerically simulated water flow in idealized waste rock pile structures, including the Herasymuik model.

The transport of oxygen in waste rock piles occurs by mechanisms of diffusion, advection due to thermal gradients, and advection due to barometric pumping (Wels et al. 2003).
The ARD reaction is exothermic. The heat generated by reaction can significantly raise the internal temperature of a waste rock pile to create thermal gradients that drive air flow, e.g. Waters and O'Kane (2003) reported combustion temperatures greater than 220 °C. Lefebvre et al. (2001a) described coupled physical processes for pyrite oxidation in a waste rock pile, including oxygen consumption, heat production, gas transfer by advection and diffusion, water infiltration, and dissolved mass transfer in unsaturated heterogeneous coarse porous media. Lefebvre et al. (2001b) further described results of numerical simulation of the processes occurring in two types of waste rock piles. The flow of air for waste rock piles with high internal temperatures is analogous to a campfire, with fresh air containing oxygen drawn in through porous zones at the base of the pile.

While waste rock pile structure allows transport of water and oxygen to sulphide minerals in the pile, the length of time of ARD production is ultimately limited by the mass of sulphide minerals present. Waste rock piles are some of the largest structures built by man, i.e. billions of tonnes of material. The quantities of sulphide minerals are proportionately large. Assuming that water and oxygen will be available in the future from the atmosphere, the time required to exhaust available sulphide minerals in a waste rock pile may run into the thousands of years.

1.2.3 Overview

The problem of ARD likely started when man began to use metal and the problem is likely to continue for some time. Compared to rock that is unexposed and underground, the porous nature of end-dumped waste rock facilitates transport of water and oxygen to
metal sulphide minerals. Consequently, the structure of waste rock piles promotes weathering and the production of ARD. The common use of end dumping as a disposal technique for waste rock, and the sheer size of waste rock piles, make ARD one of the largest global environmental issues facing the mining industry in 2006. Government and industry have responded to the issue with initiatives such as the Canadian Mine Environment Neutral Drainage (MEND) program, which started in 1989 (Tremblay 2000), and the International Network for Acid Prevention (INAP) which formed in 1998 (Fleury and Kemp 2003), respectively. While no global estimate of the impact of ARD has been undertaken (Fleury and Kemp 2003), INAP estimated total liability costs for potentially acid generating wastes at mining sites in 2003 to be US$530 million in Australia, US$1.2-20.6 billion in the United States, and US$1.3-3.3 billion in Canada (INAP 2003). Effective methods for preventing ARD include submergence of acid producing wastes under water, and store and release covers for semi-arid climates. The current (2006) preferred method of treatment of existing ARD problems is to collect and treat acidic effluents during mine operation, after mine closure, and in perpetuity.

1.3 Tailings
The second waste stream produced by the mining industry consists of tailings. Tailings are what remain of the ore after the product mineral(s) have been removed. Ore is ground to achieve a fine particle size distribution that facilitates the extraction process, e.g. flotation. After production minerals are extracted, the ‘tailings’ consist primarily of ground rock particles, water, and reagents from the extraction process. Tailings particle size distributions typically range from clay to sand sizes, depending on the extraction method. The chemistry of the tailings depends on the ore mineralogy, extraction process,
and post-milling treatments such as neutralization. Tailings are typically slurried and pumped to impoundments for disposal. Sample case histories for disposal of different types of tailings include uranium tailings (Balych and Sinclair 1997), taconite tailings (Klohn 1979), gold tailings (Blight and Steffen 1979), and phosphatic mine wastes (Bromwell and Raden 1979).

Kealy and Busch (1979) stated that “In the past waste volumes were small, land was plentiful, and the environment was not viewed as critical to our society. Cost has therefore been the governing factor, and the practice has been to ‘get rid of them as quickly and cheaply as possible.’” Kealy and Busch (1979) also stated that the disposal of tailings has historically been the responsibility of the metallurgist who had little or no geotechnical training. Conventional tailings impoundment walls, or dams, are constructed from the coarse fraction of tailings where possible (UNEP/ICOLD 2001). Slurried tailings pumped to impoundments are separated into coarse and fine particles, typically by cyclone. The larger sand sized particles from cyclone underflow are used for construction of the impoundment. The remaining clay and silt-sized particles are discharged into the impoundment. The impoundment wall is raised in lifts of the coarse underflow from the cyclone. The method requires a small starter dam of non-tailings material, but the rest of the dam is built with tailings. The costs involved are a fraction of that required for a dam constructed from earth or concrete.

The attention and effort used in the design and construction of dams retaining tailings versus dams retaining water are different (UNEP/ICOLD 2001). Water dams are revenue
generating, while tailings dams cost money but do not produce revenue. Consequently, more effort has historically been spent on the design of water dams, and less on tailings dams (Vick 1983).

1.3.1 Liquefaction Failure
There have been over 70 major catastrophic failures of tailings dams around the world since 1950 (UNEP / ICOLD 2001). Tailings have been known to liquefy upon dam failure and flow for kilometres, covering areas measured in hectares. The failures are destructive, and have resulted in loss of life and environmental damages. The average is one or two failures a year. An updated list of tailings dam failures may be found at http://www.antenna.nl/wise-database/uranium/mdaf.html (UNEP / ICOLD 2001). While certain impoundment construction and design methods, like upstream construction with low factors of safety, are a recipe for disaster (Kealy and Busch 1979), not all tailings dam failures are due to the upstream method. The common element in catastrophic failures is that unconsolidated tailings retain the ability to liquefy and flow for years after deposition.

1.3.2 Structure
Tailings are typically slurried and pumped to impoundments for disposal. The pumping characteristics, or rheology, of tailings slurry is dependent on solids content, mineralogy, temperature, chemistry of solution, particle size distribution, particle shape, and in some cases, time. The rheology of tailings slurry is often managed by addition or removal of water. Once in the impoundment the tailings settle and consolidate and water used for transport can be reclaimed. Schiffman et al. (1988) gave an excellent description of the process of tailings settlement and consolidation. Recently settled tailings solids are
loosely packed, with particles that are just touching. The low specific gravity of the tailings slurry limits the mass available to drive self-weight consolidation. The combination of low pressures and fine particle sizes produce an unconsolidated mass with little shear strength. Tailings deposited in impoundments can remain semi-fluid and unconsolidated for tens and even hundreds of years after mine closure.

The unconsolidated, semi-fluid nature of tailings in impoundments makes them difficult to reclaim. Williams (1990) noted that old coal tailings deposits consist of a thin crust overlying tailings that are essentially in slurry form. Tailings that remain soft may be untrafficable (Williams 1996a). In many cases a water cover must be maintained over tailings to ensure the tailings do not “turn acid”. What is more important is that tailings in impoundments remain in an unconsolidated state that makes them prone to liquefaction. Tailings dams with semi-fluid tailings and/or water behind them must be managed against erosion and failure, often after mine closure and in perpetuity.

1.3.3 Summary

The method of pumping slurried tailings to impoundments for disposal has historically been preferred by the mining industry because of its low initial cost. Tailings pumped to impoundments remain loose and unconsolidated, with little shear strength. As a result, tailings in impoundments have the potential to liquefy and flow upon failure. There have been many catastrophic failures of tailings dams resulting in loss of life, environmental damages, and hugely expensive reparations. The potential for catastrophic liquefaction failure make tailings impoundments a long-term liability that must be managed against erosion and failure during mine life, after closure, and in perpetuity.
1.4 Mixtures of Waste Rock and Tailings

The recurrent problems and long term liability created by traditional disposal methods suggest that a fundamental re-thinking of mine waste management is necessary. Scoble et al. (2000) have called for complete re-engineering of the mining industry in order to focus on waste management and to reduce the impact of mine wastes. Ideally, a waste management technique should provide a walk-away solution at a low cost. Mixing waste rock and tailings for disposal is a relatively new concept in mine waste management with the potential to avoid the problems associated with current methods.

Potential advantages of mixing waste rock and tailings compared to conventional methods of disposal include:

i) reduction of ARD and metal leaching,

ii) reduced potential for liquefaction failure,

iii) no tailings dam required, and

iv) less total volume required for disposal.

Filling the void space of the waste rock with tailings slows the flow of water and oxygen and thereby limits the ARD reaction. With the tailings held in the voids of the waste rock, there is no need for a tailings dam. Elimination of a tailings dam eliminates the capital costs required for building the dam, eliminates the potential for catastrophic liquefaction failure, eliminates the requirement of perpetual management of drainage, and eliminates problems associated with reclamation of soft tailings. By using the near 30% void space in waste rock (by volume), the land required for waste disposal is reduced, and reclamation costs are also reduced.
The concept of mixing of waste rock and tailings for disposal has many potential advantages over conventional methods. Indeed, if any of the advantages listed above can incur economic and/or environmental benefits over conventional methods, then the idea is worth trying. Past investigations, described in Chapter Two, indicate that mixtures of waste rock and tailings can have geotechnical properties that are desirable relative to segregated waste rock and tailings. However, the idea of disposal of homogeneous mixtures of waste rock and tailings is untried. The cost and sheer volume of material involved demand thorough investigation of any new mine waste disposal technique prior to implementation. Investigations are required to satisfy economic, operational, and geotechnical concerns.

Classical soil mechanics describes geotechnical behaviours of shear strength, volume change, and permeability. In addition, liquefaction potential and water retention characteristics are also important for mine wastes. It should be noted that although conventional disposal methods for sulphide bearing materials are prone to the issues of ARD, and slurried tailings are prone to liquefaction failure, mixtures of waste rock and tailings might have other issues. Consequently, the comprehensive investigation of geotechnical properties and behaviour of mixtures is required prior to implementation.
1.5 Scope and Objectives of Research

The primary goal of the present thesis is to provide a fundamental theoretical basis for evaluating mixtures of mine wastes relative to conventional disposal methods. Specific objectives are:

i) to provide a state-of-the-art review of literature on co-disposal research, with an emphasis on existing theory for mixtures,

ii) to review studies of geotechnical behaviour of soil mixtures for existing relations between mixture structure and behaviour,

iii) to develop a conceptual model for understanding the behaviour of mixtures of waste rock and tailings with respect to particle structure,

iv) to develop a method for designing mixtures of waste rock and tailings with an emphasis on understanding the effect of mixture design variables on mixture structure and behaviour,

v) to observe and report geotechnical properties and behaviours of mixtures of waste rock and tailings including volume change, hydraulic conductivity, and soil-water characteristics,

vi) to evaluate the developed theory, design method and conceptual model with respect to observed phenomena, and

vii) to compare the geotechnical properties and behaviours of mixtures with waste rock and tailings disposed by conventional methods.

The scope of research includes investigation of hydraulic conductivity, volume change, and soil-water retention for one type of tailings, one type of waste rock, and mixtures thereof. Literature review of mine waste disposal, co-disposal research, mixture theory,
and geotechnical investigations of soil mixtures are also included. Mixture design is examined and includes an investigation of tailings rheology. Due to practical limitations, the scope of this study does not include investigation of shear strength, economic or cost benefit analysis, operational concerns, geochemistry, implementation methodology, potential for piping, seismic or static liquefaction, or freeze-thaw effects, although such investigations are acknowledged to be fundamental to evaluation and implementation.

1.6 Layout of the Thesis

The thesis is organized into three sections of three chapters each. The first section includes literature review and theory. Chapter One defines problems with current methods of mine waste disposal. Chapter Two reviews past work on co-disposal of mine wastes. Chapter Three includes a review of particle packing theory for mixtures of waste rock and tailings, a review of studies of soil mixtures, and introduces a conceptual model for mixture particle structure. The second section presents practical investigations and results. Chapter Four describes the experimental methodology used to evaluate mixture behaviour. Chapters Five and Six present results from the laboratory and column studies. The third section of the thesis presents analyses, discussion, and conclusions. Chapter Seven includes a comparison of laboratory and column study findings. Chapter Eight includes a discussion of the results. Chapter 9 presents conclusions of the thesis.
CHAPTER TWO. LITERATURE REVIEW

Chapter Two reviews available literature on the co-disposal of waste rock and tailings.

2.1 Introduction

A review of the literature indicates that little research has been done on mixtures of waste rock and tailings. Consequently, the following review focuses on related types of co-disposal, which, as listed in Table 2.1, are differentiated by the degree of mixing and method of placement.

Table 2.1 Methods of co-disposal.

| Homogeneous mixtures - waste rock and tailings are blended to form a homogeneous mass (method unknown) | Increasing degree of mixing |
| pumped co-disposal - coarse and fine materials are pumped to impoundments for disposal (segregation occurs) |
| layered co-mingling - alternating layers of waste rock and tailings |
| waste rock is added to a tailings impoundment |
| tailings are added to a waste rock dump |
| waste rock and tailings are disposed in the same topographic depression |
| separate disposal - waste rock in dumps, tailings in impoundments |

The majority of work done has investigated pumped co-disposal. A limited amount of work has been done to investigate co-disposal of metal mining wastes and includes layered co-mingling and the use of co-disposed mine wastes as materials for soil covers. Cement stabilized underground backfills are also considered briefly here.
2.2 Co-Disposal of Metal Mine Wastes

This literature review found five studies of co-disposal for metal mine waste rock and tailings, including Eger et al. (1984), Johnson et al. (1995), Wilson et al. (2000 and 2003a), and Leduc et al. (2004), with papers describing the present study excluded. Three concept papers (Brawner 1978, Wilson 2001, Vector Engineering Personnel 2004) are also included in this review.

Brawner (1978) introduced the combined waste rock – tailings storage concept. Brawner (1978) hypothesized that at reasonably high stripping ratios of 3:1 or more it is theoretically possible to store all tailings in the void space of coarse waste rock. Practical methods suggested to implement the concept include introducing tailings into the voids of already dumped rock in thin lifts, or by introducing rock into soft, unconsolidated tailings. Main advantages cited include reduction of dust from tailings dumps, increased strength and long term stability, reduction of erosion of tailings, retardation of leaching and breakdown of rock, and reduction in land area required for disposal. Disadvantages cited include added cost to the mine operator (although considered to be substantially less than underground disposal), initial difficulty in finding a construction technique, and also increasing the difficulty of re-processing tailings at a later date (Brawner 1978).

Eger et al. (1984) examined drainage quality and quantity for mixtures of tailings and waste rock produced from a massive intrusive formation containing low-grade copper and nickel sulphides called the Duluth Gabbro Complex. The purpose of the study was to determine whether or not mixing waste rock and tailings would inhibit the production of
acid rock drainage (ARD). The waste rock used was known to be ARD producing and contained 0.63% sulphur, while tailings contained approximately 0.4% sulphur. Mixtures were prepared by placing a 0.10 m lift of dry tailings onto a 0.6 m lift of waste rock and then washing the tailings into the waste rock with water. Three configurations were tested in bins instrumented with base and surface drains, and oxygen samplers including:

i) tailings mixed with waste rock,
ii) tailings mixed with waste rock with a 0.15 m thick dry tailings cover, and
iii) waste rock only as a control.

Each bin was 1.5 m wide, 9.8 m long, and 1.8 m in height. Each profile included a sloping surface at 3:1. The bins were located outdoors, and allowed exposure to atmospheric conditions near Babbitt, Minnesota. Drainage quantity and quality was monitored during 1983, vegetation growth was periodically measured, and oxygen concentrations within the bins were measured. No significant reduction in oxygen content was observed in the mixtures or waste rock. Tailings placed on surface as a vegetative medium were observed to erode during heavy rainfall. Drainage was reduced by 66% and 33% for vegetated and non-vegetated mixtures compared to waste rock due to increased storage and evaporation. Trace metal release was 87% and 72% lower for vegetated and non-vegetated mixtures compared to waste rock, respectively, in part due to the buffering effect of the tailings. The study concluded that mixing rock and tailings successfully reduced trace metal release, but that metal concentrations were still higher than for local waters. It was also noted that thorough mixing was impossible with the methods used and that no attempt was made to optimize the mixture ratio (Eger et al. 1984).
Johnson et al. (1995) investigated mixtures of goldfield tailings and waste rock in lysimeter and column studies. The purpose of the study was to quantify water recovery, relative masses of wastes and tails that could be mixed, and also determination of the order in which co-disposed materials should be placed. Detailed particle size distributions were not provided, but it was noted that waste rock used in the tests was crushed to pass 12 mm. The pulp tailings used in the test were 50% solids (mass solids by mass total). The lysimeters used were boxes 1.2 m across with a surface decant and underdrain. Three mixture configurations were tested. The first lysimeter contained tailings only as a control. The second lysimeter contained waste rock added to tailings slurry. The third lysimeter contained tailings slurry added to waste rock. It was noted that the lysimeters with waste rock immediately produced a competent surface that did not visibly distort when walked on, while the tailings remained wet and soft. The tailings only lysimeter was disturbed in an effort to induce drainage from the underflow. All lysimeter tests were monitored for approximately 160 hours, with results summarized in Table 2.2 (Johnson et al. 1995). Estimated length of column and lysimeter fills presented in Table 2.2 are based on assumptions of 1.2 m square lysimeters, 0.15 m diameter columns, and specific gravities of 2.7 and 2.8 for waste rock and tailings solids, respectively.
Table 2.2. Summary of results from Johnson et al. (1995).

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Ratio of waste rock to tails by dry mass</th>
<th>Water Recovered from Tails (%)</th>
<th>Estimated Length of fill (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lysimeters:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tailings only</td>
<td>1:1</td>
<td>44</td>
<td>0.3</td>
</tr>
<tr>
<td>waste rock into tailings</td>
<td>3.8:1</td>
<td>39</td>
<td>0.5</td>
</tr>
<tr>
<td>tailings into waste rock</td>
<td>4.3:1</td>
<td>38</td>
<td>0.5</td>
</tr>
<tr>
<td>Columns:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tailings only</td>
<td>0:1</td>
<td>51</td>
<td>1.6</td>
</tr>
<tr>
<td>tails into waste rock</td>
<td>4.8:1</td>
<td>9</td>
<td>1.7</td>
</tr>
<tr>
<td>waste rock into tailings</td>
<td>1.2:1</td>
<td>50</td>
<td>2.1</td>
</tr>
<tr>
<td>tailings into waste rock*</td>
<td>4.1:1</td>
<td>23</td>
<td>2.2</td>
</tr>
</tbody>
</table>

*(minus 1 mm material removed from the waste rock)*

Johnson et al. (1995) also studied drainage of mixtures in columns to try to quantify an advantage to adding tailings to waste rock or vice versa. Four configurations were tested, with results summarized in Table 2.2. The waste into tails column produced water for approximately 400 hours after which time all column mixture trials were stopped. The tailings only column produced water for more than 1000 hours after which time production levelled off and the test was stopped. Conclusions and observations from the study by Johnson et al (1995) included:

i) The percentage of water recovered was similar for mixtures as for tailings without rock present.

ii) The rate of drainage from mixtures was faster than for tailings due to the mechanism of preferential flow along the surfaces of waste rock particles.

iii) Waste rock should be deposited into the tailings rather than vice versa unless the waste rock is coarse because the presence of fine material in the waste rock inhibits the flow of tailings into the waste rock void space.
iv) Many gold operations have strip ratios greater than 7:1, so there is room in the 30% void space of waste rock to store the tailings.

v) It may be possible to increase water recovery compared to conventional tailings disposal by mixing

vi) Stability is an important consideration

vii) Less engineering may be required than conventional means of disposal because there should be no need for a liquid retaining structure

viii) Rehabilitation will be simpler than conventional means and may be progressively completed during operations

Advantages of co-disposal appear to include: no need for a separate tailings storage facility or tailings impoundment rehabilitation, less land disturbed, more stable waste dumps, increased water and reagent recovery from waste. Disadvantages included increased supervision of tailings deposition and also more design input for waste dump layout and waste scheduling (Johnson et al. 1995).

Wilson et al. (2000) presented results of a computer simulation comparing saturated/unsaturated flow in a mixture of 70% waste rock and 30% tailings, compacted glacial till from the Equity Silver Mine, and coarse waste rock. The conceptual study indicated potential improved performance of co-disposed materials relative to waste rock alone for prevention of ARD and emphasized the need for investigation of co-disposal as a mine waste disposal technique (Wilson et al 2000).
Wilson (2001) provided a general overview of the state of co-disposal in the mining industry. Co-disposal had been implemented for pumped coal washery wastes at several mines in Australia, USA, and Indonesia. In-pit co-disposal is used at metal mines, such as Kidston Gold Mines in Australia, in a non-integrated form where waste rock and tailings are dumped into the same mining void. Co-disposal has the potential to prevent both ARD and catastrophic failure of tailings dams. Waste rock hydrology and co-disposal were noted to require further study (Wilson 2001).

Wilson et al. (2003a) presented theoretical constitutive surfaces of volume change and water retention for tailings, waste rock, and a mixture of waste rock and tailings. Hydraulic conductivity data were compared for several laboratory studies of co-disposal. The study indicated that mixed waste rock and tailings can have hydraulic conductivities approaching $10^{-9}$ m/s and air entry values greater than 100 kPa (Wilson et al. 2003a).

Leduc et al. (2004) presented an evaluation of co-disposal technology for cyanide mill tailings from the proposed Esquel mine in Argentina. Argentinean regulatory agencies and proponents did not view conventional tailings disposal be a good option for the site, and co-disposal was investigated as an alternative. The main potential benefits of tailings co-disposal were stated to be the lack of a tailings impoundment and reduced potential for ARD by restriction of access of water and oxygen to the waste rock dump. Options for physical placement were also examined and included:

i) placement of tailings in ponds or layers in the dump,

ii) blending, either in a haul truck or by mixing at dump crest,
iii) injection of tailings paste into an existing waste rock dump, and
iv) application of thin layers of tailings to the face of a waste rock tip.

Stability and permeability were considered to be crucial to design. Leduc et al. (2004) presented data indicating an increase in both friction angle and permeability with mixture ratio. Mixture ratios tested include 4:1, 2:1, 1:1, and tailings only for permeability. Hydraulic conductivity was shown to decrease from approximately $5 \times 10^{-3}$ m/s for a 4:1 mixture to approximately $2 \times 10^{-8}$ m/s for tailings only. Direct shear testing was conducted on mixture ratios of waste rock only, 8:1, 6:1, 4:1, 2:1, 1:1, 0.5:1 and tailings only. It was noted that friction angle decreases sharply between 4:1 and 2:1, but all mixtures, including tailings alone, had friction angles above 30°. No details on density or water content were provided. Addition of binders such as Portland cement may be required to stabilize the co-disposed waste mass (Leduc et al. 2004). Vector Engineering (2004) further described the Esquel study related by Leduc et al. (2004). Costs for the proposed Esquel project in Argentina were estimated to be $0.50/tonne for co-disposal versus greater than $1.00/tonne for a lined tailings impoundment (Vector Engineering 2004). Development of the Esquel project was suspended due to local opposition.

2.3 Co-Disposed Materials for Cover Systems

This literature review found two papers that investigated the use of co-disposed waste materials as cover materials for unsaturated wastes, including Williams et al. (2003b), and Fines et al. (2003a).
Williams et al. (2003b) evaluated the potential of using co-disposed waste materials as an oxygen/water barrier in a store and release cover system for Cadia Hill Gold Mine in New South Wales, Australia. One benefit of using mine wastes as cover materials is on-site availability, unlike traditional cover materials like clay borrow. Materials investigated included waste rock, trafficked waste rock containing fines, clay borrow materials, and tailings. Mixtures with ratios of 5:1, 10:1, 15:1 and 20:1 for trafficked rock and tailings and one mixture of 5.5:1 untrafficked rock and tailings were also investigated. Laboratory testing included determination of particle size distribution, specific gravity, Atterberg Limits, flakiness index, compaction, soil-water characteristic curves (SWCC), and saturated hydraulic conductivity. The waste rock used in the tests was scalped of sizes greater than 75 mm. The tailings used in the tests were primarily sand to silt sizes prepared at 58% solids, which represented the average solids concentration for thickener underflow at the Cadia Hill Mine. Trafficked waste rock was found to be well graded compared to untrafficked rock and was used in most of the mixtures tested. Hydraulic conductivity of mixtures was found to increase with mixture ratio, ranging from $1\times10^{-8}$ m/s for a 5:1 mixture ratio to $7\times10^{-6}$ m/s for a 20:1 mixture ratio. It was noted that the 5:1 mixture had a water content that was wet of optimum for compaction, but had a high density without compaction. Mixtures with ratios equal or higher than 15:1 could be compacted to reduce hydraulic conductivity by an order of magnitude, but had high values of hydraulic conductivity prior to compaction. It was also demonstrated that tailings slurry prepared wet had a lower hydraulic conductivity than tailings slurry prepared from dry tailings and water. Mixtures of trafficked waste rock and tailings at a ratio of 5:1 by dry mass, or 3:1 by wet mass were recommended for
use as cover materials due to high density and low permeability without compaction. The study recommended that the cover of co-disposed material be covered with further material to prevent desiccation. The additional cover layer is required, presumably, to act as a store and release layer, but also to prevent desiccation of co-disposed materials. Desiccation may result in a reduced ability to prevent the ingress of oxygen due to desaturation, and possibly cracking (Williams et al. 2003b).

Fines et al. (2003a) presented results of a laboratory testing program investigating the feasibility of using mixtures of waste rock, tailings and slag from Copper Cliff Mine as cover materials. Tests were conducted to determine particle size distribution, slump, compaction, hydraulic conductivity, and soil-water characteristic curves. Variation of hydraulic conductivity of mixtures was investigated for addition of bentonite (1.5% by mass) and for compaction. Addition of 1.5% bentonite resulted in hydraulic conductivities as low as $5 \times 10^{-9}$ m/s for mixtures of 1:1:2 waste rock to slag to tails (by mass). Similarly, Standard Proctor compactive effort resulted in hydraulic conductivities as low as $4 \times 10^{-8}$ m/s for mixtures of 1:1:2 waste rock to slag to tails (by mass). Results indicated that it is feasible to use mixtures of wastes as cover construction materials based on the criteria of hydraulic conductivity, density, handling, and volume change. Slump tests indicate that mixtures may be pumped but may require bentonite addition or compaction to achieve low permeability (Fines et al. 2003a).

2.4 Co-Disposal of Coal Washery Wastes

Washing of mined coal to meet industry specifications produces coarse granular wastes and also fine grained wastes referred to as tailings. Coarse coal wastes are subject to
ARD and also to spontaneous combustion due to the heat produced by the exothermic ARD reaction. Coal tailings are typically silt sized or finer than 0.06 mm (Williams 1996c). Williams (1996c) used the term “coarse reject” to describe coarse coal wastes up to 100 mm diameter in size. Williams and Kuganathan of the University of Queensland, Australia, contributed the majority of work done in the area of pumped co-disposal. Stewart and Atkins (1982) examined engineering properties of combined coarse and fine coal wastes. Van Rooyen (1992) investigated layered co-disposal of coarse and fine coal wastes. Gosling (1999) presented a study of co-disposal of coal washery wastes with sand mine wastes.

2.4.1 Pumped Co-Disposal

Shortage of tailings disposal facilities in the Hunter Valley Coalfields in Australia initiated the use of expensive mechanical thickeners to consolidate the tailings as a cake that could be trucked for disposal with coarse reject. Williams (1990) suggested several alternative practices for waste disposal. Observations at the Mount Thorely Coal Mine indicated that it was possible to place a significant quantity of coarse waste into uncrusted tailings slurry before exceeding the bearing capacity of the slurry. Williams (1990) suggested pushing coarse wastes into tailings slurry: “…if the two materials can be mixed to create a loose packing of coarse waste in a matrix of tailings, a mixture with reasonable engineering material properties may result.” The approach was anticipated to reduce the cost associated with conventional dams and rehabilitation of tailings deposits. Williams (1990) noted that potential problems would include loss of return water and a loss of stability of spoil piles due to the introduction of fine materials. Williams (1991)
discussed co-disposal of coarse and fine wastes in spoil piles and also reported the success of trials of pumped co-disposal at the Jeebropilly Mine.

Williams and Kuganathan (1992b) provided details on the operational implementation of co-disposal of coarse and fine coal wastes by combined pumping at the Jeebropilly Mine in the Ipswich Coalfields, Queensland, Australia. Conventional practice for tailings disposal had included thickening of tailings with 0.05% flocculant to achieve 30% to 35% solids by mass for sub-aerial disposal in an impoundment. Tailings would undergo beaching, sedimentation, self-weight consolidation, and crusting if surface desiccation occurred. The resulting geometry included tailings beach slopes of 100:1 with finer particles at greater distances from the discharge due to hydraulic sorting. Alternative methods of mechanical dewatering required further use of 0.5% flocculant by mass to achieve solids contents of near 70%. The cost of flocculant for mechanical dewatering was a prohibitive $1 to $2 per tonne of coal produced. The newly implemented method of pumped co-disposal involved combining coarse and fine wastes with water in a hopper at approximately 30% solids and mixture ratios of approximately 5.3:1 (all mixture ratios relate coarse to fine wastes by dry mass), and then pumping the waste to a pit or depression for disposal. Particles from clay sizes to 100 mm were pumped through a 200 mm inner diameter pipe over distances greater than 1 km. Beach slopes forming from combined wastes were approximately 15:1 and were found to be immediately trafficable by machinery. Segregation of fines by hydraulic sorting was noted to occur due to the high velocity and low solids contents required for pumping coarse materials. The method of pumped co-disposal was used for disposal in depressions only and it was noted that demonstration of stability would be required for elevated disposal above the
existing ground surface. The costs of pumped co-disposal were estimated to be about half that of conventional waste disposal practice at Jeebropilly (Williams and Kuganathan 1992b).

Williams and Kuganathan (1992b) carried out an intensive testing program to determine basic geotechnical properties of co-disposed, pumped coal washery wastes. Shear strength tests were conducted in a 100 mm square direct shear box. Wastes scalped of sizes greater than 9.5 mm had friction angles of greater than 32° over a normal stress range of 24 to 1000 kPa. Rowe consolidation tests of 250 mm diameter specimens scalped of sizes greater than 19.5 mm indicate values of compression index ($C_c$) of approximately 0.8 between 10 kPa and 100 kPa, and near 1 between 100 kPa and 1000 kPa. Values of recompression index ($C_r$) for the same specimens were between 0.01 and 0.02 for the range of 10 kPa and 1000 kPa. Initial void ratios were 0.53 to 0.48, and final void ratios were approximately 0.34 following unloading. Compressibility was not found to vary significantly with particle size distribution. Hydraulic conductivity was tested using a purpose built 300 mm diameter consolidation cylinder that allowed constant head testing while maintaining a confining load. Specimens were loaded in increments between 0 kPa and 600 kPa. Specimens were allowed to consolidate at each load increment and then tested for hydraulic conductivity by constant head test. Hydraulic conductivity was found to vary with both confining stress and with hydraulic gradient. Mixed coarse and fine wastes were found to have hydraulic conductivities that were three orders of magnitude greater than fine wastes alone. The increase in hydraulic conductivity due to the addition of coarse waste was attributed to preferential seepage.
along the relatively flat surfaces of coarse particles. Flat surfaces prevented fine particles from packing tightly and thus created preferential flow paths (Williams and Kuganathan 1992b).

Williams and Kuganathan (1992a) examined the effect of varying mixture ratio on static strength, consolidation, and coefficient of permeability for mixtures of coarse and fine coal wastes from the New Hope Colliery, Australia. Angle of effective friction was investigated in a 100 mm square direct shear box for mixture ratios of 9:1, 5.7:1, 4:1, and 1.5:1 (ratios relate coarse to fine wastes by dry mass) over normal stresses varying from 24 kPa to 1000 kPa. Angle of effective friction was greater than 30 degrees for all specimens, and it was concluded that drained strengths were not an important parameter in determining optimum mixture ratio. Compressibility testing was carried out in a 250 mm diameter Rowe consolidometer on specimens scalped of sizes greater than 19 mm. Mixture ratios tested included 4:1, 2.3:1, 1.5:1 and 1:1.5 for applied stresses ranging from 10 to 640 kPa. Specimens were observed to dissipate pore pressures rapidly and thereafter undergo small ongoing settlements (creep). Consolidation results were compared to oedometer results for tailings only. It was noted that void ratio decreases with higher mixture ratios and that compressibility is an important factor for determining mixture ratio (Williams and Kuganathan 1992a).

Williams and Kuganathan (1992a), and Williams (1992) presented hydraulic conductivity testing results for specimens with mixture ratios of coarse only, 4:1, and 1.5:1 (all ratios relate coarse to fine wastes by dry mass). Specimens were tested in a purpose-built
consolidation cylinder (likely the same as described above) that allowed constant head tests of 300 mm diameter specimens scalped of sizes greater than 19 mm. Specimens were loaded with applied stresses ranging from 0 to 600 kPa. For each load increment specimens were allowed to consolidate and then tested for hydraulic conductivity by constant head test under gradients as high as 0.9. Specimens with mixture ratios of 9:1 were tested for hydraulic conductivity in an unconsolidated state in an 80 mm diameter cylinder (specimens were scalped of sizes greater than 9.5 mm). Results indicate that hydraulic conductivity is highly dependent on mixture ratio and slightly dependent on hydraulic gradient. Dependency on hydraulic gradient was more pronounced with a higher proportion of tailings. Hydraulic conductivity ranged from $10^{-9}$ m/s to $10^{-07}$ m/s for tailings alone, to $5*10^{-5}$ m/s for mixtures with ratios of 1.5:1, to $4*10^{-2}$ m/s for coarse waste alone. It was concluded that permeability is a key parameter to consider in deciding optimum mixture ratio (Williams and Kuganathan 1992a).

Williams and Kuganathan (1993) presented results of a testing program to determine geotechnical properties of samples of pumped wastes from the Jeebropilly Mine, as well as laboratory-constructed mixtures of coal washery wastes composed of Gordonstone run-of-mine coal. Shear tests, Rowe consolidometer tests, and permeability tests were conducted (presumably) using similar equipment as described by Williams and Kuganathan (1992a and 1992b). Specimens from Jeebropilly at mixture ratios of 5.3:1 and those constructed with Gordonstone run-of-mine wastes at mixture ratios of 3:1 and 1:1 were found to have similar angles of internal friction as specimens from the New Hope Colliery described by Williams and Kuganathan (1992b). Rowe consolidometer
testing of Jeebropilly specimens at mixture ratios of 5.3:1 and 1.9:1, and of Gordonstone constructed specimens at mixture ratios 3:1 and 1:1 were found to be reasonably similar to results from New Hope Colliery, and were insensitive to mixture ratio. Hydraulic conductivity testing of Jeebropilly specimens at mixture ratios of 5.3:1, and of Gordonstone constructed specimens at mixture ratios 1.5:1 were found to broadly confirm results from New Hope Colliery specimens reported by Williams and Kuganathan (1992a). However, it was noted that the hydraulic conductivity of Gordonstone and Jeebropilly specimens were more and less sensitive, respectively, to changes in applied pressure than New Hope Colliery specimens. Finally, it was concluded that co-disposal has the potential to tie up less water and occupy 60% to 70% of the volume required by conventional disposal methods (Williams and Kuganathan 1993).

Williams and Gowan (1994) reviewed co-disposal practices in Australia, as summarised in Table 2.3.

Table 2.3. Pumped co-disposal in Australia summarized from Williams and Gowan (1994).

<table>
<thead>
<tr>
<th>Location</th>
<th>Mixture ratio for pumping (coarse to fine by dry mass)</th>
<th>% Solids Pumped</th>
<th>Pumping Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jeebropilly Colliery</td>
<td>5.7:1 to 5:1</td>
<td>30</td>
<td>4.5</td>
</tr>
<tr>
<td>Gordonstone Mine</td>
<td>1.5:1</td>
<td>20-25</td>
<td>4.2</td>
</tr>
<tr>
<td>Goonyella Mine</td>
<td>5.5:1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hunter Valley No.1 Mine</td>
<td>2:1 to 4.5:1, average 3.5:1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Williams and Gowan (1994) noted that the optimum mixture ratio for retention of all tailings within the coarse reject varied with porosity and specific gravity of both coarse
and fine wastes, but was typically in the range of 3:1 to 5:1 (coarse to fine by dry mass). Excess tailings tended to segregate and form a tailings deposit downslope of the mixed beach. Deposits with excess coarse materials have reduced engineering properties. All co-disposed materials were deposited in depressions or pits. However, trials to use pumped co-disposal to produce an elevated landform were proposed for the Gordonstone and North Goonyella Collieries. Advantages cited for the use of co-disposal include elimination of the tailings impoundment, improved stability and the potential for elevated landform disposal, elimination of haul trucks and drivers for coarse reject, reduction in operating costs of up to 85%, recovery of up to 40% more water from wastes, savings of up to 40% in volume, better rehabilitation, and opportunities for landscaping and reclamation of existing tailings deposits. Disadvantages included pipeline blockages, pipe wear and replacement, and also segregation of fines (Williams and Gowan 1994).

Williams et al. (1995) reviewed the history of worldwide implementation and practice of co-disposal of coal mine wastes. Mechanical co-disposal of coarse reject and de-watered tailings occurred at a number of collieries worldwide but costs for mechanical dewatering were high. Pumped co-disposal had been tried at coal mines in the UK, in South Africa, and had been used primarily in Australia, but also the USA, and Indonesia (Petangis). Williams et al. (1995) reviewed practice of co-disposal at Gordonstone and Petangis Collieries, and at North Goonyella mine. Gordonstone and Petangis designed co-disposal facilities with downhill deposition with a series of small leaky catch wall dams to retain suspended tailings above a water return dam. Both sites observed more segregation of fines than expected, with little settling of fines occurring downstream of the co-disposal
beach. The tailings then flowed and built up behind water return dams. In contrast, the North Goonyella mine used uphill deposition with a tailings beach forming and being covered by the co-disposal beach. Tailings-free supernatant water was filtered by leaky catch wall dams and then decanted off the tailings beach via trenches. It was concluded that discharge of pumped co-disposal should be in an uphill direction. It was also concluded that excess tailings beaches due to segregation are inevitable for pumped co-disposal (Williams et al. 1995).

Williams (1996a, 1996b, 1996c, 1997) provided an overview of the logistics, advantages and disadvantages of pumped co-disposal. Williams (1996c, 1997) noted that loose dumped coarse reject is subject to oxidation by spontaneous combustion and ARD, but pumped co-disposal solved this problem at Jeebropilly Colliery. Williams (1996a) reviewed tailings characterization, methods of tailings dam construction, in-pit tailings disposal, underground tailings disposal, underwater disposal, central thickened tailings discharge, mechanically dewatering of tailings, and finally co-disposal of coarse wastes and tailings. Gravel size particles of coarse waste may be used with cement and coarse tailings as a backfill for underground workings. Co-disposal of larger size particles requires mechanical mixing and is suitable where co-disposed materials may be deposited into a mining void or where the tailings may be incorporated within a coarse waste dump. Methods of mechanical mixing include progressively pushing coarse waste into uncrusted tailings or the tailings slurry may be sprayed over loose coarse waste. Williams (1996a) stated: "There is an optimum mixing time after tailings disposal at which the coarse reject should be added to achieve good mixing. In the field this is between one and two days.
If the coarse reject is added immediately after tailings disposal, the coarse reject particles will form a segregated layer at the base of the storage. If the delay before addition of coarse reject is too long, the tailings will settle to the base of the storage, forming a segregated layer." Co-disposal or mechanical mixing of waste rock and tailings was under consideration for several Australian gold mines and included backfilling tails and rock into an existing mining void (Williams 1996a).

Williams (1998) compared depositional behaviour of coal tailings, loose-dumped coal reject, and pumped co-disposed coal wastes. Observations of note include the fact that coarse rejects that are prone to breakdown initially experience an increase in friction angle as fines fill the voids between the coarse particles. With further increase in the proportion of fine particles, the friction angle will decrease (Williams 1998).

2.4.2 Problems with Pumped Co-Disposal of Coal Washery Wastes

Segregation remains a problem with pumped co-disposal. Morris and Williams (1997c) reported results from field trials using a transportable pumping test rig for mixtures of coarse rejects and tailings from Goonyella-Riverside, Hunter Valley No. 1, and Warkworth mines. Ratios tested ranged from 0.52:1 up to 3.6:1 coarse reject to tailings by dry mass. In situ density of co-disposed pumped wastes were similar to that achieved for mechanical compaction of coarse wastes alone. Segregation remained unavoidable due to gap grading and fine particles in the flow (Morris and Williams 1997c). Fine particles are required for the hydraulic transport of larger particles (Sellgren and Addie 1997). Studies by Morris and Williams (1997a, 1997b) and Williams (1998) demonstrated that existing theory for sub-aerial deposition can be used to deal with
segregation problems for pumped co-disposal wastes and that deposition of segregated and co-disposed coarse and fine wastes can be reasonably well described by the river transport approach, respectively. Morris and Williams (1999) investigated segregation of co-disposed wastes by examining percentage solids retained on beaches. Morris and Williams (1999) introduced some theory of mixture design as it relates to segregation and the porosity of co-disposed wastes that is discussed in Chapter Three.

2.4.3 Other Co-Disposal of Coal Wastes

Stewart and Atkins (1982) examined engineering properties of mixtures of coarse and fine coal wastes. Specimens with 30% to 40% fines content had maximum density and minimum permeability while shear strength increased with fines content from 20% to 40%, then increased slightly or levelled off at fines contents of 60%. It was noted that somewhere between 30% and 40% fines content the fine particles “...fill the voids of the coarse particles but do not replace or spread out the coarse particles.” Optimum water content and permeability were the properties most influenced by addition of fines (Stewart and Atkins 1982).

Spontaneous combustion is known to occur in coal waste dumps due to oxidation of pyrite, producing temperatures greater than 350°C within the dump. Van Rooyen (1992) investigated the saturation of thin lifts of coarse coal waste with tailings slurry in an attempt to reduce spontaneous combustion. Finer materials are less prone to spontaneous combustion than coarser ones because of inhibition of oxygen transport. The field trial included deposition of coarse waste smaller than 150 mm in diameter deposited in layers approximately 200 mm thick. Tailings slurry was poured onto each lift of coarse waste
until, presumably, the voids of the coarse waste were filled. No excess slurry was left on surface. Traffic of coarse waste dumpers was continually moved to assist in compaction of co-disposed wastes. The resulting mass and a control coarse waste pile constructed by end dumping were tested for void ratio, particle size distribution, infiltration by ring infiltrometer tests, compaction by dynamic cone penetrometer, density, and water content (Van Rooyen 1992). Results are summarized in Table 2.4.

Table 2.4 Summary of results from layered co-disposal trials by Van Rooyen (1992).

<table>
<thead>
<tr>
<th>Waste</th>
<th>Void ratio (%)</th>
<th>Ring infiltrometer (mm/minutes)</th>
<th>Dry density (g/cm³)</th>
<th>Moisture (volume %)</th>
<th>Dynamic cone penetrometer (mm/blows)</th>
</tr>
</thead>
<tbody>
<tr>
<td>compacted coarse</td>
<td>0.08</td>
<td>520/10</td>
<td>1.79 -</td>
<td>2.8 - 3.1</td>
<td>180/10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>512/20</td>
<td>2.09</td>
<td></td>
<td>260/20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>444/30</td>
<td></td>
<td></td>
<td>340/30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>324/40</td>
<td></td>
<td></td>
<td>360/40</td>
</tr>
<tr>
<td>un-compacted coarse</td>
<td>0.22</td>
<td></td>
<td>1.78</td>
<td>3.9</td>
<td>140/10</td>
</tr>
<tr>
<td>co-disposed wastes</td>
<td>0.08</td>
<td>180/10</td>
<td>1.7 -</td>
<td>5 - 5.6</td>
<td>180/20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>110/20</td>
<td>2.4</td>
<td></td>
<td>235/30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>70/30</td>
<td></td>
<td></td>
<td>295/40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60/40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tailings slurry</td>
<td>1.195</td>
<td></td>
<td>0.75 - 1.2</td>
<td>45.6</td>
<td></td>
</tr>
</tbody>
</table>

Observations during field testing indicated that vehicles made much less impression on co-disposed materials than on coarse waste alone. Vehicle movement was inhibited on layers of partially dried slurry. Ponding due to rainfall was also observed on co-disposed wastes. Co-disposed materials had reduced potential for spontaneous combustion compared to coarse waste alone due to higher water retention, which inhibits air recharge and enhances dissipation of heat. The integrated dump had 74% saturation, while the compacted dump had 41%. It was noted that improved water retention should aid in reclamation and may eliminate the need for application of a vegetative medium during
reclamation. The co-disposal dump achieved compaction test results and densities that were similar to compacted wastes, but did so without any deliberate compactive effort. Advantages of the layered co-disposal over conventional methods of disposal included:

i) reduced cost of compaction,

ii) reduced initial capital cost of large slurry dams,

iii) less land needed for construction of slurry dams,

iv) preservation of coal waste material in the dump for future use,

v) less topsoil required for reclamation, and

vi) less/no air and ground water pollution – preventative rather than reactive action.

Disadvantages included:

i) larger volume of return water is retained in the dump,

ii) slurry which may be sold is trapped in the dump,

iii) conversion costs for equipment, and

iv) danger of slurry spills due to damaged slurry pipelines.

The study concluded that different mines will have to try different mixture ratios (Van Rooyen 1995).

Gosling (1999) examined layered and homogeneous mixtures of coarse coal reject with tailings from the Clarence Colliery and adjacent Kable’s Transport Sand Mine, respectively. The investigation was designed to determine whether co-disposal of sand and clay tailings in the void space of coarse coal reject would reduce acid drainage by limiting oxygen transport and infiltration. The experimental method involved 22 mixed and layered configurations of co-disposed wastes in 205 L drums (565 mm diameter) equipped with plastic liners, base drains, and runoff collection systems. Ten drums were
filled with 8 cm lifts of coal reject and sand or clay tailings, with the base layer always being tailings. Ratios of lift materials were systematically varied. Heterogeneous mixtures of coal reject and sand and clay tailings were also tried, as were three controls of coal only, sand only and clay only. Three weeks after construction of drum fills, a constant flux of de-ionised water was applied at a rate of approximately 4 mL/min (8 times annual rainfall) to the surface of the drums for a least one month. Run-off and leachate volumes were quantified, and leachate was periodically tested for pH, conductivity, and metal concentration. Percolation tests were completed on co-disposed materials. After leachate had stopped, each drum was plugged with cement, then tipped and cut lengthwise with a plasma cutter. Drum fills were inspected for voids, preferential flow paths, visible oxidation, and material distributions. In addition, direct shear testing in a purpose built 300 mm by 300 mm shear box was conducted on specimens cut from drum fills. The results of the study indicated that none of the co-disposal options investigated lowered leachate metal concentrations to regulatory guidelines for allowable metal concentrations in discharge water. Advantages cited for co-disposal included use for re-shaping topography and revegetation, reduced ARD production due to reduced exposure of coal wastes to oxygen and water, and higher structural stability than segregated reject. Gosling (1999) concluded that co-disposal of coal washery reject with clay tailings from the sand mine is a feasible option for managing acidic drainage and recommended field trials.

Davies et al. (1998) examined the effect of addition of cement on the strength of mixtures of coal reject and thickened tailings with the purpose of stabilizing colliery spoil.
Consolidated, undrained triaxial testing of tailings and mixed waste specimens were conducted for different additions of cement. Mixed specimens tested were constructed with tailings with water contents between 25% and 66% at mixture ratios of coarse reject to tailings of 8:1 and 2:1. Addition of cement was predicted to increase the bearing capacity of mixed spoil to provide support of buildings with limited settlement tolerances (Davies et al. 1998).

2.5 Layered Co-Mingling of Waste Rock and Tailings
Layered co-mingling is a form of co-disposal where waste rock and tailings are placed in alternating lifts. Poulin et al. (1994) related an unpublished report that described layering of waste rock and dewatered tailings at the American Girl gold mine in California. Poulin et al. (1994, 1996) investigated layered co-mingling in leach columns (180 cm height, 15 cm diameter). The purpose of the investigation was to determine whether layered co-mingling of tailings and waste rock was a possible means of controlling ARD and mitigating its effects. Wastes included weathered waste rock and unweathered tailings from a copper porphyry deposit at Gibraltar Mine, BC. Columns were filled with rock crushed to pass 38 mm and compacted in lifts. Tailings were dewatered and placed in lifts compacted with a Proctor hammer. Geometry of placement included six lifts of material, three each for tailings and for waste rock. The ratios of rock to tailings (by mass) in the columns were rock only, 5:1, 7:1, and 7:1 with 3% cement by mass added to the tailings. Distilled water was delivered to the upper surface of the columns for 40 weeks and leachate quality was measured. Results from the study indicate reduced concentrations of metals and higher pH in leachate from co-disposed materials compared to waste rock alone. Poulin et al. (1996) concluded that layered co-mingling has the
potential to reduce ARD in the short term. The study was extended to include monitoring of oxygen and water content within the wastes, and also permeability testing of each layer in an attempt to evaluate capillary barrier effects (Poulin et al. 1998).

Lamontagne et al. (1999) presented results of a computer simulation study of layered co-mingling of waste rock and tailings. Waste rock properties similar to those from a gold mine, La Mine Doyon, Quebec, and tailings properties similar to those from a copper-zinc-silver-gold-mine, the Selbaie mine, Quebec were used as inputs to a multiphase, multi-component simulator for flow of fluids and heat in porous and fractured media. The model computed oxygen loss and heat production resulting from pyrite oxidation as a function of temperature, oxygen concentration, and pyrite mass fraction. Model geometries included a 30 m high waste rock dump with no tailings, and a similar waste rock dump with three horizontal layers of compacted tailings. Results from the simulations indicated that layered co-mingling reduces thermal convection, infiltration and the transport of contaminants. However, layered co-mingling did not eliminate ARD and was therefore considered to be a temporary measure (Lamontagne 1999).

Lamontagne et al. (2000) presented results from a column study based upon the work of Poulin et al. (1996, 1998). Waste rock and tailings from La Mine Doyon, and Selbaie mine, respectively, were used examined. The waste rock was known to be reactive. The tailings were silt to sand size, with a median diameter of 20 µm, and were described as 'typical' in physical, chemical, and mineralogical characteristics. Five columns were constructed (1.5 m height, 0.15 m diameter). One column contained waste rock only as a
control. The other four columns contained alternating layers of waste rock that were 0.5 m thick, and layers of tailings that were 0.1 m thick. In three of the layered columns the tailings were mixed with chemical additives. The experiment involved application of de-ionized water to the surface of five columns and subsequent monitoring of tailings layers for water content by TDR probe, waste rock layers for gas composition, and leachate for water chemistry. The experiment was run for more than 250 days and then bacterial counts were carried out for column fills. Results indicated that bacterial activity in the waste rock was limited but not completely inhibited by extremely low oxygen concentrations because ferric iron (Fe$^{3+}$) was still available as an electron acceptor. The study concluded that layered co-mingling can reduce ARD by limiting the transport of oxygen to diffusion through saturated tailings and by limiting the flow of water within the waste rock (Lamontagne et al. 2000).

Fortin et al. (2000) described the results from the columns with alkaline additives referred to by Lamontagne et al. (2000). Additives to the tailings layers included 10% cement kiln dust for column 3, and 10% red muds from the aluminum industry for column 5. Data collected were similar to Lamontagne et al. (2000). Results indicated that addition of alkaline materials reduced bacterial activity. Alkaline materials also acted as a trap for different metals subject to precipitation (Fortin et al. 2000).

Fortin and Poulin (2000) presented results technical and economic feasibility study of layered co-mingling with alkaline additives. An acid generating site, La Mine Doyon, Quebec, was used as a case study for estimating the cost for construction of a layered co-
mingled waste facility including addition of either cement kiln dust or red muds described by Fortin et al. (2000). Geometry of a theoretical co-mingled dump included three 10 m thick layers of waste rock alternating with three 1 m thick layers of compacted dewatered tailings combined with alkaline additives. Costs of implementation of approximately $0.79/tonne were predicted, which were in the same order as other methods such as collect and treat ($0.64/tonne), but less than relocation ($1.0-1.50/tonne). Costs for implementation were noted to be incurred during the life of the mine and there was a further savings if all tailings were co-disposed and no tailings dam was required. The study concluded that layered co-mingling with alkaline additives may be economically advantageous, but costs are sensitive to transport distances for alkaline additives (Fortin and Poulin 2000).

2.6 Underground Backfills
Mitchell and Smith (1979) stated that hydraulically placed tailings and other granular materials have been used for ground control in underground mine workings, “for many years”. Mitchell and Smith (1979) provided a design method that considered backfill quantity calculations, drainage requirements, percolation rate, and strength. Backfills consist of tailings and Portland cement. The addition of small quantities of cement does not initially alter the hydraulic properties of the tailings, but within days the cementation process reduces hydraulic conductivity. When slimes (minus 0.02 mm sizes) are included in hydraulic backfills there are typically problems with segregation of fine materials. Layers of fine material are soft and considered dangerous to the stability of exposed backfills. Elimination of slimes from tails used for backfill has been practiced to reduce problems in decant systems and to increase hydraulic conductivity of placed fills.
It was concluded that cemented backfills are useful if significant heights of backfill are to be exposed at a later mining stage, and that uncemented fills provide comparable confined support characteristics (Mitchell and Smith 1979). The design method suggested by Mitchell and Smith has been applied to backfills consisting of tailings, cement, and waste rock scalped or crushed to smaller sizes.

Kuganathan and Sheppard (2001) introduced a rocky paste backfill consisting of waste rock, thickened tailings, and Portland cement for the Mount Isa Mine. The mine had historically used an unmixed combination of cemented deslimed tailings and rock to fill underground voids. The coarse underflow from cycloned tailings was mixed with cement and ground slag. Rock fill from a nearby quarry was blasted, crushed, and sorted to produce sizes greater than 25 mm. Historically, rock fill to cemented tailings ratios were in the order of 1.5:1, and as high as 3:1. The rock and tailings were simultaneously discharged into the mining void from above, whereupon a cone of rock fill surrounded by a beach of cemented tailings formed. Segregation was a problem and new methods were considered as options for backfill. Two backfill options included cement and water with sized rock aggregate, and also rocky paste fill. Criteria for the fill included a 765 kPa strength at 28 days curing, rapid drainage to allow continuous filling without build up of pore-water pressures in the fill, and no release of water from the fill. Mixing trials, drop testing, and strength testing were conducted on both cemented rock fills and rocky paste fills and considered the effects of maximum particle size, grain size distribution, water content, binder content, and flow admixtures. Cemented rock fills were found to form a cone upon drop testing. Rocky paste fills composed of graded rock and cemented,
deslimed tailings produced non-segregating homogenous mixtures that “flowed like lava” to fill mining voids. Rocky paste fills also required the least amount of binder to achieve required strengths, and did not produce free water after placement. Testing indicated that an optimum ratio of 2:1 rock fill to tailings, with tailings at 72% solids produced mixtures at water contents of approximately 13%. Disadvantages of rocky paste fill included a high capital expenditure for an underground rock fill conveyor system (Kuganathan and Sheppard 2001).

Landriault et al. (2001) presented a paste disposal design of tailings and waste rock for underground workings the Bulyanhulu Mine in Tanzania. Waste rock, of which 75% was ordinarily hoisted to the surface, could be combined with dry tailings filter cake, Portland cement, and water to produce a backfill of desired slump and strength characteristics. Addition of waste rock to backfills would reduce the requirement for Portland cement to achieve the same strength as paste-only backfills. Addition of waste rock would also increase the strength of mixtures, and could increase pipeline wear if not managed properly. The proposed design used waste rock of minus 12.5 mm sizes at mixture ratios including 3:1 and 1:1 (Landriault et al. 2001).

2.7 Summary
The methods and major findings of studies of co-disposal available in the literature are presented. It should be noted that several private, unpublished investigations of co-disposal have been completed by consultants for mining companies, but are not included. In 2006, several mining/geotechnical consulting firms in Canada listed co-disposal as a
specialty or area of expertise. The present literature review has only considered published studies.

The use of cemented tailings and waste rock for ground control in underground workings was likely one of the earliest forms of co-disposal. Backfills have been constructed as a form of low strength concrete, with a binder such as Portland cement, and aggregate consisting of tailings and waste rock. The addition of rock to tailings backfills has been shown to increase the strength of backfills while reducing the requirement for cement. Tailings used for underground backfill may have smaller size particles, or slimes, removed to avoid problems such as low strength pockets or layers of segregated fine materials. Crushing and/or the use of quarried aggregate are required to attain a particle size distribution necessary for backfill drainage and strength properties. More recently, paste backfills have been documented for use as underground backfill.

As of 2006, pumped co-disposal had been implemented at several coal mines. The main reasons for investigation and implementation of co-disposal of coal mine wastes were economics and lack of space for wet tailings disposal in impoundments. Pumped co-disposal has since demonstrated several advantages over conventional means of disposal. Primary benefits of pumped co-disposal include:

i) no/smaller tailings dam required,

ii) fewer/no haul trucks required,

iii) reduced final area and volume of wastes,

iv) easier reclamation/trafficability,
v) reduced potential for ARD and spontaneous combustion of coarse wastes, and
vi) associated cost savings of greater than 50% compared to conventional disposal.

Pumped co-disposal has a maximum particle size limited by pipe size. The maximum
documented particle size used in pumped co-disposal was approximately 100 mm in
diameter. All pumped co-disposal was deposited into existing pits or depressions. This
literature review did not indicate the use of pumped co-disposal for construction of an
elevated landform. Mixture design for pumped co-disposal was dominated by concerns
for behaviour while pumping (rheology), including pipe blockages and wear on pumps
and piping. The mixture designs that have been implemented had low solids contents and
high fines content and have resulted in the segregation and beaching of finer materials.
However, the method of pumped co-disposal has proven to be a viable alternative mine
waste disposal technique for some coal mines.

Layered co-mingling has the potential to reduce ARD and eliminate the requirement of a
tailings impoundment by adding saturated layers of tailings to waste rock piles. The
addition of alkaline materials to tailings layers will further reduce ARD in the short term.
Layered co-mingling requires mechanically dewatered tailings and was implemented at
the American Girl gold mine, California, USA. Mechanically dewatered tailings were
also disposed with coarse wastes in the Hunter Valley Coal Fields in Australia prior to
implementation of pumped co-disposal.

Existing studies have demonstrated the potential for disposal of waste rock and tailings as
homogeneous mixtures to reduce or eliminate ARD, to facilitate water return, to
eliminate the need for tailings dams, and to accommodate larger waste rock sizes. While appearing to offer several potential benefits, the concept of mixing for disposal is unproven, and requires further research with respect to mixture theory, implementation methodology, and geotechnical properties and behaviour.
CHAPTER THREE. CONCEPTS AND THEORY FOR MIXTURES OF WASTE ROCK AND TAILINGS

The purpose of Chapter Three is to provide a theoretical basis for linking design to behaviour for mixtures of mine waste rock and tailings. Section 3.2 reviews existing theory for co-disposal mixture design, Section 3.3 reviews particle packing theory as a basis for a particle model presented in Section 3.4. The usefulness of the model introduced in Section 3.4 is demonstrated in Section 3.5 by a review of the relationship between particle structure and behaviour for other types of soil mixtures reported in the literature. Finally, Section 3.6 summarizes the relationship between mixture design and behaviour.

3.1 Introduction

Evaluation of mixtures as a mine waste disposal technique is complicated by the fact that geotechnical properties and behaviour depend the ratio of waste rock and tailings in a mixture. Waste rock and tailings have highly differing geotechnical properties. Waste rock is typically unsaturated, highly porous and permeable with large open voids, has high angles of repose and shear strength, and may include particles greater than one metre in diameter. In contrast, tailings are typically saturated, with low permeability, slow time-rate consolidation properties, and are composed of clay to sand size particles. The properties of a homogeneous mixture of waste rock and tailings will depend on the properties of the parent materials, but also on the mixture ratio. At one extreme, a mixture of 100% waste rock will behave like waste rock. At the other extreme, a mixture of 100% tailings will behave like tailings. The behaviour of the intermediate range of
mixtures will be dependent on the proportion of waste rock to tailings, or mixture ratio. Similarly, other mixture design variables such as tailings solids content and particle size distribution will affect the properties of a mixture. An understanding of the effect of mixture design variables on behaviour is therefore vital to the evaluation of mixtures as a mine waste disposal technique.

There is a lack of available theory for understanding the effects of mixture design variables on geotechnical behaviour. Chapter Three provides theory useful for the design of mixtures of waste rock and tailings. The theory presented is meant to facilitate the evaluation of co-disposal as a mine waste disposal technique, and to provide concepts and methodology for design of future mixtures. The theory facilitates evaluation of mixtures as a mine waste disposal technique by increasing knowledge and providing an understanding of the effects of mixture design variables on behaviour. The theory also facilitates mixture design by reducing the amount of empirical testing required, and by improving confidence in the design process. Existing particle packing theory is adopted as a means to understand mixture behaviour in terms of structure, or particle packing arrangements.

Chapter Three includes a review of existing literature of co-disposal for mixture theory, definitions of mixture design variables for waste rock and tailings, a review of literature on particle packing theory, definition of a conceptual particle model for mixture structure, and a review of literature for studies of the geotechnical behaviour of soil mixtures. The review of existing theory for co-disposal indicated an absence of a fundamental basis for
relating mixture design to mixture structure and behaviour. The review of particle packing theory provides a basis for understanding the effects of design variables on mixture structure. The conceptual model provides a basis for understanding mixture design with respect to particle structure. The review of geotechnical behaviours of soil mixtures indicates that mixture structure is quantifiable and can be directly linked to geotechnical behaviour. The theory and conceptual model are used for mixture design, and also for interpreting the behaviour of mixtures observed in the experimental component of this thesis included in Chapters Four through Six.

3.2 Mixture Theory for Co-Disposal

3.2.1 Existing Studies

Previous studies of co-disposal of mine wastes reviewed in Chapter Two were largely preliminary and used empirical approaches to mixture design, with the exception of pumped co-disposal and more recent work on mixtures of waste rock and tailings. Williams et al. (1995) and Morris and Williams (2000b) presented methods for the design and description of pumped co-disposal of coal washery waste mixtures that were partly based on particle packing theory. For mixtures of waste rock and tailings Wilson et al. (2003a), Fines et al. (2003a), and Williams et al. (2003b) approached the design of mixtures with a method of blending similar to that used for asphalt and concrete technologies.

Williams et al. (1995) provided a basis for choosing mixture ratio (coarse to fine by dry mass) for pumped co-disposal of coal washery wastes based on the void space available between coarse reject particles. Williams et al. (1995) stated that the mixture ratio that
develops upon hydraulic deposition of co-disposed coal washery wastes is "ideal" and occurs without physical blending. Pumped co-disposal results in hydraulic deposition of coarse and fine particles in an impoundment. Coarse particles settle out first in a configuration where particles are just touching. Finer tailings particles then fill in the void space of the resulting coarse particle structure. The ideal mixture ratio depends on specific gravity, particle size distribution, particle shape, and breakdown potential of the rejects. Collieries try to construct mixture designs with an ideal mixture ratio in order to maintain the desired physical properties of mixtures, and to reduce segregation of fines. If the actual mixture ratio (relating coarse reject to fine tailings by dry mass) is greater than the ideal, then a tailings shortfall occurs. If the actual mixture ratio is less than the ideal, then a tailings excess occurs. A tailings excess typically results in segregation and the formation of a tailings beach. Williams et al. (1995) noted that the ideal ratio can be determined from the volume of tailings that will "just fill" the void space of coarse reject.

Practical trials of pumped co-disposal at an ideal mixture ratio resulted in segregation because the velocities associated with hydraulic deposition did not allow the tailings to be retained in the coarse reject (Williams et al. 1995). Trials with a co-disposal pilot pumping rig indicated that certain factors increase segregation including: low mixture ratio, gap-gradedness of mixtures, irregular coarse particle shapes (which have higher porosities), low solids contents, and high discharge velocities. It was also noted that coarse reject may break down during transport and further reduce the mixture ratio. The method of pumped co-disposal used for coal washery wastes therefore does not produce ideal packings, and always results in segregation. Operations that have actually
implemented pumped co-disposal typically pump at low solids contents and lower than ideal mixture ratios to prevent pipe blockages as well as wear on pumps and piping (Williams et al. 1995).

Morris and Williams (1999) investigated segregation of pumped co-disposal and presented an empirical method to predict retention of solids on co-disposal beaches. Morris and Williams (2000b) used concepts of particle packing theory to predict matrix porosity of mixtures of coarse and fine coal wastes. For a mixture of coarse reject and tailings, a "perfect packing" was defined to occur at a specific mixture ratio where tailings particles "just fill" the voids of the coarse material. If coarse reject is added to the perfect packing case, then the tailings no longer fill the coarse reject void space, and the matrix of the mixture is considered to be made up of coarse reject only. If tailings are added to the perfect packing case, then the coarse particles lose contact with each other, or dilate, and the mixture is considered to have a tailings dominated matrix, with coarse particles as a part of the tailings matrix.

Morris and Williams (2000b) examined twelve deposits of co-disposed wastes formed by hydraulic deposition and pumping with respect to matrix type. Ten deposits were found to have coarse reject matrices, and two deposits were found to have tailings matrices. Matrix porosity was then related to the retention of solids on co-disposal beaches by an empirical equation. Further empirical relations were presented to relate solids retention to mixture ratio, initial gravimetric solids concentration, and diameters of waste inputs (Morris and Williams 2000b). With respect to particle packing theory, Morris and
Williams (2000b) referenced earlier studies of McGeary (1961) and Statham (1974) as a basis for predictions of initial porosity and structure of mixtures. While Morris and Williams (2000b) used particle packing theory to predict initial structure they did not relate structure to behaviour other than solids retention. Morris and Williams (2000b) state that volume change “...can be predicted by means of conventional geotechnical consolidation tests.” The work of Morris and Williams (2000b) is important to this thesis because similar assumptions and theory are used to predict the initial porosity of mixtures from specific gravity of waste rock and tailings, input tailings solids and water contents, and mixture ratio.

Wilson et al. (2003a) proposed a design method for mixtures of waste rock and tailings where tailings and waste rock are blended to produce a design gradation. Wilson et al. (2003a) noted that materials with well-graded particle size distributions, such as glacial till, have near ideal physical characteristics with respect to hydraulic conductivity and strength. Wilson et al. (2003a) blended waste rock and tailings with known particle size distributions in proportions calculated to duplicate or mimic the particle size distribution of a glacial till with good geotechnical performance. Mixture batch trials were tested for soil-water characteristic curves and saturated hydraulic conductivity following design. Fines et al. (2003a) used the same blending design method for mixtures of waste rock, tailings, and slag, but did not mention a design criteria or gradation. Williams et al. (2003b) blended tailings, fresh waste rock, trafficked waste rock, and two types of clay to produce well-graded particle design curves, and then tested the mixtures for geotechnical properties.
Of the methods proposed for design of mixtures of waste rock and tailings, this thesis more closely follows the method suggested by Williams et al. (1995) whereby the pore space between coarse particles is “just filled” with tailings. Wilson et al. (2003a) suggest a method of blending to duplicate a design particle size distribution. It is acknowledged that the method of blending particle size distributions suggested by Wilson et al. (2003a) has been proven to produce mixtures with beneficial material properties. However, waste rock and tailings typically have a large difference in absolute particle size. Consequently, mixtures of waste rock and tailings will be tend to be gap-graded, with stepped, or uneven particle size distributions. It is anticipated that significant effort would be required to produce a design gradation. For example, the preparation of aggregates for asphalts and concretes involves crushing, sizing, sorting, weighing and then recombination in specific proportions to achieve a mixture design.

The method of blending used by Wilson et al. (2003a), Fines et al. (2003a) and Williams et al. (2003b) is analogous or identical to methods used for design of gradations of asphalt pavements and concrete. Concrete and asphalt pavement technologies use performance-based mixture design methods (e.g. Goetz 1989, Asphalt Institute 1990, McIntosh 1966, Roberts et al. 2002) where mixture proportions are first chosen and then tested in mixture trials. The mixture proportions are then adjusted to meet criteria or performance-based specifications such as minimum compressive strength. The initial proportions of mixture components can be based on experience, or, more importantly, on particle packing theory, which is discussed below. German (1989) provided a list of
several empirically derived design particle size gradations producing minimum porosities.

Although mixtures with smooth, well-graded particle size distributions may be preferred and may perform better than gap-graded ones, it is unlikely that a well graded mixture with a smooth particle size distribution could be created by combining coarse waste rock with fine tailings. The tailings particle size distribution is typically selected to facilitate the mineral extraction process in the mill, while the waste rock particle size distribution is governed by blasting process and rock type. It is assumed here that tailings particle size distributions are inalterable. It is also anticipated that significant effort would be required to change the particle size distribution of waste rock for re-combination with fine tailings to produce a well-graded mixture. Consequently, the method of blending to reproduce a design particle size distribution will economically unfeasible in a mining context and is therefore not attempted here. The approach to the design of mixtures of waste rock and tailings used for this thesis is similar to that suggested by Williams et al. (1995). Mixture ratio, along with the gradations of waste rock and of tailings, are considered here as mixture design variables.

3.2.2 Design Variables for Mixtures of Waste Rock and Tailings

Previous studies of co-disposal have considered mixture particle size distribution and mixture ratio as design variables. A mixture design variable is defined as a variable that may be altered to effect a change in the properties of a mixture. A more comprehensive list of design variables for mixtures of waste rock and tailings includes:

i) initial particle size distribution of the waste rock,
ii) initial particle size distribution of the tailings,

iii) waste rock particle shape

iv) tailings particle shape

v) initial water content of the waste rock,

vi) initial solids/water content of the tailings, and

vii) the mixture ratio of waste rock to tailings.

Admixtures such as Portland cement, flocculants or other chemical treatments, and methods of mixing, placement, and compaction (e.g. vibration, static compaction) also affect mixture properties but are not considered here. From a practical point of view, the mixture design variable that is simplest to alter is mixture ratio. Water content of waste rock and tailings may also be readily increased. Water content of tailings may be reduced with some effort by settling in thickeners with or without flocculants, and by mechanical methods. Waste rock particle size distribution and shape are dependent on rock type and the use of explosives. Changing the waste rock particle size distribution or shape may require crushing and sorting, or a change in blasting technique. The tailings particle size distribution is typically pre-determined to aid in the extraction of production minerals. Tailings particle shape is determined by the milling process and rock type. Some extraction processes produce more than one type of tailings, differentiated by gradation and chemistry, which are subsequently re-combined for disposal.

Design variables considered by this thesis include mixture ratio and also initial tailings water/solids content. Mixture ratio is examined primarily in Chapter Three with respect to particle packing theory, but also in mixture trials presented in Chapters Four and Five.
Tailings water/solids content is briefly examined in Chapter Three, and also in practical investigations presented in Chapters Four and Five.

3.2.3 Summary and Discussion

The preliminary nature of existing studies of co-disposal has thus far precluded the development of theory for relating design to behaviour. Review of literature indicates that two approaches have been tried for co-disposal design. However, neither approach provides a theoretical basis for linking design variables to behaviour. The first approach was formulated for pumped co-disposal of coal washery wastes. The ideal or optimum mixture ratio for maximum packing efficiency is one where the voids of coarse spoil are "just filled" with tailings (Williams et al. 1995). The "just filled" approach is based on particle packing theory, and provides a maximum efficiency of storage. Where the technique has been implemented, as described in Chapter Two, operational concerns have overridden mixture design. The second approach to mixture design, proposed by Wilson et al. (2003a) involves the combination or blending of known particle size distributions to produce a design gradation. The approach is analogous to the method of blending used in the design of asphalt and concrete mixtures and is unlikely to be economical in a mining context. Neither Williams et al. (1995) nor Wilson et al. (2003a) have provided a theoretical basis for linking mixture design to geotechnical behaviour.

Mixture design variables that are assumed to be practically controllable include mixture ratio and tailings solids content. The effect of tailings solids content was investigated in laboratory investigations, which are described later in Chapters Four and Five. The effect of mixture ratio on particle structure and on behaviour was investigated through reviews...
of literature on particle packing theory and studies of soil mixtures. Particle packing theory is reviewed in Section 3.3.

3.3 Particle Packing Theory

Particle packing theory forms the basis of understanding material behaviour in many fields including pharmaceuticals, asphalt, sands, metal powders, gravel, catalysts, amorphous solids, nuclear fuels, ceramics, composites, coal, and metal-filled plastics (German 1989). The literature was reviewed for particle packing theory and concepts relevant to mixtures of waste rock and tailings. The review confirmed that values of mixture design variables, such as mixture ratio, can be selected based on theory, rather than simply by arbitrary choice or experience. The review also revealed that particle packing theory can aid in predicting and understanding the geotechnical behaviours and properties of mixtures.

Particle packing theory can provide a basis for the design and understanding of mixtures of waste rock and tailings, as it does for other particulate mixtures. The key contribution of particle packing theory is the provision of a logical basis for predicting mixture structure, or particle packing arrangement. The knowledge of mixture particle packing arrangements is useful because, as will be illustrated in Section 3.5, particle structure can be related quantitatively to geotechnical properties and behaviours. Structure predicted by particle packing theory provides a link between mixture design variables and performance, and the theory therefore provides a rational basis for mixture design. The term 'structure' is used here to refer to 'particle structure', or 'particle packing arrangement' throughout the paper.
The history of particle packing theory extends from examination of ordered arrangements of ideal spheres, (e.g. Barlow 1883) through many empirical studies, through analytical models (e.g. Yu and Standish 1991), to computer simulations of dynamically interacting distributions of real-shaped particles (e.g. Latham et al. 2002). As German (1989) stated, particle packing is of interest to many different groups, or fields of study. German (1989) also stated that a lack of communication between working groups has resulted in “replicated effort in learning the same basic principles over and over.” Basic concepts of particle packing theory are reviewed here to educate the reader, to provide an understanding of the depth of study, and to provide a basis for a particle model for mixtures of waste rock and tailings introduced in Section 3.4. Together, Sections 3.3 and 3.4 provide a theoretical basis for mixture design, and also for understanding mixture structure and behaviour.

The primary finding of the literature review of particle packing theory was that binary mixture theory is useful for predicting the structure of mixtures of waste rock and tailings. Investigations of particle packing considered for this thesis are listed in Table 3.1. Studies are divided into those that consider single size mixtures, binary mixtures (mixtures of two sizes), and multiple component mixtures. Most studies considered the porosity of packing arrangements, and the majority of concepts described here are related to the effect on porosity. A summary of particle packing phenomena relevant to mixtures of waste rock and tailings is included below in Section 3.3.4.
Table 3.1 Studies of particle packing.

<table>
<thead>
<tr>
<th>Subject</th>
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<tr>
<td>Packings of single size particles</td>
<td>Barlow 1883</td>
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<td>Graton and Fraser 1935</td>
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<td>Kolbuszewski 1948</td>
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<td>Rosenbluth and Rosenbluth 1954</td>
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<td>Alder et al. 1955</td>
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<td>Scott 1960</td>
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<td>Rutgers 1962</td>
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<td>Smalley 1962</td>
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<tr>
<td>Packing of two sizes of particles</td>
<td>Furnas 1928</td>
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<td>(binary mixtures)</td>
<td>Fraser 1935</td>
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### 3.3.1 Single Size Packing Arrangements

Initial studies of particle packing examined the arrangement and porosity of packings of single size ideal spheres. Single size packings may be ordered or random. Ordered packings are geometrically systematic and may be constructed by individually placing particles or through techniques involving mechanical agitation. Ordered packings of single size spheres are not directly relevant to waste rock or tailings particles, but do provide a basis for understanding minimum volume packing arrangements. Graton and Fraser (1935) defined six ordered packing arrangements for single size spheres. Barlow (1883) previously described five packing arrangements. The most compact packing arrangement of mono-sized spheres is called rhombohedral packing, and has a porosity of 26%, (defined as volume of voids divided by total volume). Non-ordered or random packing arrangements of ideal spheres have porosities ranging from 35% to 43% (Graton and Fraser 1935).
Attempts to formulate a theoretical treatment for random packings of mono-sized spheres include work by Rosenbluth and Rosenbluth (1954), Alder et al. (1955), Wood and Jacobson (1957), and Smalley (1962). The random variation in packing arrangements has stymied all efforts to accurately predict porosity, even for ideal spheres. The random nature of non-ordered packing arrangements make theoretical and mathematical description difficult, and investigations have tended to study physical models, e.g. steel balls (Scott 1960), nylon balls (Rutgers 1962), and ion exchange beads (Parrish 1961).

Key factors that affect the packing arrangements of single size particles include:

i) particle size,

ii) particle shape,

iii) container wall effect, and

iv) packing method.

3.3.1.1 Effect of Particle Size

For ideal particles, absolute particle size does not influence packing arrangement. Real particles are subject to friction, chemical and surface effects, gravity, and stress. The dominant forces acting on particles in a packing are determined by absolute particle size because absolute size is related to surface area. The continuum of particle sizes spans orders of magnitude, and the ratio of surface area to volume varies dramatically. Small particles, such as clay and silt sizes, are more subject to capillary phenomena, physico-chemical surface reactions, etc., than larger particles (Burmister 1938). For larger particles, such as sand to boulder sizes, gravity and inter-particle friction dominate. For even larger particles, the physical properties of the particles come into effect, with shape
distortion and breakage occurring due to higher stresses. As a generalization relative to the span of particle sizes, packings of smaller particles have larger porosities than packings of larger particles due to surface effects.

It should be noted that similar sized particles will be subject to similar forces, and slight differences in size are not important. Youd (1973) concluded that the maximum and minimum void ratio limits of clean sands are controlled by particle shape, size range, and the particle size distribution, but are not dependent on absolute particle size. For waste rock and tailings, it is expected that the packing of waste rock sizes is gravity dominated, and that tailings particles are more subject to capillary, surface and chemical effects. The packing of mixtures of waste rock and tailings are therefore treated individually by this thesis, as detailed in Section 3.4.

3.3.1.2 Effect of Particle Shape
Particle shape influences the porosity and/or density of particle packing arrangements. Contrary to intuition, spherical particles produce lower porosity packings than most other particle shapes. Non-spherical particles such as flat and needle-like forms have greater porosities than spherical shapes of equivalent volume. Fraser (1935) noted that a specimen of crushed mica with flat particles had a porosity exceeding 90%. Tons and Goetz (1968) investigated the effects of angularity and roughness (rugosity) for single size packing volumes and confirmed that the porosity of arrangements of irregular particle shapes is greater than for spherical ones. Dickin (1973) found that both maximum and minimum porosity decreased with increasing sphericity of sand and glass ballotini. Zou and Yu (1996) found that porosity was strongly influenced by particle
shape. Porosity was found to decrease with an increase of sphericity for loose packings. Porosity was found to decrease to a minimum, and then increase with sphericity for dense packings (Zou and Yu 1996). Exceptions of non-spherical shapes with lower porosity packing arrangements than spheres include recently fractured but undisturbed rock, and ellipsoids. As a general rule, porosity of a packing increases with particle angularity and deviation from a spherical shape.

Various methods have been developed in an attempt to quantify the shape of particles (e.g. Powers 1953). Waste rock particles examined in this study were angular and non-spherical. Angularity increases porosity of packing arrangements, and both tailings and waste rock are expected to have packing arrangements that are less dense than packings of spherical particles.

3.3.1.3 Container Wall Effect

Container wall effect is a localized increase porosity that occurs when particles are placed in a smooth walled container. With ordinary packing arrangements, spheres may interlock or mesh overlap like the teeth on a gear. The introduction of a flat surface, such as a container wall, prevents close packing and interlocking and results in an increase in porosity next to the flat surface. Graton and Fraser (1935) illustrated that the packing of discs was disrupted at a container wall, and the degree of disruption depended on container shape. Other investigations of container wall effect have noted an increase in the porosity of packings near surfaces that are flat or have different radii (e.g. Furnas 1929, Verman and Bannerjee 1946, Brown and Hawksley 1946, Ridgeway and Tarbuck 1966). An illustration of container wall effect is shown in Figure 3.1.
Examining Figure 3.1, it is evident that even a regular packing arrangement has smaller voids in the centre of the packing, somewhat larger voids at the side walls, and noticeably larger voids at the upper and lower container walls. Ridgeway and Tarbuck (1966) noted that the influence of the wall of a cylindrical container increased porosity of non-ordered packings of mono-sized particles to a distance of about five particle diameters from the container wall.

Container wall effect is mentioned here because it is important to the investigation of soil mixtures with different sized particles. Larger particles can act as a flat surface to disrupt the packing of smaller particles. Localized increases in porosity at the surface of larger particles will influence the permeability of a mixture. Applying the concept to mine wastes, waste rock particles may act as a flat surface that disrupts the packing of tailings particles, causing a localized increase in permeability.

3.3.1.4 Packing Method
The method of formation of a packing will affect the porosity of a packing arrangement. Application of energy by mechanical agitation, vibration, compaction, etc. can reduce the porosity of mixtures by temporarily increasing lateral and vertical acceleration. Various procedures have been developed to standardize the method of placement. For example,
Kolbuszewski (1948) proposed experimental procedures to determine the maximum and minimum porosity of sands.

3.3.1.5 Summary of Single Size Particle Effects

The particle size, particle shape, packing method, and container influence the porosity of particle packings. The large number of degrees of freedom with respect to geometrical arrangement, and even just particle shape, make theoretical simulation of mixture structure difficult. It should be noted that some level of empirical investigation or representation is currently required for the prediction of porosity of packing arrangements of particles due to particle shape irregularity, size effects, packing method, and randomness of packing. While the effects of size, shape, packing method, and container size are best illustrated by studies of mono-sized particles, the same effects apply to mixtures with more than one size.

3.3.2 Mixtures of Two Particle Sizes

A mixture of two groups of mono-sized particles with different diameters is called a binary mixture. The following concepts apply to ideal spheres, but also to other particle shapes where the two groups of particles have different mean diameters. The packing arrangements of binary mixtures depend on:

i) particle size ratio, or ratio of diameters of the component particles;

ii) mixture ratio, the proportion of each particle size; and

iii) the density of packing of the individual components.

Mixtures of waste rock and tailings are treated as binary mixtures by this thesis.
3.3.2.1 Particle Size Ratio

Particle size ratio is the ratio of the mean diameters of each group of particles in a binary mixture. Particles with a similar size ratio interfere with each other in the formation of close packing arrangements to a greater degree than particles with a large size difference. In binary mixtures with a large particle size ratio the smaller set of particles may occupy the spaces between larger particles. If smaller particles are smaller than the "critical ratio of entrance," defined as the pore throat diameter of an arrangement of the larger particles, then the smaller particles may move between the void spaces of the larger particles. The critical ratio of entrance for the tightest ordered packing (rhombohedral packing) is 0.154, or less than about a 7:1 particle diameter ratio (Graton and Fraser 1935). The concept of critical ratio of entrance is illustrated in Figure 3.2.

![Figure 3.2 Critical ratio of entrance for round shapes.](image)

In order for the small particle in Figure 3.2 to fit through the pore throat opening between the three large particles, it must be slightly smaller than $\frac{1}{7}$th the diameter of the larger particles. It should be noted that the critical ratio of entrance will vary with the randomness of packing, and that the value of approximately 7:1 represents a minimum
value of particle size ratio for the most efficient theoretical packing arrangement. Lees (1970) measured the void openings in plastic filled packings of real particles and found that the critical ratio of entrance is typically slightly smaller than 7:1. Furnas (1931), Epstein and Young (1962), and Ridgeway and Tarbuck (1968) demonstrated that as the particle size ratio increases, the porosity of a mixture decreases relative to the porosity of pure components. The effect is illustrated below.

3.3.2.2 Mixture Ratio

Mixture ratio plays an important role in the packing of binary mixtures and has a dominant effect on mixture structure. Mixture ratio is defined as the percentage (or ratio) of large to small particles and may refer to either mass or volume of particles. Fraser (1935) made several observations about the structure of binary mixtures. One extreme of mixture ratio is an arrangement of all small particles. The other extreme is an arrangement of all large particles. For a mixture dominated by larger sizes, the particle structure consists of a skeleton of large particles with smaller particles occupying the voids in the skeleton. For a mixture dominated by smaller particles, the structure consists of large particles that are isolated from each other, or “floating,” within a matrix of small particles. At a specific or singular mixture ratio the matrix of smaller particles ‘just fills’ the voids of the skeleton of larger particles. If large particles are added to the ‘just filled’ case, then the smaller particles no longer fill the void space and overall porosity increases. If smaller particles are added to the ‘just filled’ case then the larger particles move away from each other, or dilate, and over-all porosity increases (Fraser 1935).
Furnas (1928) was the first author noted by this review to describe the above "just filled" concept for binary mixtures. Furnas (1928) provided a method for predicting the possible compositions of a binary mixture with infinite particle size ratios. Figure 3.3 shows a plot of specific volume, developed by Furnas (1928), which plots the reciprocal of specific gravity (volume per unit weight) versus percentage of large constituent of a binary mixture by weight. The field of existence of a two-part, or binary mixture, is represented by the triangle defined by points ABC in Figure 3.3.

![Specific volume relations in a two-component system, after Furnas (1928).](image)

Line AB in Figure 3.3 is defined by equation [3.1],

\[
\text{Specific Volume} = \frac{y}{G_{\text{large}}(1 - n_{\text{large}})},
\]

where \(y\) is the weight percentage of the large constituent in the mixture, \(G_{\text{large}}\) is the specific gravity of the large component, and \(n_{\text{large}}\) is the porosity, or proportion of void content, of the large component (Furnas 1928).
Line CD in Figure 3.3 is defined by equation [3.2],

$$\text{Specific Volume} = \left[ \frac{(1 - y)}{((1 - n_{\text{small}})G_{\text{small}})} \right] + \left( \frac{y}{G_{\text{small}}} \right),$$

where $G_{\text{small}}$ is the specific gravity of the small constituent, and $n_{\text{small}}$ is the porosity, or proportion of void content, of the small component (Furnas 1928).

Point B in Figure 3.3, the intersection of lines AB and CD, represents the weight percentage of the large constituent where the mixture has a minimum specific volume or maximum density. Furnas (1928) stated that the particle saturation point B may be calculated from equation [3.3]:

$$y = \frac{((1 - n_{\text{large}})G_{\text{large}})}{((1 - n_{\text{large}})G_{\text{large}} + n_{\text{large}}(1 - n_{\text{small}})G_{\text{small}})}$$

In order to plot Figure 3.3, the values of $G_{\text{small}}$ and $G_{\text{large}}$ were arbitrarily assumed to be 2.7, and the values of $n_{\text{small}}$ and $n_{\text{large}}$ were arbitrarily assumed to be 0.5.

Furnas (1928) proved that the maximum density of a binary mixture occurs at point B - the mixture ratio where the smaller particles saturate or “just fill” the voids of the larger particles. Figure 3.3 and equations [3.1], [3.2], and [3.3], are all defined for the limiting case of a maximum particle size ratio. Maximum particle size ratio occurs when the small constituent particles are infinitely small, or the large component particles are infinitely large. Furnas (1928) gave the analogy of a binary mixture consisting of solid particles and water as an example of a near infinite particle size difference. Ignoring capillary, chemical effects, and absorption, the particles of water do not interfere with the
packing of the solid particles, and vice versa. In reality there will always be a disruption of the larger particles by the smaller ones, and vice versa. When particle size ratios become smaller, or the particles approach each other in mean diameter, then container wall effect becomes more dominant, the packings of the individual components are disrupted, and the porosity of the mixture increases.

Furnas (1928) provided a theoretical basis for demonstrating the effect of mixture ratio on binary mixtures with infinite particle size ratios but resorted to experimental results to demonstrate the effect of other particle size ratios. Furnas (1928) examined the effect of particle size ratio by comparing several different binary mixtures with different particle size ratios at a series of mixture ratios, the results of which are summarized in Figure 3.4. The lower boundary in Figure 3.4 is represented by an infinite particle size ratio that may be conceptualized as a mixture of solid particles and water, and may be calculated from equations [3.1], [3.2], and [3.3].
Figure 3.4 Experimentally determined porosity versus mixture ratio for binary mixtures, (voids of single components = 0.40, modified from Furnas (1928)).

Figure 3.4 illustrates the effects of both mixture ratio and particle size ratio. Mixtures of similar size particles, or small particle size ratio, have higher porosities than mixtures with different size particles, or large particle size ratios.

3.3.2.3 Density of Packing of Individual Components

In Figure 3.4 the “just filled” mixture ratio for an infinite particle size ratio is approximately 72% large particles by volume. The mixture ratio at the “just filled” point depends on the density of packing of the individual particle sizes. Figure 3.4 was created with the assumption that the different sizes had individual porosities of 0.4, or 40%. The porosity of individual components can be read from the vertical axes in Figure 3.4. Changing the porosity, or density of packing, of the individual components of a binary mixture directly changes the voids of the mixture and also shifts the “just filled” mixture
ratio. If the larger particle size had a lower porosity, then the “just filled” mixture ratio would be lower. If the smaller particle size had a lower porosity, then the “just filled” mixture ratio would be higher. The idea of “just filling” the voids of the large particles with smaller particles is fundamental to understanding the structure and behaviour of soil mixtures and of mixtures of waste rock and tailings.

It is noted that, for binary mixtures, the effects of particle size and shape on individual density are accounted for by the porosity of the individual components in a mixture. Once mixed, the individual porosity of each component will be increased by container wall effect, or the disruption of regular packing by the presence of the other particle size. The increase in porosity is greatest for similar size particles and smallest for infinite particle size ratios.

3.3.2.4 Quantitative Description of the Structure of Binary Mixtures

Fraser (1935) provided a qualitative description of the structure of a binary mixture with respect to the matrix and filler. Bodman and Constantin (1965) used the void ratios of individual components to provide a quantitative description of the structure of mixtures. Bodman and Constantin (1965) distinguished between “matrix” and “embedded” particles in a mixture based on the void ratios of each component (e.g. the two components of a binary mixture include large particles and small particles). In a homogenous mixture, a component at the minimum, or parent, void ratio was defined to compose a “matrix” with continuous contact between particles. A component at greater than the minimum, or parent, void ratio is not in continuous contact and was defined to be “embedded” within the “matrix” (Bodman and Constantin 1965). Similar definitions are
used to define the particle structure of mixtures of waste rock and tailings in Section 3.4. As mentioned above, the term 'structure' refers here to particle packing arrangements, rather soil fabric or anisotropy.

3.3.2.5 Extension of Binary Mixture Theory to Real Particles

While particle packing theory was developed using ideal spherical particles, practical experiments and applications always deal with real particles. Experimental models of even mono-sized spherical shapes are never perfect and have a particle size distribution, or a range of sizes. Real soils consist of a range of particle sizes and shapes, and the same is true of waste rock and of tailings. Calculations and models for ideal rigid spheres have been considered as a starting place in particle packing theory, useful as “… first approximation to systems with more realistic potentials of interaction…” (McLellan and Alder 1951). However, evidence in the literature indicates that particle packing theory for binary mixtures of ideal spheres can be applied to real particles with a distribution of sizes.

Figure 3.4 was constructed by Furnas (1928) using data from sets of real particles rather than ideal mono-sized spheres. Other researchers have extended particle packing theory to mixtures of two groups of particles with a mean difference in size. Powers (1964) used concepts related by Furnas (1928) as a basis for predicting the porosity of binary mixtures of concrete aggregates. Bodman and Constantin (1965) noted an agreement between observed and predicted porosity for mixtures of sand and beads with particle size ratios of 11 and 41. Bodman and Constantin (1965) showed that predicted minimum bulk volumes of mixtures of real soils were similar to observed values and concluded that
increases in bulk volume for real soil mixtures relative to predicted ones occurred due to small particle size ratios, frictional interactions between particles during mixing, and possibly air trapped in the mixtures. Bodman and Constantin (1965) also stated that binary mixture theory should apply to sets of real particles with continuous distributions, noting that deviation from minimum porosity occurs at smaller particle size ratios. Sohn and Moreland (1968) demonstrated that maxima of density occurred at mixture ratios of 1.2:1 and 3:1 (large particles by mass) for mixtures of two types of sand particles with Gaussian and log-normal distributions. Al-Jarallah and Tons (1981) presented a method for predicting void ratio of compacted binary mixtures for particle size ratios between 2.5 and 5 based on the work of Furnas (1928) and Powers (1964). Al-Jarallah and Tons (1981) demonstrated that two size, or binary, aggregate mixes that had the same particle size ratio, equal total packing volume, and equal compactive effort also had the same porosity. Yu and Standish (1987) found that the porosity of multi-component mixtures may be "...confidently predicted..." from the results of binary mixtures. Experimental evidence indicates that the simple theory of Furnas (1928) can be used to predict porosity of mixtures with large particle size ratios from the properties of the mixture components.

As with random packings of single size particles, studies of binary mixtures have focused on models of binary mixtures, e.g. glass spheres (Epstein and Young 1962), and Lucite spheres (Mangelsdorf and Washington 1960). Efforts have also been made to numerically simulate binary packings, including McLellan and Alder (1956), Smith and Lea (1960), Dodds (1975), Dodds and Kuno (1977), and Leitzelement et al. (1985). The details and development of these predictive models are often complex, involving multiple
coefficients that are not necessarily measurable. The reader is directed to the references for further details. The work of Furnas (1928) provides an elegant, or simple, approach useful for the design of mixtures of waste rock and tailings.

While ideal monosized particles form the basis for predicting porosity of mixtures of two sizes of particles, it has been shown from physical studies reported in the literature that the concepts of particle size ratio and mixture ratio also apply to mixtures composed of two groups of real particles which differ in mean particle size. Therefore, the same theory can be used for predicting the structure, or porosity of binary mixtures of waste rock and tailings. Increasing particle size ratio reduces the interference of smaller particles in the packing of larger particles. For mixtures of two groups of particles with particle size distributions, there will also be a specific mixture ratio where density of the mixture is a minimum. Particle packing theory for binary mixtures may therefore be applied to mixtures of waste rock and tailings with particle size distributions and a large difference in mean particle diameter. As mentioned above, Morris and Williams (2000b) used a similar approach to describe the structure of deposits of co-disposed coarse coal rejects and tailings.

3.3.3 Mixtures of Three or More Particle Sizes

One of the most common and important questions addressed by particle packing theory is how to design a mixture for minimum porosity, or maximum density. For example, the particle size distribution for concrete aggregates is typically selected to produce mixtures with a maximum density (Fuller and Thompson 1907). Maximum density mixtures have many advantages including a maximum efficiency in the use of volume, minimum
permeability, and maximum strength. This thesis uses particle packing theory for binary mixtures to address the same question for mixtures of waste rock and tailings. There are other methods for predicting the density of particulate mixtures with more than two sizes based on particle size distribution and arbitrary coefficients. Such methods were considered for use in this thesis, but not used. Treatments of multiple component mixtures are briefly discussed here to provide a context for the present development, and also for the reference of those readers who are interested in theoretical predictions of porosity.

Several methods have been developed to predict or design minimum porosity mixtures based on particle size distribution. As a reference point for discussions of minimum porosity, it was noted above that ordered packing of ideal spheres have a porosity near 26%. For mixtures of real particles of multiple particle sizes, the work by McGeary (1961) provides another reference point.

3.3.3.1 Mixtures of Discrete Sizes
In a fundamental study of ordered packing arrangements, McGeary (1961) vibrated multiple sizes of particles into a high-density orthorhombic arrangement. Largest particles were first vibrated into a container. Successively smaller particle sizes were then vibrated into the voids of the existing packing arrangement. Particle size differences of approximately 7:1 were required to allow smaller particles to move between the voids of larger particles, corresponding to the "critical ratio of entrance." The minimum porosity achieved for a packing of four sizes was found to be 4.9%, which was higher than the theoretical minimum of 2.5% (McGeary 1961) expressed packing structure or
porosity as a percentage of theoretical density defined as the density of a solid). The high
density arrangement could only be achieved by ordered addition of largest to smallest
components to a container during mechanical application of approximately sinusoidal
axial vibration. It was noted that attempts to pre-mix and subsequently place the same
combination of particle sizes resulted in segregation. Pre-mixing the same particles
produced arrangements with much lower densities than ordered packings created by the
method described above, even when subjected to the same mechanical vibration
(McGeary 1961). The porosity of 4.9% was the lowest found by this review. It was
noted that the mixture could be poured out of the container (McGeary 1961).

Controlled conditions that facilitate the construction of ordered packings (as per
McGeary 1961) are not typical. Randomly placed mixtures tend to have variable
microstructures dependent on bridging effects and seemingly random packing
arrangements. Ayer and Soppet (1965) investigated vibratory compaction of mixtures,
and developed empirical equations for predicting packing efficiency of multi-component
mixtures based on particle diameter ratios.

Most studies of mixtures involve non-ordered, or random packings. Several studies have
been conducted for mixtures of three or more particle sizes, and have resulted in semi-
empirical or analytical equations for predicting porosity and permeability of mixtures
(Furnas 1929, Standish and Borger 1979, Standish and Mellor 1980, Standish and Collins
1985). Westman and Hugill (1930), and later Bodman and Constantin (1965) presented
developments for predicting the structure of tertiary mixtures, or mixtures of three
particle sizes. Dodds (1975), and also Standish and Leyshon (1981) presented results of investigations of four part mixtures. Yu and Standish (1987) concluded that the porosity of multi-component mixtures can be predicted from the results of binary mixtures. Other theoretical approaches have been tried to predict the structure of mixtures with continuous particle size distributions.

3.3.3.2 Mixtures With Continuous Particle Size Distributions
Soils and aggregates encountered by civil engineers are typically composed of a range of particle sizes. Different approaches have been tried to predict or simulate porosity from particle size distribution and particle shape, with methods ranging from analytical/empirical methods, to semi-analytical methods that describe observed distributions with “ideal” distributions. Methods for the design and/or prediction of the structure of mixtures with multiple sizes examined for this thesis include Fuller and Thompson (1905), Peronius and Sweeting (1985), Stovall et al. (1986), Yu and Standish (1987, 1988, 1991, 1993), Aberg (1996a, 1996b), Tsirel (1997), Stroeven and Stroeven (1999), and Latham et al. (2002). Most of these works contain lengthy theoretical developments that fall outside the scope of this thesis.

Fuller and Thompson (1905) observed that certain particle size distributions produced high-density mixtures for concrete aggregates. Fuller and Thompson (1905) fitted equations to the gradation curves of the “ideal” distributions for use in the design of future mixtures, but noted that the curves were specific to a type of material. The approach of Fuller and Thompson (1905) has been used to design concrete mixtures for
many years. German (1989) provided a list of other types of “ideal” gradations derived from empirical testing.

Peronius and Sweeting (1985) provided a history of the development of methods for predicting porosity from particle size distribution. Peronius and Sweeting (1985) noted that porosity was difficult to accurately predict due to difficulty in comprehensively describing particle size distribution and in measuring particle shape. Predictive methods often involve coefficients specific to material types.

Latham et al. (2002) provided a general review of packing theory and the progress of numerical computer simulations for predicting porosity of particulate mixtures. Latham et al. (2002) concluded that better methods for characterizing particle shape irregularity are required.

3.3.4 Summary of Particle Packing Theory

This review of particle packing theory has revealed several principles that may be applied to mixtures of waste rock and tailings:

1. Geometrically systematic, ordered packings of spherical particles have higher density than random packings. Creation of ordered packings typically requires significant mechanical agitation, vibration, or labour intensive placement of individual particles. Random particle packing arrangements with higher porosities are more common than ordered ones, even for ideal spheres. Random arrangements are subject to bridging effects and may include non-continuous
portions of ordered packings. Mixtures of waste rock and tailings will form random arrangements.

2. Packings of irregular particle shapes tend to have greater porosity than packings of ideal spheres, all other factors held equal (e.g. compactive effort). The presence of flat container walls will disrupt the packing of particles, leaving a zone of lower porosity at container walls. In a similar manner, larger particles with flat surfaces will disrupt the packing of smaller particles, resulting in zones of lower porosity at the surface of larger particles.

3. Binary mixtures, or mixtures of two groups of single-sized particles with different diameters have lower porosity than arrangements of either size alone. Mixtures have lower porosities because the smaller particles may fill in the void space of the larger particles. The porosity of binary mixtures is governed by particle size ratio, mixture ratio, and by the density of individual components. Particle diameter must differ by approximately 7:1 to allow movement of smaller particles between the larger ones. As difference in particle size increases in a binary mixture, the smaller particles interfere less and less with the packing of larger particles. A maximum packing density occurs at a singular or unique mixture ratio where the smaller particles “just fill” the void space of the larger ones.

3.4 A Particle Model for Mixtures of Waste Rock and Tailings

What follows is the development of a particle model for mixtures of waste rock and fine tailings. The proposed model is analogous to similar models of other soil mixtures, but is based on particle packing concepts related above, and more directly on theory related by Furnas (1928), Westman and Hugill (1930), Fraser (1935), Bodman and Constantin.
(1965), Al Jarallah and Tons (1981), and also as Morris and Williams (2000b). The model provides a basis for quantitatively predicting mixture particle structure from design variables.

3.4.1 Basic Assumptions and Definitions

Basic assumptions for the conceptual particle model include:

1. Mixtures consist of waste rock particles, tailings particles, water, and air.
2. Waste rock, tailings particles, and water are incompressible.
3. Tailings slurry is composed of tailings solids particles and water.
4. Mixtures are homogeneous, meaning that tailings, waste rock, and water are evenly distributed through the mixture mass.
5. Mixtures containing air may be non-homogeneous.
6. The mass of air is ignored.
7. Waste rock and tailings have a large difference in average particle diameter (greater than 20), implying that tailings particles are physically much smaller than the "critical ratio of entrance," or pore throat diameter of openings between particles of waste rock.

On the basis of large differences in mean particle size, mixtures of waste rock and tailings are considered to be binary mixtures. As such, the particle structure of mixtures of waste rock and tailings are governed by the particle size ratio, mixture ratio, and the individual porosities of waste rock, and of tailings.

Possible mixture particle structure configurations are shown in Figure 3.5.
waste rock only

waste rock void space partly filled with tailings

waste rock void "just filled" with tailings

tailings with "floating" waste rock particles

tailings only

Figure 3.5 Mixture particle structure configurations.

The phase diagram for a mixture of waste rock and fine tailings is shown in Figure 3.6
Figure 3.6 Phase diagram for mixture of waste rock and tailings.

Referring to Figure 3.6, $V$ is total volume, $V_v$ is the volume of voids, $V_s$ is the volume of solids, $V_a$ is the volume of air, $V_w$ is the volume of water, $V_t$ is the volume of tailings solids, and $V_r$ is the volume of waste rock solids. Similarly, $M$ is total mass, $M_w$ is the mass of water, $M_s$ is the mass of solids, $M_t$ is the mass of tailings solids, $M_r$ is the mass of rock. The mass of water may be further divided into mass of water from the tailings, and mass of water from the waste rock. Terms used to quantitatively describe mixture particle structure in this thesis include void ratio, $e$; porosity, $n$; waste rock skeleton void ratio, $e_r$, and porosity, $n_r$, tailings matrix void ratio, $e_t$, and porosity, $n_t$, as described by [3.4], through [3.9], respectively.

[3.4] $e = V_v/V_s$

[3.5] $n = V_v/V$

[3.6] $e_r = (V_a + V_w + V_t) / V_r$

[3.7] $n_r = (V_a + V_w + V_t) / V$

[3.8] $e_t = V_w / V_t$

[3.9] $n_t = (V_a + V_w) / (V_a + V_w + V_t)$
Mixture-specific parameters for describing structure, such as $e_t$ and $e_r$ defined above, are directly related to geotechnical behaviours. If the value of $e_t$ is equal to the value of $e$ for the source tailings alone, then the tailings in the mixture form a continuous matrix. If the value of $e_r$ is equal to the value of $e$ for source-waste-rock alone, then the mixture has a structure where the waste rock particles are in continuous contact and form a continuous skeleton. If the value of $e_r$ is higher than the value of $e$ for waste rock alone, then the rock particles are separated by tailings particles, and do not form a continuous skeleton. The definitions are similar to those described by Bodman and Constantin (1965) for other mixtures.

The particle model described above allows prediction of mixture structure from design variables and therefore allows examination of the effect of mixture design variables on mixture structure. The effect of mixture design variables of particle size ratio, mixture ratio, particle size, shape, and distribution are related to mixture structure below.

3.4.2 Particle Size Ratio

Particle size ratios between mean diameters of waste rock and tailings particles are typically large, and support the assumption that tailings particles will not significantly interfere with the packing of waste rock particles. The absolute difference in particle size between waste rock and tailings is much greater than the critical ratio of entrance of 7:1 for perfect packing of spheres. For example, the mean particle diameter ratio for waste rock to tailings is near 1000, similar to gravel and silt. Morris and Williams (2000b) noted that particle size ratio ranged from 43 to about 1950 for pumped co-disposal deposits of coarse coal reject and tailings. Tailings are significantly smaller than the
critical ratio of entrance for waste rock and may therefore move within the void space of the waste rock.

Interference of the packing of waste rock particles by tailings particles may occur but the effect is expected to be small. Two factors make interference of packing of waste rock particles by individual tailings particles insignificant. The first is that contacts between angular rock particles are small in area. It is unlikely that small tailings particles will interfere significantly with point to rock contacts. Second, if mechanical effort is used for mixing, then the tailings will tend to move to the void space of the waste rock, rather than remain between point-to-rock contacts of the larger waste rock particles. It is therefore assumed that, due to particle size ratio, the tailings do not interfere significantly with the packing of waste rock particles. Morris and Williams (2000b) are acknowledged to have previously used this simplifying assumption to determine the porosity of co-disposed coal washery wastes. The assumption is also supported by previous research (see Section 3.3.2.5). The effect of particle size ratio should not be confused with the effect of mixture ratio, where the numerous small particles may interfere with the packing of larger particles.

3.4.3 Mixture Ratio
The description of effect of mixture ratio on particle structure is fundamental enough to be repeated in relation to mixtures of waste rock and tailings, described in Table 3.2, and illustrated in Figures 3.7, 3.8, and 3.9. The effect of mixture ratio on structure of binary mixtures was previously demonstrated by Furnas (1928), Westman and Hugill (1930), and Bodman and Constantin (1965); qualitatively described by Fraser (1935); confirmed
by Powers (1964) and Al-Jarallah and Tons (1981); and independently re-discovered by Morris and Williams (2000b), and by Vallejo (2001). Mixture ratio is at a maximum limit for waste rock alone. Mixture ratio decreases with the addition of tailings slurry to the waste rock. It is assumed that the added tailings slurry is accommodated in the void space of the waste rock and that no change in total volume of the mixture occurs at higher mixture ratios. Once the volume of slurry added is equal to the volume of voids of the waste rock, the voids are "just filled." The "just filled" case represents a maximum of initial density and is considered a design optimum. Further addition of tailings to the "just filled" case causes the waste rock particles to separate in order to accommodate the tailings slurry, and the volume of the mixture will increase linearly with the volume of slurry added. Once separated, the waste rock particles are considered to be isolated, or "floating" within a tailings matrix. With further addition of tailings slurry, or removal of waste rock particles, the mixture ratio decreases to a minimum where the mixture consists of all tailings with no waste rock. Possible structural configurations are listed in Table 3.2, and illustrated in Figure 3.5.

Table 3.2 Particle structure and continuity in a mixture.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Waste Rock Skeleton</th>
<th>Tailings Slurry Matrix</th>
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</thead>
<tbody>
<tr>
<td>waste rock alone</td>
<td>continuous</td>
<td>none</td>
</tr>
<tr>
<td>waste rock voids partly filled with tailings slurry</td>
<td>continuous</td>
<td>non-continuous</td>
</tr>
<tr>
<td>the &quot;just filled&quot; case</td>
<td>continuous</td>
<td>continuous</td>
</tr>
<tr>
<td>waste rock particles floating in tailings slurry</td>
<td>non-continuous</td>
<td>continuous</td>
</tr>
<tr>
<td>tailings slurry alone</td>
<td>none</td>
<td>continuous</td>
</tr>
</tbody>
</table>
Phase diagrams provide another illustration of the effect of mixture ratio on structure. The properties of waste rock and tailings used in laboratory investigations described in Chapters Four and Five were used to calculate the mass and volume proportions for mixtures with a range of mixture ratios. Mass proportions were calculated based on the assumptions given in Section 3.4.1, and are illustrated in Figure 3.7 with the use of Equation [3.10]:

\[ 3.10 \text{ Mass total} = M_w + M_r + M_t \]

Material properties used to produce Figure 3.7 are taken from materials investigated in Chapters Four, Five, and Six and include an initial waste rock water content of 0.1%, and an initial tailings solids content of 48.4% (mass solids/mass total), or gravimetric water content of 107% (mass water/mass solids).
Figure 3.7 Predicted mass proportions versus mixture ratio.

Figure 3.7 demonstrates that tailings slurry contributes most of the water in the mixture. By assuming densities and initial void ratios for the same waste rock and tailings, the mass proportions shown in Figure 3.7 may be converted to volume proportions using Equations [3.11] and [3.12]

\[ [3.11] \text{density} = \frac{M}{V}, \text{ and} \]
\[ [3.12] \quad V = V_a + V_r + V_t + V_w \]

Volume proportions are shown in Figure 3.8. Properties used to produce Figure 3.8 include an initial void ratio for waste rock of 0.71, with densities of 2.7 g/cm$^3$ for waste rock solids, 2.89 g/cm$^3$ for tailings solids, and 1 g/cm$^3$ for water.
In order to construct Figure 3.8, it was assumed that the void ratio of the waste rock does not change with the addition of tailings until the volume of tailings slurry exceeds the initial volume of voids in the waste rock. Figure 3.8 indicates that mixtures with mixture ratios greater than approximately 5:1 will contain air because the volume of tailings slurry is less than the volume of voids in the waste rock. At mixture ratios less than 5:1, the volume of tailings slurry is greater than the volume of voids in the waste rock skeleton. Figure 3.8 does not necessarily define what is physically possible for homogeneous mixtures, but does provide an illustration of how the different phases may fit together. The data from Figures 3.7 and 3.8 can be used to derive simple geotechnical properties such as water content, dry density, and degree of saturation, using [3.13], [3.14], and [3.15]:

\[ 3.13 \text{ gravimetric water content } = 100\% \cdot \frac{M_w}{M_r + M_t} \]
[3.14] dry density = \((M_r + M_t)/(V_r + V_t + V_a + V_w)\)

[3.15] degree of saturation = \(100\% \times V_w/(V_a + V_w)\).

Predictions of simple density related properties are shown in Figure 3.9.

The most striking relation shown in Figure 3.8 is that a maximum of dry density occurs where the void space of the waste rock is "just filled" with tailings slurry, at a mixture ratio of approximately 5:1 (waste rock to tailings by dry mass). At mixture ratios less than 5:1 the density of the mixture is reduced by the replacement of rock solids with tailings slurry that has a much lower density due to high water content. At mixture ratios greater than 5:1 the density of the mixture is reduced by the introduction of air into the void space of the waste rock. The degree of saturation is 100% at mixture ratios where the volume of tailings exceeds the volume of the waste rock pore space, and less than 100% where the volume of tailings is less than the void space of the waste rock. Figure 3.9 Predicted saturation, density, and water content versus mixture ratio.
3.8 also indicates that water content increases with a decrease in the ratio of waste rock to tailings, or more simply, water content increases with tailings content. Figures 3.7, 3.8, and 3.9 are derived from arbitrary material properties specific to one type of tailings and one type of waste rock. It follows that the “just filled” mixture ratio will be different for different materials and that similar figures could be constructed with other material properties.

3.4.4 Individual Porosity of Waste Rock and Tailings

Figures 3.7, 3.8, and 3.9 are predictions of the initial, unconsolidated, condition of mixtures. The porosity or void ratios of waste rock and tailings used to construct Figures 3.7, 3.8 and 3.9 are based on values measured for two real materials, and could be changed to describe other materials. The initial porosity of each material accounts for differences in packing arrangement due to absolute particle size, shape, and size distribution. More importantly to tailings, initial packing arrangement and porosity depends on solids/water content. De-watering the tailings could reduce the initial porosity of the source tailings slurry. Altering initial porosities of source materials will change the predicted phase relations shown in Figures 3.7, 3.8, and 3.9. If the waste rock has a lower initial porosity, then the “just filled” point will shift to a higher mixture ratio. If the tailings have a lower initial porosity, then the “just filled” point will shift to a lower mixture ratio. Consolidation of mixtures after blending will also change mixture properties and will presumably lower water content, increase density, and may increase degree of saturation in some cases. Observations on the effect of consolidation and tailings solids content are related in the experimental program described in Chapters Four and Five.
3.4.5 Summary

The particle model defined in Section 3.4 provides a basis for design of mixtures, and for describing the structure of mixtures of waste rock and tailings. It has been demonstrated that application of particle packing theory for binary mixtures allows selection of mixture ratio to produce a mixture with maximum density, where the voids in the waste rock are “just filled” with tailings slurry. The model introduced in Section 3.4 allows structure to be defined in terms of the individual void ratios of waste rock and tailings. The model thereby allows a quantitative description of whether the waste rock particles form a continuous “waste rock skeleton,” and whether or not the tailings form a continuous “tailings matrix.” The knowledge of and the ability to predict particle structure are important because structure directly affects geotechnical behaviour, as indicated by a review of literature on other types of soil mixtures.

3.5 Studies of Soil Mixture Geotechnical Behaviours

The literature was reviewed to determine how soil particle structure influences the geotechnical behaviour of particulate mixtures. Little or no work has been done to relate structure to behaviour for mixtures of waste rock and tailings. Consequently, studies of analogous soil mixtures were reviewed for correlations between mixture structure and behaviour.

The primary findings of the review indicate that mixture behaviour is dominated, or entirely dependent on particle structure. Soil mechanics has typically relied on qualitative descriptions and understanding of soil macro structure in terms of fabric, anisotropy, etc., e.g. (Mitchell 1993). However, binary mixture theory provides a basis
for quantitatively describing another level of structure with respect to the continuity of large and small particle matrices. Studies of soil mixtures that were reviewed are listed in Table 3.3, along with mixture materials and the primary subjects of each study. German's (1989) statement about researchers in different fields re-inventing the same basic principles for particle packing theory holds true within soil mechanics. No general understanding of the relationship between soil mixture behaviour and structure exists. Some researchers have treated soil structure as a black box, providing experimental evidence that structure controls behaviour, but without any theoretical explanations. Other researchers have provided particle models to explain unique behaviours. The findings of the review are summarized here, and indicate that most, if not all, geotechnical behaviours may be quantitatively related to mixture particle structure.

Key areas of research of soil mixtures included compaction and packing, permeability, volume change, and shear strength. The relationships between each subject area and mixture particle structure are summarized in Section 3.5.4.
Table 3.3 Studies of soil mixtures.

<table>
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<th>Volume Change</th>
<th>Strength</th>
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### 3.5.1 Compaction and Packing

The relation between compaction, packing and particle structure has been examined with experimental and theoretical approaches. Gordon et al. (1964) examined the effect of rock content on the compaction of unsaturated clay-gravel mixtures. Compaction curves were found to have sharp peaks, indicating sensitivity to water content. Specimen densities were also noted to increase with rock content (Gordon et al. 1964). The
approach of Gordon et al. (1964) was experimental, with few correlations made between behaviour and structure.

Other studies have involved more complex theoretical developments to predict or describe particle structure. Studies of soil particle packing arrangements often refer to particle packing theory. Lees (1970) described soil void size distribution rather than particle size distribution for the design of mixture gradations. Aberg (1992) presented a stochastic model of void structure and void size of compacted non-plastic soils. Yang and Lo (2001) presented a statistical approach to describe the structure of gravely cobble deposits. Soils were typically described as continuous distributions rather than binary mixtures.

Coughlan et al. (1978) examined mixtures of swelling clay and sand with respect to binary mixture theory. Coughlan et al. (1978) described mixture particle structure using the void ratios of coarse particles and fine particles. With increasing clay content, mixture porosity increased up to 10% clay content, then decreased up to 40% clay, and finally increased for one specimen, but stayed constant for another. The finding was attributed to clay entering between sand junctions and increasing porosity by expanding the coarse particle matrix. From water retention and swelling data, transition from coarse to fine particle matrix occurred at about 40% clay content for both specimens. Review of other studies indicated that particle packing theory for binary mixtures could be applied to clay – soil mixtures in a semi-quantitative fashion (Coughlan 1978).
Winter et al. (1998) examined the effect of large particles on compaction for earthworks. Transition from behaviour dominated by the smaller particle matrix to behaviour dominated by stones in a mixture occurred at approximately 45% to 50% stone content. Specimens with stone contents between approximately 35% and 50% had an increase in maximum dry density and a decrease in optimum water content for constant air voids of between 0% and 5%. Transition stone contents of some 30 datasets from the literature were found to agree with the 45% to 50% transitional case for dominance in compaction behaviour (Winter et al. 1998).

3.5.2 Strength
Studies examining the strength of soil mixtures are grouped here by categories including shear strength, scalping effects, triaxial behaviour, and liquefaction. A strong correlation between binary mixture structure and behaviour has been made for the strength of soil mixtures.

3.5.2.1 Shear Strength
The influence of mixture composition on shear strength is some of the first documented evidence for a relationship between structure and behaviour. Holtz and Ellis (1961) presented results of large scale triaxial testing of clay-gravel soils. Data indicated that shear strength for clay gravel mixtures increased at gravel contents above 50%, while shear strength of sand gravel mixtures increased at gravel contents above 20%. Mixture structure was described in terms of gravel content, but was not related to particle structure (Holtz and Ellis 1961).
Particle models and particle packing theory have been used to relate shear strength behaviour to soil structure. Statham (1974) found that angle of repose of mixtures of glass beads was dependent on mixture proportions and reached a peak value with minimum porosity. Oda (1977) presented a technique to use coordination number (the number of contacts each sphere in a mixture has with other spheres in the mixture) to determine angle of internal friction. Vallejo (2001) studied the shear strength of glass beads and claimed to have provided the first explanation of why shear strength behaviour of binary mixtures of glass beads is dependent on mixture ratio. When two bead sizes with a particle diameter ratio of 12.5 were combined at mixture ratios greater than 2.3 (large to small beads by mass) then the shear strength of the mixture was controlled by the frictional resistance of the larger beads. At mixture ratios below 1.5 the shear strength of the mixture was controlled by the frictional resistance of the smaller beads. Shear strength of mixtures with mixture ratios between 1.5 and 2.3 was partly controlled by the larger beads. Vallejo (2001) explained the relationship between mixture ratio and shear strength in terms of particle structure. With decrease in mixture ratio the structure of the mixture changes from one which is large particle dominated, with smaller particles simply occupying void space between load-bearing large particles, to one which is small particle dominated, with large particles isolated, or “floating” in a matrix of load bearing smaller particles. Vallejo (2001) described mixture particle structure quantitatively in terms of the porosity of coarse and fine mixture components, similar to definitions provided for waste rock and tailings in Section 3.4.
Lupini et al. (1981) presented the most significant study of the effect of soil structure on shearing behaviour considered by this review. Lupini et al. (1981) examined the effect of mixture ratio and particle structure on mechanisms of residual shear for mixtures of sand and London clay, sand and bentonite, and sand and mica. Lupini et al. (1981) defined a granular void ratio analogous to $e_r$ defined in Section 3.4 and proposed a conceptual particle-packing model based on ideal spheres to explain residual shear behaviour. Lupini et al. (1981) concluded that type of shearing mode and magnitude of residual strength were directly correlated with granular void ratio.

In a related study, Tan et al. (1994) found that shear strength of clay-sand mixtures increased when the void ratio of the sand in the mixture was less than 5 (porosity of 83%), caused by "geometrical proximity of the sand particles." In other words, the sand particles contributed to shear strength when in contact with other sand particles. The liquid limit of clay-sand mixtures appeared to follow a linear mixture law for sand contents lower than 60% (Tan et al. 1994).

The above studies have indicated that the shear strength of soil mixtures is a function of whether or not large particles are in contact with other large particles. The concept of a granular void ratio, similar to $e_r$ defined in Section 3.4, provides a measure of packing of the larger particles in the mixture that can be directly related to shear strength.

3.5.2.2 Scalping Effects
A series of studies investigated the effect of scalping larger particles from soil samples to allow testing in standard geotechnical equipment for soils. Standard soil triaxial testing
equipment typically cannot handle 50 mm size particles. Samples with larger particle sizes must be treated differently than samples with smaller sizes. Donaghe and Townsend (1976) found that scalping larger particles and replacing them with an equal mass of smaller particles worked well when gravel contents were larger than 70%, while applying corrections to results from scalped samples worked better for gravel contents up to 70%. Siddiqi (1984) demonstrated that the strength of samples with larger particles that were not touching, but “floating in a matrix of fine material,” were dependent on properties of the matrix material. Siddiqi (1984) stated that soils with up to 40-45% oversize particles (by mass) constitute the floating case, soils with from 40% to 60% oversize particles constitute a transitional case, and soils with greater than 60% oversize particles have a load-bearing large particle skeleton. Donaghe and Torrey (1985) found that neither scalp-and-replace nor scalping-only procedures were satisfactory for determining undrained strength for samples with oversize particles.

Fragaszy et al. (1990) introduced a “far field matrix density model” to describe the density of soils containing large particles isolated within a matrix of finer soil. The model implies that the density of packing of finer matrix particles is affected near larger particles, relative to the packing of finer particles far from larger particles. Fragaszy et al. (1990) briefly discussed ideal particle packing, real packing, reasons for density reduction, and presented a method to quantify reduction of density due to disruption of packing of smaller particles near the surface of larger ones. Siddiqi and Fragaszy (1991) introduced a “matrix method” for testing cyclic strength of “floating case” triaxial specimens, and defined the maximum size of a test specimen as six times $D_{60}$ when
samples are scalped of sizes greater than $D_{60}$. Fragaszy et al. (1992) confirmed the matrix method presented by Siddiqi and Fragaszy (1991) with consolidated drained triaxial testing of mixtures of sand and sub-rounded gravel specimens of the floating case.

As a basis for the far field matrix density model, Fragaszy et al. (1992) stated “When large particles are added to a matrix of finer soil, the additional volume is greater than the summation of the individual volumes of the added particles alone.” This review notes that Fraser (1935) previously documented the phenomenon. Fraser (1935) presented measurements of the porosity of sand indicating that porosity is greater near pebbles than away from them and stated: “This entire behaviour is, in reality, a type of manifestation of container effect.”

Evans and Zhou (1994,1995) described the cyclic strength of mixtures of sand and gravel in terms of a gravel skeleton with a sand matrix. Maximum density of mixtures of sand and gravel were found to occur at 60% gravel content (by mass). Evans and Zhou (1994) indicated that gravel was considered “to float in a sand matrix” at gravel contents less than 40%. Evans and Zhou (1995) demonstrated that inclusion of gravel particles can significantly increase the cyclic loading resistance of gap-graded mixtures of sand and gravel. Sasitharan et al. (1998) used scalping methods suggested by Fragaszy et al. (1992) and Evans and Zhou (1995) to model the strength of gravely fills.

The findings of a series of studies of the effect of scalping larger particles from a soil indicated that particle structure controls behaviour. If the larger particles in a soil
mixture do not initially form a load bearing skeleton then the mechanical properties of the mixture are similar to those of the finer material in the mixture located away from the larger particles. The finding validates the approach of testing samples scalped of larger sizes at densities similar to the matrix material in soil mixtures.

3.5.2.3 Other Triaxial Behaviours

Graham et al. (1989) tested sand-bentonite mixtures for swelling pressure, uniaxial compression, triaxial compression and shear strength. Graham et al. (1989) presented triaxial testing results in light of a conceptual model based on critical state soil mechanics and particle packing theory. Results were presented in $p' - q - v_c$ space, where $v_c$ is clay component specific volume (defined by Equation [3.17] below) and $p'$ and $q$ are triaxial stress parameters. Swelling pressure was also correlated with $v_c$ (Graham et al. 1989).

Wan et al. (1990) investigated the influence of soil structure on the behaviour of compacted triaxial specimens of 1:1 sand-bentonite mixtures and concluded that method of preparation influences ‘soil macrostructure,’ which in turn influences stress-strain characteristics of the mixture. Georgiannou et al. (1990, 1991a, 1991b) related granular void ratio (similar to $e_r$), and clay content to the triaxial compression and extension of mixtures of silt, sand and kaolin clay.

3.5.2.4 Liquefaction

Kuerbis et al. (1988) introduced a particle model consisting of a sand skeleton with silt filler to explain why increasing silt content up to 20% did not significantly change the liquefaction response of sand-silt mixtures under monotonic and cyclic undrained
loading. Simply put, the silt particles were found to passively occupy the void space of sand particles at higher mixture ratios. Pitman et al. (1994) found that undrained brittleness of sands containing fine material may be controlled by the amount of fines, rather than whether the fines present were plastic or non-plastic. Structural stability of sands was found to increase with fines content, and 20% fines appeared to mark a transition between sand-to-sand contact and silt-dominated behaviour (Pitman et al. 1994).

Koester (1994) performed 500 undrained cyclic triaxial tests on mixtures of sand, silt, and plastic clay to determine the influence of the fines fraction on liquefaction resistance and pore pressure generation characteristics. Lowest liquefaction resistance for specimens with sand components at 50% relative density occurred at fines contents between 20% and 26%. Cyclic strength was reduced with addition of fines up to 24% to 30% fines content, and cyclic strength increased with further addition of fines, with fines fraction dominating cyclic loading response. Residual strengths of soils with 20% low plasticity fines were low, with essentially unlimited deformation potential. The study concluded that cyclic strength may not be characterized on the basis of gradation alone and that sand mixtures containing fines up to about 24% may be inherently collapsible due to relative compressibility of finer soil between sand grains (Koester 1994). Singh (1994) had similar findings in a study of silt and sand mixtures, where sands at 50% relative density and with 10%, 20% and 30% silt contents had lower liquefaction resistance.
Thevanayagam et al. (1996) noted that case histories of liquefaction of silty sands typically involved sands with high fines contents. Thevanayagam (1998) demonstrated that fines content, sand skeleton void ratio (analogous to \( e_r \) defined in Section 3.4), and fines matrix void ratio (analogous to \( e_t \) defined in Section 3.4) all play important roles in determining the undrained shear strength of silty sands. Thevanayagam (1998) noted that the transition between sand- to silt-like shear behaviour occurs at a relative density of the sand skeleton that increases with fines content. Thevanayagam and Mohan (2000) concluded that sand skeleton void ratio and fines matrix void ratio are state variables that control the stress-strain behaviour of silty sands.

Thevanayagam et al. (2000) argued that as traditionally defined, void ratio \( e \), and Relative Density \( D_r \), is not valid for representing behaviour of soil mixtures with different fines contents. Thevanayagam et al. (2000), Thevanayagam (2000), and also Day and Thevanayagam (1999) presented particle packing models for mixtures of silts and sands, relations for calculating charts of void ratio versus fines content for mixtures, and relations for determining skeletal and matrix void ratios. Thevanayagam et al. (2002) also encouraged the use of skeletal and matrix void ratios (analogous to the \( e_r \) and \( e_t \)) to characterize stress-strain response of gap graded granular mixtures. Thevanayagam (2000), Thevanayagam et al. (2000) and Thevanayagam et al. (2002) also presented equivalent void ratios for the description of structures that are transitional to sand dominated and silt dominated ones.
Lade and Yamamuro (1997) introduced a model for static liquefaction behaviour of mixtures of silt and sand wherein application of normal or shear stress causes silt particles to move into the voids of the sand skeleton, resulting in contraction and liquefaction. Yamamuro and Lade (1997) examined static liquefaction of loose sands under monotonic loading and hypothesized that the presence of fines can create highly compressible particle structures. Yamamuro and Lade (1998) stated that liquefaction of mixtures of silt and sand depends on void ratio, fines content, and confining stress. Lade et al. (1998) presented a fundamental review of particle packing theory and its application to the porosity of mixtures of sand and non-plastic fines. Lade et al. (1998) concluded that fines content plays an important role in determining structure, which will influence the compressibility and static liquefaction potential of a sand deposit. Naeini and Baziar (2004) used the concepts of particle packing put forth by Lade et al. (1998) and Yamamuro and Lade (1998) and found that increasing silt content to 35% in sand-silt mixtures results in a decrease in both peak and residual strengths.

Baziar and Dobry (1995) related standard penetration index to vertical effective stress for liquefaction potential of silty sands with a minimum 10% fines content. Amini and Sama (1999) and Amini and Qi (2000) stated that liquefaction resistance of non-plastic soils increased with silt content, but was not significantly influenced by layering. Chien et al. (2002), and Chien and Oh (2002) examined liquefaction resistance and shear strength for sand-silt mixtures considering the effects of Relative Density, void ratio, and fines content.
Studies of liquefaction of mixtures of silt and sand have produced the most detailed particle models for soil mixture structure. Mechanistic explanations of behaviour use particle packing models and theory for binary mixtures to relate changes in mixture structure to liquefaction behaviour. Soil structure was most commonly described in terms of large and small particle void ratios, which are similar to $e_r$ and $e_l$ defined in Section 3.4.

3.5.3 Volume Change and Hydraulic Conductivity

Studies of volume change and hydraulic conductivity have related index properties to compressibility, while other studies have related compressibility to mixture particle structure.

The first group of studies considered here focused on index properties. Srinivasa Murthy et al. (1987) concluded that coarse particles reduce the liquid limit of fine soils in a linear fashion, and that compressibility of such soils may be predicted using a modified Liquid Limit. Pandian et al. (1995) presented a method to predict permeability and compressibility of mixtures of sand and bentonite based on Liquid Limit. Pandian et al. (1995) also stated that clay particles form a coating on sand particles that prevents sand-particle-to-sand-particle contact.

The second group of studies related behaviour to particle structure. Fukue et al. (1985) presented results of consolidation testing of sand-bentonite mixtures at five mixture ratios. Mixture structure was conceptualized using a four phase structural model made up of clay, sand, water and air, and described by clay content, a sand void ratio analogous to
and a clay void ratio analogous to $e_t$. Fukue et al. (1985) noted a threshold value of sand void ratio below which the sand particles dominate compressibility behaviour of sand bentonite mixtures. The threshold value of sand void ratio was slightly higher than the maximum void ratio of the sand without clay present, near 1.4 and 1.25 (or porosities of 58.3% and 55.6%, respectively), depending on mixture ratio. Fukue et al. (1985) noted that Seed et al. (1964) had previously assumed that the threshold value of sand void ratio was near 0.8 (or 44% porosity) for behavioural dominance. Because frictional behaviour was found to be dependent on non-clay void ratio and clay content, Fukue et al. (1985) proposed a new classification system for clayey and sandy soils based on mechanical properties where the threshold sand-skeleton void ratio, dry density, and clay content, are used rather than index of plasticity and clay content alone.

Rattay and Gunther (1989) conducted oedometer tests on mixtures of clay and gravel and related constrained modulus to fines content and phase composition.

Wagg and Konrad (1990) related mixture structure to hydraulic conductivity, consolidation, and plasticity for mixtures of silt and kaolin clay. Mixtures with less than 30% clay had small changes in void ratio for large changes in hydraulic conductivity inferred from oedometer tests. Wagg and Konrad (1990) proposed a model for the structure of clay-silt mixtures including two pore sizes to explain permeability behaviour. The soil skeleton became clay dominated at clay contents above 30%, where the volume of clay exceeded the volume of large pores in the silt. Consolidation behaviour appeared to be dependent on the structure of the silt particle matrix (Wagg and Konrad 1990).
Mollins et al. (1996) related swelling pressure to the bentonite content of sand-bentonite mixtures and presented a method to predict hydraulic conductivity of mixtures based on a three phase mixture model of clay, sand, and water. Parameters used to describe the structure of sand-bentonite mixtures included bentonite content, clay void ratio and sand porosity (Mollins et al. 1996). The method presented by Mollins et al. (1996) is followed by this thesis for mixtures of waste rock and tailings and is further discussed in Chapter Seven.

Kumar and Muir Wood (1997) found that the permeability, compressibility, shear strength, and pore pressure response of mixtures of sand and kaolin clay were dependent on clay content, and changed below clay contents of 40%. Kumar and Muir Wood (1997) described mixture structure in terms of the value of a granular void ratio (as proposed by Lupini et al. (1981) and analogous to $e_r$ defined in Section 3.4) and also by granular specific volume, $v_g$, defined by [3.16]; clay specific volume, $v_c$, defined by equation [3.17], and also clay water content and clay void ratio (which is analogous to $e_t$ for a saturated mixture defined by [3.8]).

\[ v_g = \frac{V_g + V_w + V_c}{V_g} \]  
\[ v_c = \frac{V_w + V_c}{V_c} \]

In [3.16] and [3.17] $V_c$ is the volume of clay, and $V_g$ is volume of sand in the mixture. $V_c$ and $V_g$ are equivalent to $V_I$ and $V_F$ in terms of the phase diagram in Figure 3.6, and volume of air is assumed to be zero. Kumar and Muir Wood (1999) described the same effect of mixture structure on the mechanical behaviour of mixtures of clay and gravel,
using structural descriptors of granular content, clay matrix void ratio, clay specific volume, and gravel specific volume.

Studds et al. (1998) presented a model and design method for predicting void ratio versus stress relationships for mixtures of sand and bentonite. By assuming a series of mixture void ratios, it is possible to predict a void ratio versus pressure relationship that is similar to laboratory testing results. The stress required to achieve a mixture void ratio is taken as the sum of the stresses required to achieve equivalent clay void ratios and sand porosities in the parent mixture materials (Studds et al. 1998). The method is applied to mixtures of waste rock and tailings in Chapter Seven. Studds et al. (1998) also predicted the hydraulic conductivity of mixtures from mixture structure using the method proposed by Mollins et al. (1996) and developed by Porter et al. (1960).

Huang et al. (1999) found that sands with 15% silt content were much more compressible than clean sands in triaxial tests, and described structure in terms of conventionally defined void ratio and relative density.

Martins et al. (2001) presented results of oedometer and $K_O$-triaxial tests on a clayey sand, and described soil structure by conventional definitions of void ratio and specific volume. Borgesson et al. (2003) found that structural heterogeneity had a "very strong" influence on the hydromechanical properties of mixtures of bentonite and crushed rock. Borgesson et al. (2003) described mixture structure in terms of effective clay void ratio, ideal clay void ratio, ballast void ratio, ballast porosity, and also bentonite content.
Côté and Konrad (2003) studied effect of fines content in base coarse materials on hydraulic conductivity and soil-water characteristic curve. Côté and Konrad (2003) presented a porosity model including a coarse-grained skeleton with a uniformly distributed fine-grained matrix that relates overall porosity and fines content to fines porosity (fines porosity is analogous to $n_t$ defined in Section 3.4). Côté and Konrad (2003) presented relationships between fines porosity and air entry value, fines porosity and pore-size distribution index, fines porosity and saturated hydraulic conductivity and specific surface area of the fines present. Base coarse materials tested had fines contents between 3% and 12.5%. The $D_{50}$ of base coarse tested ranged between 3.1 mm and 8.1 mm, while the maximum size of fines materials was 80 μm. Although Côté and Konrad (2003) did not discuss effects of particle size ratio, the particle size ratio of $D_{50}$ of base coarse materials to $D_{100}$ of fines materials is calculated to have varied between 39 and 100.

Stewart et al. (2003) used the method presented by Studds et al. (1998) to predict swelling and the method presented by Mollins et al. (1996) to predict hydraulic conductivity of mixtures of sand with 10% and 20% bentonite. Stewart et al. (2003) characterized swelling behaviour of sand bentonite to include two distinct behaviours divided by a threshold stress. The threshold stress was stated to correspond to the transition from sand grains “floating” within the bentonite matrix, to sand grains coming in contact with other sand grains and forming a load bearing skeleton (Stewart et al.
Stewart et al. (2003) described mixture structure in terms of bentonite void ratio and sand void ratio.

The literature reviewed here has provided evidence and theory for quantitatively relating compressibility and hydraulic conductivity of soil mixtures to mixture structure. Compressibility of soil mixtures is a function of the compressibility of the parent materials and the mixture ratio. Hydraulic conductivity is a function of the small particle void ratio, similar to $e_t$ defined in Section 3.4.

3.5.4 Summary and Discussion

While little work has been done to relate particle structure to the behaviour of mixtures of waste rock and tailings, a review of the literature indicates that soil particle structure has been related to the behaviour of other soil mixtures. This review found that soil mixture structure has been related to geotechnical behaviours including cyclic and static liquefaction resistance, shear strength, hydraulic conductivity, and compressibility. Mixtures examined typically consisted of two types of soils, and were often described in terms of the individual porosities of each soil type. For example, the sand particle porosity and clay void ratio were used to describe the density of individual components in sand-clay mixtures (Studds et al. 1998). Mixture-specific structural descriptors were stated to be better indicators of behaviour than traditional structural descriptors such as porosity, void ratio, and Relative Density. Key findings indicate that the small particle void ratio is significant to mixture hydraulic conductivity, and the large particle void ratio is significant to the mechanical properties of shear strength, compressibility, and liquefaction.
Typically, mixture behaviour was found to be strongly influenced by mixture ratio defined by clay content, gravel content, etc. While some investigators merely noted a correlation between composition and behaviour (e.g. Gordon et al. 1964), some investigators presented structural particle models to provide mechanistic explanations of behaviour (e.g. Lade and Yamamuro 1997). Even further, some investigators quantified mixture structure in order to predict mixture behaviours (e.g. Studds et al. 1998).

While individual models of soil particle structure have been presented as a basis for explaining and predicting the behaviour of soil mixtures (e.g. Fukue et al. 1986, Wagg and Konrad 1990), references to existing particle-packing theory are rare. Aside from Lade et al. (1998), and investigations of void ratios and porosity of sands (e.g. Kolbuszewski 1948, Lees 1970), few investigators appear to be aware of the relationship between soil mixture structure and particle packing theory. The fact that a number of researchers investigating diverse properties and materials have independently re-discovered basic properties of particle packing theory supports the idea that particle packing theory can provide a fundamental basis for understanding soil mixture structure as it relates to geotechnical behaviour.

Soils are made up of particles and as such are well represented by the concepts of particle packing theory. The irregular particle shapes and arrangements of soil particles decrease packing efficiency and increase porosity compared to idealized spheres. The random nature of soil particle packing can cause a variation in porosity for different arrangements
of the same particles. Consequently, the porosities of soils are often described as a range
delineated by maximum and minimum values. Non-cohesive soils are better represented
by particle packing theory than cohesive ones. With smaller particles, and for clay
minerals in particular, physicochemical effects result in behaviours such as cohesion and
swelling that fall outside particle packing theory. However, particle models of soil
mixtures containing cohesive soils have been successfully used to explain and predict
behaviour, e.g. Studds et al. (1998). Most studies have involved mixtures of two soil
types and are consequently subject to binary mixture particle packing concepts of size
ratio and mixture ratio.

Naturally occurring soils, soil mixtures, and mixtures of waste rock and tailings may be
conceptualized as mixtures of two different sizes of particles. The ratios of particle
diameters of different soil types, as defined by ASTM D 422- D653, are shown in Tables
3.4 and 3.5.

<table>
<thead>
<tr>
<th></th>
<th>Clay (0.005 mm)</th>
<th>Silt (0.075 mm)</th>
<th>Sand (4.75 mm)</th>
<th>Gravel (75 mm)</th>
<th>Cobble (300 mm)</th>
<th>Boulder (&gt;1000 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay (0.001 mm)</td>
<td>5</td>
<td>75</td>
<td>4750</td>
<td>75,000</td>
<td>300,000</td>
<td>&gt;1,000,000</td>
</tr>
<tr>
<td>Silt (0.005 mm)</td>
<td>1</td>
<td>15</td>
<td>950</td>
<td>15,000</td>
<td>60,000</td>
<td>&gt;200,000</td>
</tr>
<tr>
<td>Sand (0.075 mm)</td>
<td>15</td>
<td>1</td>
<td>63</td>
<td>1,000</td>
<td>4,000</td>
<td>&gt;13,333</td>
</tr>
<tr>
<td>Gravel (4.75 mm)</td>
<td>950</td>
<td>63</td>
<td>1</td>
<td>16</td>
<td>63</td>
<td>&gt;210</td>
</tr>
<tr>
<td>Cobble (75 mm)</td>
<td>15,000</td>
<td>1000</td>
<td>16</td>
<td>1</td>
<td>4</td>
<td>&gt;13</td>
</tr>
<tr>
<td>Boulder (300 mm)</td>
<td>60,000</td>
<td>4000</td>
<td>63</td>
<td>4</td>
<td>1</td>
<td>&gt;3.33</td>
</tr>
</tbody>
</table>
Table 3.5 Ratios of average soil particle diameters.

<table>
<thead>
<tr>
<th></th>
<th>Clay 0.003 mm</th>
<th>Silt 0.04 mm</th>
<th>Sand 2.41 mm</th>
<th>Gravel 39.9 mm</th>
<th>Cobble 187.5 mm</th>
<th>Boulder 1000 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>1</td>
<td>13.3</td>
<td>803</td>
<td>13,330</td>
<td>62,500</td>
<td>333,333</td>
</tr>
<tr>
<td>Silt</td>
<td>1</td>
<td>60</td>
<td>998</td>
<td>4688</td>
<td>25,000</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>1</td>
<td>17</td>
<td>78</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td>1</td>
<td>4.7</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cobble</td>
<td>1</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Boulder</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.4 and 3.5 illustrate that soils designated by the ASTM classification system tend to have large particle size ratios, even within the same soil type. Most existing studies of soil mixtures have examined soils with large differences in absolute particle size. As an exception, Lade and Yamamuro (1997), and Yamamuro and Lade (1997, 1998) examined silt sand mixtures with a $D_{50}$ particle size difference of less than 7:1, where $D_{50}$ is the sieve opening size through which 50% of the mass of soil particles passes. Consequently, the effect of mixture ratio was typically more dominant than the effect of particle size ratio for the majority of studies reviewed. Because existing studies have focused on mixtures of soils with large differences in absolute particle size, mixture ratio has been recognized as fundamental to the structure and behaviour of soil mixtures, while particle size ratio has been neglected or ignored.
The body of evidence provided by previous studies of soil mixtures indicates that mixture structure, as defined by the particle model proposed in Section 3.4, should provide a basis for understanding and predicting the geotechnical behaviour of mixtures of mine waste rock and tailings.

3.6 Summary

Chapter Three presents a review of existing mixture theory for co-disposal, definition of mixture design variables for mixtures of waste rock and tailings, a review of particle packing theory concepts, definition of a particle model for mixtures of waste rock and tailings, and a review of geotechnical investigations of soil mixtures.

Existing studies of co-disposal have not provided a theoretical basis for relating mixture design variables to structure and geotechnical performance. Two methods of mixture design have been used for co-disposed mine wastes. Studies of pumped co-disposal indicate that maximum efficiency packing for gap-graded mixtures of coal washery occurs at a mixture ratio where tailings particles “just fill” the void space of larger coarse reject particles. The other mixture design method for co-disposal involved blending of waste rock and tailings in order to reproduce a design particle size distribution, such as is done for concrete and asphalt. While existing studies have presented methods to choose mixture design, no relation has been made between geotechnical properties and mixture structure, aside from predictions of mixture density porosity by Morris and Williams (2000b). Chapter Three therefore provides a theoretical basis for relating mixture design variables to behaviour. Design variables considered by this thesis include the mixture ratio of waste rock to tailings, and also tailings solids content.
Particle packing theory provides a basis for predicting and quantifying the structure of binary particulate mixtures. Mixtures of waste rock and tailings are described using particle packing theory for binary mixtures on the basis that particle size ratios are large. The structure of binary mixtures is dependent on mixture ratio, particle size ratio, and the porosity of individual components. Similarly, the structure of mixtures of waste rock and tailings will be dependent on the particle size ratio, mixture ratio and the porosity of individual components. Particle packing theory for binary mixtures supports the idea of using the “just filled” design criteria for mixture ratio suggested by Williams et al. (1995) to produce maximum density mixtures.

Particle packing theory also provides a basis for a conceptual particle model of the structure of mixtures of waste rock and tailings defined in Section 3.4. The model allows mixture design variables to be related to structure, thus to geotechnical properties and behaviours.

A review of geotechnical studies of soil mixtures indicated that the behaviour of soil mixtures is a function of mixture structure. Earlier studies indicated that geotechnical properties of soil mixtures depended on mixture ratio. Later studies invoked particle packing theory to provide mechanistic explanations of behaviour in terms of mixture particle structure. More recent studies have begun to predict behaviour based on particle structure. Existing studies of soil mixtures indicate that concepts from particle packing theory are directly relevant to geotechnical performance.
Specific findings relating concepts of particle packing theory to structure and geotechnical performance include:

i) particle size ratio and mixture ratio are unifying concepts that apply to all binary soil mixtures.

ii) With the exception of studies of silt-sand mixtures, the particle size ratio of soil mixtures in the literature was large and ignored. The particle size ratio of waste rock and tailings is large, and individual tailings particles will therefore tend not to interfere with the packing of waste rock particles.

iii) Mixture ratio determines whether a binary mixture is dominated by small or large sized particles. The mixture ratio of waste rock to tailings is therefore critical to determining the structure and therefore the behaviour of mixtures of waste rock and tailings.

iv) Initial tailings solids/water content governs the initial structure of a mixture with respect to the density of the tailings slurry component.

v) The structure of soil mixtures has been independently described by a number of investigators in terms of the density of packing of large and small particles. A similar description of structure is adopted here for mixtures of waste rock and tailings, e.g. waste rock skeleton void ratio, and tailings matrix void ratio.

vi) Large particle void ratios of soil mixtures have been related to mechanical strength, and provide an indication of when large particles are in contact. It is hypothesized that waste rock skeleton void ratio can provide a similar indication for mixtures of waste rock and tailings.
vii) Small particle void ratios of soil mixtures have been related to mixture hydraulic properties. Similarly, it is hypothesized that tailings matrix void ratio will govern the hydraulic conductivity of mixtures of waste rock and tailings.

Practical investigations of geotechnical properties and behaviours of mixtures of waste rock and tailings are presented in Chapters Four, Five and Six.
CHAPTER FOUR. EXPERIMENTAL METHODS

Chapter Four presents experimental methods used in laboratory and field investigations.

4.1 Introduction

A laboratory testing program and a meso-scale column study were conducted to achieve the objectives of the thesis. Laboratory scale investigations included measurement of hydraulic conductivity, consolidation, soil-water characteristic curves, tailings rheology, and particle size distributions. The meso-scale column study investigated self-weight consolidation behaviour of mixtures and waste rock. The column study was also simulated by finite element computer model.

It should be noted that the column experiment was largely complete prior to starting the laboratory testing program. The meso-scale column study was started as a Masters thesis project investigating the self-weight consolidation of mixtures of waste rock and tailings. The program of study was switched to a Ph.D. as the column study was nearing completion and the scope of research was expanded to include laboratory studies of compressibility, hydraulic conductivity, and tailings rheology. The column study took place at the Porgera Mine site, and laboratory testing took place at both the Porgera Mine, and at the University of British Columbia (UBC). Samples of waste rock and tailings were shipped to UBC for laboratory testing. The testing and results are presented out of order relative to the time of completion for the sake of clarity of presentation. Results are presented in Chapters 5 and 6.
4.2 Materials

Materials used in laboratory and column investigations included waste rock and carbon in pulp (CIP) tailings from the Porgera Gold Mine, Enga Province, Papua New Guinea. The waste rock consisted of altered sedimentary rock with some diorite. The waste rock was semi-competent, non-friable, angular with sharp edges, and was not known to contain high levels of sulphide minerals. The waste rock was scalped of larger sizes depending on the test method, with maximum particle size generally less than six times the inner diameter of testing apparatus. The tailings used were CIP tailings, which were primarily silt to clay size particles. The tailings were thickened with a flocculant and treated with calcium hypochlorite to destroy cyanide at the mill. Tailings used for laboratory tests at UBC typically settled during shipping and were re-slurried with tap water. The chemistry and mineralogy of the waste rock and tailings was not investigated, and both are considered chemically unreactive for the purposes of this thesis.

4.3 Laboratory Investigations

Laboratory tests conducted included:

i) compressibility testing of waste rock, tailings, and mixtures of the same waste rock and tailings in a large diameter consolidation cell,

ii) hydraulic conductivity testing of mixtures and of tailings by falling head test alternated with static loading,

iii) tailings rheology by rotational viscometer with mixture trials,

iv) soil-water characteristic curves of tailings and mixtures by pressure plate test,

v) Atterberg Limits of CIP tailings,

vi) particle size analysis, and
vii) gravimetric water content.

Specific details for each testing program are described in the following sections.

4.3.1 Compressibility

Compressibility testing involved placing a specimen in a rigid walled consolidation cell, applying a vertical stress to the material until the specimen came to a constant height. The results of the test allow description of compressibility in relation to the applied stress, and can be used to predict the rate and magnitude of volume change.

Specimens of tailings, waste rock, and two mixtures of tailings and waste rock were tested in a large diameter consolidation cell designed for slurry specimens. A photograph of the consolidation cell is shown in Figure 4.1 and a schematic cross section is shown in Figure 4.2.
Figure 4.1 Photograph of large diameter consolidation cell.
Figure 4.2 Schematic of large diameter consolidation cell cross section.

The consolidation cell had an inner diameter of 30.8 cm and accommodated specimens up to approximately 25 cm in height. In use, a specimen was placed the cell, a plunger was placed on top of the specimen, and then the upper lid was sealed. The plunger divided the cell into an upper air chamber and a lower specimen chamber. Vertical loads
were applied to the specimen through the plunger by pressurizing the upper air chamber. The plunger was fitted with a guide and an o-ring that sealed against the inside wall of the cell, which was chromed and honed to help maintain an air- and water-tight seal. The plunger effectively divided the cell into an upper air chamber and lower specimen chamber. The consolidation cell was plumbed to allow double drainage with ports through the base, and also through the plunger. Porous polypropylene discs of 3.7 mm thickness were placed above and below each specimen to aid in drainage. Plastic discs of 12.2 mm thickness were placed above and below the waste rock only test specimen and the cell was lined with paper in an effort to prevent damage to the chromed inner walls.

Each specimen was loaded in stress increments ranging from 0 kPa to a maximum of 320 kPa. All stress increments were applied for a period of approximately 24 hours, during which time movement of the plunger was recorded as a measurement of specimen deformation. Deformation measurements were used to confirm that primary consolidation associated with drainage and dissipation of excess pore-water pressures was complete after 24 hours of loading. Deformation of the consolidation cell was also measured with an aluminum spacer instead of a specimen. The results were used as a correction for flexure of the consolidation cell (maximum of 2 mm at 320 kPa corresponding to near 1% strain for samples examined). Initial and final water contents, and final particle size distributions were recorded for each specimen. No attempt was made to saturate specimens placed in the cell, but the top and bottom drain hose outlets were maintained at equal elevations above the surface of the specimen during testing.
4.3.2 Hydraulic Conductivity

Hydraulic conductivity testing was conducted for specimens of tailings alone and for two mixtures of waste rock and tailings. The testing method included falling head hydraulic conductivity testing in a rigid walled permeameter for approximately 24 hours alternated with static loading for a period of one to three hours. A 155 mm inner diameter aluminum mold used for hydraulic conductivity testing is shown in Figure 4.3, and a sketch is shown in Figure 4.4. The maximum particle size used was 25 mm, thereby preserving a minimum 6:1 ratio between specimen diameter and maximum particle size. Also shown in Figure 4.3 are two discs of 6.35 mm thick porous polypropylene that were machined to fit the inside diameter of the testing mold. The polypropylene discs had a high hydraulic conductivity and were placed above and below the specimen to allow removal of the base plates from the mold (also shown in Figure 4.3) for static loading. Static pressures of 0 kPa, 20 kPa, 100 kPa, and 1000 kPa were achieved with free weights, and by hydraulic press (MTS 815 Rock Mechanics Test System), shown Figure 4.5.
Figure 4.3 Permeameter with rigid walled mold, base plates, and porous discs.

Figure 4.4 Schematic of permeameter cross section.
Hydraulic conductivity was determined by the falling head test method over a period of 24 hours, where a head of water was applied to the top of the sample from a burette. The bottom of the mold was kept at a constant elevation, and hydraulic conductivity was determined from measurements of burette water level and time. The hydraulic conductivity test followed ASTM D5868-95 Test Method B with the exceptions that outflows were not measured and the ratio of specimen height to maximum particle was near 4:1 for the mixture specimens. After each falling head test for hydraulic conductivity, the end plates were removed and the specimen and mold were placed in a pan of water for static loading. The surface of the specimen was kept under a head of water during transition to prevent desaturation and desiccation. Each specimen was then loaded, and then re-tested for hydraulic conductivity by constant head test. Specimen heights were measured relative to the specimen mold before and after each increment of loading (or before and after hydraulic conductivity testing) to ensure that samples did not
change in dimensions during the hydraulic conductivity test. A photograph taken during hydraulic conductivity testing is shown in Figure 4.6.

Figure 4.6 Falling head hydraulic conductivity testing.

4.3.3 CIP Tailings Rheology

Quantitative rheological testing of the flow behaviour of tailings was conducted to determine the effect of tailings solids content on mixtures of waste rock and tailings. Mixing of tailings with waste rock requires the tailings to flow into the voids between the
rocks. If the viscosity of the tailings is too low, the tailings will not be retained within the voids and if viscosity is too high, the tailings will not be able to flow to fill the voids. Therefore the rheology (flow behaviour) of the tailings is important to mixing and retention within the waste rock voids.

The rheological properties of tailings at higher solid contents are usually “non-Newtonian” with a non-linear relationship between shear stress and shear rate, meaning the rheology cannot be characterized by a single parameter i.e. viscosity. These types of suspensions often exhibit shear-thinning properties, which have high apparent viscosities at low shear rate conditions that decrease with increasing shear rates. At higher solid contents, tailings may also have a yield stress, which can be considered a threshold shear stress to initiate flow. Since mixing of tailings with waste rock occurs at low shear rates, the low shear flow properties (i.e. apparent viscosity and yield stress) will determine how it flows into the voids and is retained within the voids.

The rheology of CIP tailings was investigated using a concentric cylinder cup and bob rotational viscometer, shown in Figure 4.7 (model ThermoHaake VT550 equipped with MV 1P bob). Specimens of CIP tailings slurry were prepared at a range of solid contents and strained to remove larger carbon particles. During a test, each specimen of tailings was loaded into the viscometer cup, a bob was lowered into the specimen within the cup, and then the bob was rotated. The viscometer measured the torque on the surface of the bob (shear stress) at controlled shear rates. The torque required to spin the bob will varied with the rheological properties of the specimen. For each test, flow curves were
obtained by increasing the shear rate from 0 s$^{-1}$ to 100 s$^{-1}$ over a period of 180 seconds, maintaining the rate at 100 s$^{-1}$ for 30 seconds and decreasing the shear rate to 0 s$^{-1}$ over a period of 180 seconds. The test was repeated three times for each specimen producing triplicate sets of flow-curves (shear stress - shear rate relationships). The yield stress and apparent viscosity values were obtained from the flow-curve. Specimens were tested for water/solids content before and after testing.

![Rotational viscometer with cup and bob fixtures.](image)

Testing of the liquid and plastic limits of the CIP tailings was also completed to provide an index of material behaviour and the effect of tailings water content. The procedure for Atterberg Limits testing followed ASTM D 4318-84 Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

### 4.3.4 Soil-Water Characteristic Curves

Soil-water characteristic curves (SWCCs) for specimens of CIP tailings and for a mixture of waste rock and tailings were obtained by pressure plate test. The mixture material
tested for SWCC was taken from the concrete mixer truck during loading of Column 1 (described in Section 4.4). The pressure plate cell is designed to allow application of a series of increasing matric suctions. For each increment of matric suction, the test specimens were allowed to come to a constant weight, which was recorded with specimen dimensions. Matric suction was increased in a progression to the limit of pressure plate. A matric suction versus volumetric water content relationship called the soil-water characteristic curve (SWCC) was then calculated from test data. The mixture of waste rock and tailings was tested in a 150 mm diameter pressure plate cell, as shown in Figure 4.8, and tailings were tested in a smaller pressure plate cell.

Figure 4.8 Large diameter pressure plate cell with scale.

4.3.5 Particle Size Distribution

Particle size distributions were used as basic description of materials, for determining mixture ratios, and for quantifying scale effects between experimental programs. Particle
size distribution of CIP tailings were determined by hydrometer analysis, using ASTM D422-63 Standard Method for Particle-Size Analysis of Soils. Particle size distribution of waste rock specimens were determined by dry sieve analysis. Particle size distribution of mixtures of waste rock and tailings were determined by washed sieve analysis, where specimens are weighed, dried, re-weighed, washed of sizes passing 75 μm, dried, re-weighed, then sized as a waste rock specimen.

4.4 Meso Scale Column Experiment

Three mixtures of waste rock and tailings were loaded into columns for observation of behaviour under self-weight consolidation. A fourth column was loaded with waste rock and no tailings as a control. Mixture designs of 7.5:1, 10:1, and 12.5:1 (all mixture ratios relate waste rock to tailings by dry mass) were prepared in a concrete transit mixer truck. For each mixture design, a known volume of tailings slurry was added to the mixer, followed by a known mass of waste rock, and subsequently mixed. The resulting mixtures were loaded into columns using a skip and crane. A photograph of the column study apparatus is included in Figure 4.9.

Mixture ratios were chosen for the column study based on a preliminary unreported bench scale study by Dr. G. Ward Wilson of UBC conducted at the Porgera Gold Mine in 2000. The study investigated the effect of mixture ratio, rock type, and consolidation on hydraulic conductivity and slump of trial mixtures. The study examined mixture ratios of 5:1, 10:1 and 20:1, and concluded that mixtures of 10:1 had the best performance in terms of hydraulic conductivity and slump. On the basis of the preliminary study, the mixture designs for the column study were to include low, ideal, and high mixture ratios,
defined as ratios of 7.5:1, 10:1, and 12.5:1, respectively. Description of behaviour of all three types of mixtures was considered valuable.

The mixture profiles were intensively monitored for settlement, pore-water pressure response, and drainage for 100 days, then instrumented with tensiometers to measure matric suctions. The three mixture columns were decommissioned after 24 months and sampled for particle size, density, and water content. The waste rock only column was monitored for settlement for approximately 21 months, and was also flooded at 13 days after column loading to determine porosity and constant head hydraulic conductivity testing.

Figure 4.9 Meso-scale columns and instrumentation shack.
4.4.1 Materials

Approximately 60 $m^3$ of freshly blasted altered sedimentary waste rock were collected for use as column fill material. The rock was selected from 2440 m level from the open pit at the Porgera Gold Mine to provide a relatively high percentage of finer materials, typically with fragments smaller than one meter in diameter. The waste rock was scalped by hand with the aid of a Hyundai HL 760 Turbo front-end loader to produce a 30 $m^3$ stockpile of waste rock with 100% passing 150 mm. Carbon in Pulp (CIP) tailings were diverted from a mill circuit and thickened to between 39 % and 44 % solids content (by mass) using a flocculant. The tailings were also treated with calcium hypo-chloride to destroy cyanide for safety purposes. A photograph of the stockpile is shown in Figure 4.10.

![Figure 4.10 Photograph of waste rock stockpile (Photo by G.W. Wilson).](image)

It should be noted that the coarse material on the right side of the photograph belongs to the pile of scalped material greater than 150 mm.
4.4.2 Column Apparatus

A photograph of the column apparatus is shown in Figure 4.9, and a schematic is included in Figure 4.11. Four columns were constructed of high density polyethylene (HDPE) pipe fitted with a steel base plate and Linatex gasket. Each column was approximately one meter in diameter and six meters in height, and plumbed with a gooseneck drain to allow a constant head lower boundary condition at 0.6 metres above the base. The tops of the columns were fitted with lids that allowed air flow but prevented influx of rainfall. The design profile of each column consisted of a layer of 0.25 m of mixture material in the base, overlain by 0.25 m of washed sand, and then by 5.5 m of mixture material. The lower 0.25 m of mixture material in the base of the columns was used as filler, and is of little consequence to the experiment. The 0.25 m sand layer was intended as a drainage layer, and included the outlet for the base drain. Filter cloth was placed directly above and below the sand layer to help prevent the movement of fine particles. The sand layer was saturated prior to placement of the 5.5 m of mixture material to ensure a hydraulic continuity between the upper mixture fill and the drain outlet.
Figure 4.11 Schematic of column apparatus and design instrumentation

Mixtures were loaded into the columns using a skip and crane. A photograph taken during construction is shown in Figure 4.12. The skip was designed with a flap on the base that could only be opened after the skip was set down on the previous lift, ensuring that lifts were placed, and not dropped. A photograph of the skip is shown in Figure 4.13. The volume of the skip allowed placement of lifts of up to 0.75 m thickness in the
columns. Minimum mixing times for the column fills was 30 minutes. Loading times varied from approximately eight hours for Column 1 to approximately four hours each for Columns 2 and 3. Column 4 was constructed using the same methods and geometry as the mixture columns but no tailings were added to the waste rock. The waste rock only fill was mixed for approximately half an hour and then loaded into Column 4 in three and a half hours.

Figure 4.12 Photograph of column loading.
4.4.3 Instrumentation and Monitoring

Each mixture profile was intensively monitored for settlement, pore-water pressure response, and drainage for a minimum period of 100 days following column loading. Differential settlement was determined from the average distance from the top of the column to the upper and lower bounds of the magnetic fields of targets buried in the fill. Pore-water pressure measurements were taken from piezometers installed within the fills.
Drainage from the base drain was collected and measured, and the depth of water ponded on surface was also measured. Piezometer leads measured approximately 0.5 m from the column wall to piezometer tip, and were ported out the sides of the columns to avoid creation of preferential vertical flow pathways. A system was devised to measure differential settlement without creating preferential vertical flow pathways in the profiles. Custom built magnetic targets were buried in each profile near the column wall and detected by lowering a commercially available magnetic reed switch probe (manufactured by RST Instruments Inc.) down a guide tube attached to the outside of each column wall. The magnetic field of each target was detectable at a distance of approximately 10 cm, and was apparently unobstructed by the waste rock, the tailings, or the column walls. Column 1 also included two pressure plates, with one near the column wall, and one near the center of the column. The plates were placed on top of the sand layer at the base of Column 1, one of which is visible in Figure 4.14. Following the 100 day monitoring period, tensiometers were temporarily installed through the column walls at 1 m intervals to measure matric suction profiles. Total settlement was taken as the average distance between the top of the column and the top of the fill.
The locations of design instrumentation are shown in Figure 4.11. Design instrumentation for Column 1 included vibrating wire piezometers at elevations of 1 m, 1.5 m, 2 m, 3 m, and 5 m; pneumatic piezometers at elevations of 1 m, 2 m, 3 m, 4 m, and 5 m; and magnetic settlement targets at elevations 0.5 m, 2 m, 4 m and 5 m. Similarly, design instrumentation for Column 2 included vibrating wire piezometers at 1 m, 2 m, 3
m, 4 m, and 5 m; and magnetic settlement targets at 0.5 m, 2 m, 4 m and 5 m. Design instrumentation for Column 3 included pneumatic piezometers at 1 m, 2 m, 3 m, 4 m, and 5 m; and magnetic settlement targets at 0.5 m, 2 m, and 4 m. Design instrumentation for Column 4 included standpipe piezometers at 1 m, 3 m, and 5 m; and magnetic settlement targets at 0.5 m, 2 m, and 4 m. The pneumatic and vibrating wire piezometers were supplied by the mine site.

Column 4 with the waste rock only profile was monitored for settlement for approximately 21 months. Column 4 was flooded with water 13 days after loading to determine porosity and to measure hydraulic conductivity. The waste rock in Column 4 was tested for hydraulic conductivity by constant head test at 13 days and 448 days. Once flooded, the constant head test involved applying a constant flux of water to the top of the column to allow overflow of water at the elevation of the top of the column wall. The rate of flow through the profile was controlled and measured at the base drain outlet valve. The change in head between standpipe piezometer outlets (shown in Figure 6.8) was measured and, with the column area, was used to calculate a value of hydraulic conductivity.

4.4.4 Column Decommissioning

Approximately 18 months after construction, the three mixture columns were flooded with water and then allowed to drain. Approximately 24 months after construction, the three columns containing mixtures were decommissioned. Final settlement measurements were recorded, and purpose built tensiometers were installed through the column walls to measure final matric suctions. The tensiometers consisted of
approximately 40 cm of 1 cm diameter aluminum tubing with a ceramic tip glued to one end, and a rubber septum on the other end. The tensiometers were filled with de-aired water, allowed to come to equilibrium with the column fill through the ceramic tip, and the internal pore water pressure was measured using a Pocket Tensiometer (from Soil Measurement Systems) accurate to ±1 kPa. Decommissioning included excavation of the mixture fills through a slot cut into the side of each column, as shown in Figure 4.15. The slot was started at the top of each column and extended downward in 1 m intervals. Fill materials were excavated in 0.5 meter increments to allow sampling for particle size analysis, water content, and density. Density measurements involved levelling the surface of the fill, removing a sample for laboratory testing, lining the resulting depression with plastic, and then measuring the volume of water required to fill the sample depression. Final positions of instrumentation were also recorded during decommissioning.
4.5 **Computer Simulation of Column Experiment**

The column study was simulated with a finite element computer model for analysis of seepage and volume change using material properties derived from the laboratory testing programs. The objective of the computer modelling was to confirm mixture material properties observed in the column and laboratory studies. The model was constructed using Column 1 geometry and derived material properties for comparison with observed behaviour in the Column study. Seepage modelling, stress modelling, and fully coupled seepage-stress modelling were done with the GeoSlope finite element software package. A diagram of the axi-symmetric mesh used is shown in Figure 4.16.
Figure 4.16 Column 1 finite element model mesh.
4.6 Summary

Chapter Four describes methods used for investigating physical properties of mixtures, including laboratory and meso-scale column experiments, as well as a finite element computer model of the column study. Laboratory testing examined compressibility, hydraulic conductivity, soil-water characteristic curves, particle size distributions, and also CIP tailings rheology and Atterberg Limits. The column study examined consolidation behaviour under self-weight, including settlement, drainage, and also pore-water pressure response. Results of laboratory and column experiments are presented in Chapters Five and Six, respectively. Analyses and discussion of the results are included in Chapters Seven and Eight, respectively.
CHAPTER FIVE. LABORATORY TESTING RESULTS

5.1 Introduction

Chapter Five includes results and observations from laboratory tests described in Chapter Four. Laboratory studies included consolidation testing, hydraulic conductivity with static loading, tailings rheology and mixture trials, soil-water characteristic curves, index properties, and particle size distributions. Results of analyses for standard consolidation parameters from consolidation test data are also presented. A summary of laboratory test specimens is presented in Table 5.1.

Table 5.1 Summary of laboratory test specimens.

<table>
<thead>
<tr>
<th>Test</th>
<th>Source Waste Rock</th>
<th>Source Tailings</th>
<th>Specimens Tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>consolidation in large diameter slurry consolidometer</td>
<td>&gt; 50 mm</td>
<td>CIP Tailings Barrel 0</td>
<td>4.4:1 mixture</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4.8:1 mixture</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6.1:1 mixture*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6.4:1 mixture*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>waste rock only</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CIP tailings only</td>
</tr>
<tr>
<td>hydraulic conductivity with static loading</td>
<td>&gt; 25 mm</td>
<td>CIP Tailings Barrel 1</td>
<td>4.2:1 mixture</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4.3:1 mixture</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CIP tailings</td>
</tr>
<tr>
<td>rheology</td>
<td>-</td>
<td>CIP Tailings Barrel 0</td>
<td>CIP tailings</td>
</tr>
<tr>
<td>soil-water characteristic curve</td>
<td>column study</td>
<td>column study</td>
<td>4.4:1 mixture (Column 1)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CIP tailings only</td>
</tr>
</tbody>
</table>

*prepared but not tested for compressibility or particle size distribution

Mixture specimens are named by test type and mixture ratio calculated by the method described in Section 5.2.3.
5.2 Specimen Description

Laboratory specimens are described here in terms of particle size distribution, water content and specific gravity, mixture ratio, and mixture structure.

5.2.1 Particle Size Distribution

Particle size distributions of consolidation test specimens of waste rock, tailings, and two mixtures are shown in Figures 5.1 to 5.3. Figure 5.1 shows the particle size distribution for the waste rock only and CIP tailings only specimens tested in the consolidation cell. The particle size distribution of a specimen taken during construction of Column 1 at 2.75 m (meso-scale column study results are presented in Chapter Six) is also shown in Figure 5.1. The maximum particle size for all waste rock tested in the consolidation cell was 50 mm. The CIP tailings in Figure 5.1 were slightly coarser than typical mill particle size distributions due to the presence of carbon particles and precipitated crystals. The CIP tailings were observed to contain carbon particles up to 2 mm in size, which are normally reclaimed from the pulp prior to tailings disposal. Crystal composition and origin were not determined, but may have been gypsum crystals precipitated due to addition of an excess of calcium hypochlorite for the purpose of destroying cyanide.
Particle size distributions of consolidation test specimens are shown in Figures 5.2 and 5.3, with source material distributions, final measured distributions, and also particle size distributions calculated from source waste rock, tailings, and mixture ratio. The portions of waste rock particles passing 75 μm were averaged over the sieve sizes smaller than 75 μm in order to determine calculated distributions. Loss of finer material to mixing equipment may have contributed to the slight variation between measured and predicted particle size distributions, as a coating of paste tailings was observed to remain on mixing equipment.
Figure 5.2 Particle size distributions of 4.4:1 mixture consolidation specimen.

Figure 5.3 Particle size distribution of 4.8:1 mixture consolidation specimen.
Particle size distributions of hydraulic conductivity test specimens are shown in Figures 5.4 and 5.5, respectively. Waste rock for the hydraulic conductivity test specimens was scalped of sizes greater than 25 mm. It should be noted that the tailings used for hydraulic conductivity testing had a slightly finer particle size distribution than tailings used for consolidation testing. Predictions of particle size distributions for the 4.2:1 and 4.3:1 mixtures calculated on the basis of source material particle size distributions and mixture ratios are included in Figures 5.4 and 5.5. Predicted distributions were similar to measured distributions.

![Particle size distribution graph](image)

Figure 5.4 Particle size distribution of 4.2:1 mixture consolidation specimen.
5.2.2 Water Content and Specific Gravity

Specimen water contents are summarized in Table 5.2. The specific gravity of the tailings solids was 2.89 and the specific gravity of the waste rock solids was 2.70. The definitions used for water and solids content are consistent throughout the thesis, including Table 5.2, and use the following definitions. Water content, $w$, or gravimetric water content is mass of water divided by mass of solids expressed as a percentage. Pulp density, $P$, or solids content for tailings is mass of solids divided by total mass of slurry, also expressed as a percentage.
Table 5.2 Water contents of laboratory specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Source waste rock (w%)</th>
<th>Source CIP tailings (w%, P%)</th>
<th>Specimen initial (w%, P%)</th>
<th>Specimen final (w%, P%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test: Consolidation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.4:1 mixture</td>
<td>0.4</td>
<td>94.8 / 51.3</td>
<td>14.7</td>
<td>13.9</td>
</tr>
<tr>
<td>4.8:1 mixture</td>
<td>0.1</td>
<td>107 / 48.4</td>
<td>17.3</td>
<td>13.1</td>
</tr>
<tr>
<td>6.1:1 mixture</td>
<td>2.6</td>
<td>133 / 42.9</td>
<td>21.0</td>
<td>*</td>
</tr>
<tr>
<td>6.4:1 mixture</td>
<td>0.9</td>
<td>91.2 / 52.2</td>
<td>18.3</td>
<td>*</td>
</tr>
<tr>
<td>waste rock only</td>
<td></td>
<td></td>
<td>0.4</td>
<td>-</td>
</tr>
<tr>
<td>CIP tailings only</td>
<td></td>
<td></td>
<td>110 / 47.6</td>
<td>39.6 / 71.6</td>
</tr>
<tr>
<td>Test: Hydraulic conductivity</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.2:1 mixture</td>
<td>oven dry</td>
<td>108 / 48.1</td>
<td>18.6</td>
<td>13.4</td>
</tr>
<tr>
<td>4.3:1 mixture</td>
<td>oven dry</td>
<td>84.8 / 54.1</td>
<td>16.9</td>
<td>12.7</td>
</tr>
<tr>
<td>CIP tailings only</td>
<td></td>
<td></td>
<td>107 / 48.4</td>
<td>41.7 / 70.6</td>
</tr>
<tr>
<td>Test: Rheology</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CIP tailings</td>
<td></td>
<td></td>
<td>166 / 37.6</td>
<td>130 / 43.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>124 / 44.6</td>
<td>95.6 / 51.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>89.0 / 52.9</td>
<td></td>
</tr>
<tr>
<td>Test: SWCC</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.4:1 mixture</td>
<td>2.8</td>
<td>129 / 43.7</td>
<td>15.7</td>
<td>10.3</td>
</tr>
<tr>
<td>CIP tailings</td>
<td></td>
<td>112 / 47.1</td>
<td>33.7</td>
<td>74.8</td>
</tr>
<tr>
<td>CIP tailings (mill sample)</td>
<td></td>
<td>140 / 41.7</td>
<td>45.4</td>
<td>68.8</td>
</tr>
</tbody>
</table>

*not tested for compressibility

The source waste rock used for construction of consolidation mixture specimens was air dried to water contents near 0.1%. The source tailings for laboratory specimens had water contents that varied between 84.8% and 108%, which are equivalent to solids contents of 54.1% and 48.1%, respectively. The SWCC mixture test specimen was constructed of waste rock at a water content at 2.8% and tailings at 129% water content or 43.7% solids content. Initial water contents determined for mixture specimens tested in the laboratory ranged between 14.7% and 18.6%, but were not considered representative because sub-samples taken for water content were small. Final water
contents were determined from whole specimens and were therefore considered to be representative. Water/solids contents of source CIP tailings used to construct mixtures were similar to specimens of CIP tailings tested for consolidation and hydraulic conductivity, ranging between 94.8% and 128.8% water content, or between 51.3% and 43.7% solids content. The specimens of CIP tailings tested for rheology ranged between 89% and 166% water content or between 52.9% and 37.6% solids content, respectively.

5.2.3 Mixture Ratio

Mixtures described in Section 5.2 are designated by mixture ratio, $R$, the ratio of waste rock to tailings by dry mass. The method of determining mixture ratio for laboratory and column specimens from particle size distributions is described here. Mixture ratios of specimens were determined from the particle size distribution of whole specimens following completion of consolidation and hydraulic conductivity testing. Mixture ratio, $R$, is defined as ratio of waste rock to tailings by dry mass, and was calculated from final particle size distributions using [5.1]:

$$[5.1] \quad R = \frac{\text{%fines CIP} - \text{%fines mixture}}{\text{%fines mixture} - \text{%fines rock}},$$

where %fines CIP, %fines mixture, and %fines rock are the percentage passing the 0.425 mm sieve for each material by mass. Equation [5.1] was derived from simple mass-volume relations.

The 6.1:1 and 6.4:1 mixtures were prepared as mixture trials were not tested for particle size distribution due to poor handling. Consequently, mixture ratios were estimated from
water content and mass of source materials, with a correction for loss of tailings to mixing equipment.

5.2.4 Structure

Initial void ratio for the waste rock and tailings mixture components were calculated for laboratory specimens and are plotted in Figures 5.6 and 5.7, respectively.

![Figure 5.6 Initial waste rock skeleton void ratios for laboratory specimens.](image)

Specimens in Figure 5.6 are designated by mixture ratio and test type and include the initial void ratio measured for the waste rock only compressibility test specimen. Figure 5.6 indicates that the initial waste rock void ratios of the laboratory mixture specimens were greater than the void ratio for the waste rock only specimen tested for compressibility. The value of initial waste rock void ratio for each specimen was variable, being subject to particle size distribution, method of placement, and also
mixture ratio. However, it is believed that the addition of tailings slurry to the waste rock void space increased the initial void ratio of the waste rock skeleton in the laboratory specimens. The waste rock used in the hydraulic conductivity specimens had a maximum particle size of 25 mm, and compressibility specimens had a maximum particle size of 50 mm. Mixture ratios and particle size distributions shown in Section 5.2.1 indicated that the hydraulic conductivity specimens had slightly higher fines contents than compressibility specimens, or lower mixture ratios.

![Figure 5.7 Initial tailings matrix void ratios for laboratory specimens.](image)

Variation in tailings matrix void ratios between laboratory specimens shown in Figure 5.7 was likely due to the variation in source tailings void ratios (also included in Figure 5.7). Figure 5.7 indicates that the initial tailings matrix void ratios were similar to that of the source tailings.
In terms of the particle model presented in Chapter Three, Figure 5.6 indicates that the laboratory mixtures had waste rock particles that were slightly separated by tailings slurry.

5.3 CIP Tailings Rheology and Mixture Trials

During preparation of mixtures for laboratory compressibility testing, it was observed that tailings solids content governed whether the tailings slurry drained from the rock voids, stayed in the rock voids, or made the mixture too stiff to mix by hand without entrainment of large air voids. For an ideal homogeneous saturated mixture, the tailings slurry should stay in the void space of the rock without entrainment of large air voids during mixing. The rheology of tailings was investigated to provide a measure of behaviour during mixing. Photographs of two mixtures prepared with different tailings solids contents are shown in Figure 5.8.
Figure 5.8 Preparation of mixtures with tailings at 42.9% solids (left) and 51.3% solids (right).

During preparation of compressibility test specimens, the 6.1:1 and 6.4:1 mixtures were not tested for compressibility or particle size distributions. The 6.1:1 mixture was blended with tailings at 133% water content, or 42.9% solids, and the tailings slurry was observed to drain from the rock voids to the bottom of the mixing vessel. The 6.1:1 specimen was not homogeneous but instead segregated to rocks in the bottom of the consolidation cell overlain by tailings slurry. It was observed that the consistency of the tailings at 42.9% solids was too wet to "float the rock," and that the volume of the tailings was greater than the voids in the rock. Another mixture was prepared at a design ratio of 4.8:1 using tailings at 91.2% water content, or 52.2% solids. The mixing process resulted in a loss of fine particles and slurry to the mixing equipment and changed the mixture ratio to approximately 6.4:1. The resulting 6.4:1 mixture was observed to be
unsaturated, with visible air voids. The rheology of CIP tailings was investigated in order to quantify the effect of tailings water/solids content on behaviour during mixing.

5.3.1 CIP Tailings Rheology

Two key rheological parameters that affect the behaviour of tailings during mixing are the yield stress and the apparent viscosity. The yield stress of a mineral suspension is defined as the stress required to initiate flow. Apparent viscosity is derived from a flow curve as the slope of a straight line passing through the origin at a specified shear rate. A specimen flow curve for CIP tailings slurry is presented in Figure 5.9. The data in Figure 5.9 show a yield stress (flow-curve intercept with shear stress axis) that indicates non-Newtonian flow behaviour typical of particulate mineral suspensions. Figure 5.9 includes curves fit to the first shear cycle data modeled by the Casson equation [5.2], which is used to describe non-Newtonian fluid behaviour,

\[ \tau^{1/2} = \tau_C^{1/2} + (\eta_C \ast D)^{1/2} \]

where \( \tau \) is shear stress, \( \tau_C \) is Casson yield stress, \( \eta_C \) is Casson apparent viscosity, and \( D \) is shear rate (Casson 1959).
The hysteresis, or difference in shear stress between the increasing shear rate phase and the decreasing shear rate phase, for the first shear cycle (labelled CIP 40-1) indicated a loss of shear strength with shearing. The amount of hysteresis decreased with successive shear cycles, labelled CIP40-2 and CIP40-3, as did the stress required to shear the specimen. The hysteresis and the apparent decrease in shear stresses for successive shear cycles shown in Figure 5.9 indicate thixotropy. Thixotropy is a time dependent increase in strength that may be destroyed by shearing. The response shown in Figure 5.9 is typical for the tailings examined. The degree of hysteresis and loss of strength upon shearing increased with solids content. Thixotropy has a potential advantage for mixture construction. The apparent viscosity of the tailings slurry may be lowered prior to mixture construction by premixing the tailings. Once mixed with rock, the apparent
viscosity of the slurry will increase with time due to thixotropy, which will help to prevent the tailings from flowing out of the rock voids.

In order to compare flow curves for different solids contents, the increasing shear rate phase of the second and third shear cycles were averaged, and apparent viscosity for averaged data are shown in Figure 5.10. The data in Figure 5.10 indicate that apparent viscosity decreased with increased shear rate (confirming shear thinning flow properties), and that apparent viscosity increased with solids content. The flow curves for the ramp up phase of the first shear cycle had greater and more variable values of shear stress compared to the second and third shear cycles. The data in Figure 5.10 are averaged, as mentioned above, and represent behaviour of tailings that have been sheared, or mixed, for several minutes.
Casson yield stress, $\tau_C$, calculated for the average of the increasing shear rate phase of the second and third shear cycles is plotted in Figure 5.11, and indicated an increase in yield stress with solids content. The Casson equation provides an estimate of yield stress that is more realistic than other methods, such as the Bingham equation, which may overestimate apparent yield stress at low shear rates (Klein & Laskowski 1999). The values of $\tau_C$ calculated for the second and third shear cycles were similar, and represent behaviour of material that had been sheared for a few minutes, such as may occur during mixing. The decrease in $\tau_C$ at solids contents greater than 51% in Figure 5.11 does not actually represent a decrease in yield stress, but rather indicates the upper solids content that may be tested by the cup and bob apparatus of the rotary viscometer. The tailings become semi-plastic at solids contents greater than approximately 51%, with the
formation of a visible shear surface around the viscometer bob during prolonged shearing (commonly referred to as wall slip). A specimen of tailings was prepared at 58.8% solids but not tested because it was too thick to be poured into the viscometer cup.

![Graph showing yield stress versus solids content for CIP tailings.](image)

**Figure 5.11** Yield stress versus solids content for CIP tailings.

Observations during preparation of laboratory specimens indicated tailings at solids contents of 42.9% ran out, or drained, from the void space of the rock matrix during mixing, as shown in Figure 5.8. A mixture created with tailings at solids contents of 52.2% was noted to be stiff during mixing, and contained visible air voids. The results of rheological testing indicated that homogeneous, saturated mixtures could best be constructed at Casson yield stress, $\tau_C$, between 15 Pa and 30 Pa, corresponding to CIP tailings solids content between 45% and 50%.
5.3.2 CIP Tailings Atterberg Limits

The liquid and plastic limits of the CIP tailings were determined for comparison to rheology data and are summarized in Table 5.3. Both the Liquid and Plastic Limits were greater than the equivalent solids contents considered mixable by hand for homogeneous, high density, unsaturated slurries. The results indicated that rheological testing methods describing yield stress are more suitable to the measurement and description of behaviour of mineral suspensions than plasticity indices commonly used in geotechnical engineering. In fact, studies have indicated that the undrained shear strengths of soils at the Atterberg Liquid Limit are typically around 1.7 kPa, regardless of soil type (Sharma and Bora 2003). Vane shear tests or slump tests are recommended for future work in characterizing tailings yield stress, as per Morris and Williams (2000), and Clayton et al. (2003).

Table 5.3 Atterberg Limits for CIP tailings.

<table>
<thead>
<tr>
<th>CIP Tailings</th>
<th>Water Content (%)</th>
<th>Solids Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Barrel 0</td>
<td>50.2</td>
<td>66.6</td>
</tr>
<tr>
<td>Barrel 1</td>
<td>50.2</td>
<td>66.6</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Barrel 0</td>
<td>32.8</td>
<td>75.3</td>
</tr>
<tr>
<td>Barrel 1</td>
<td>33.6</td>
<td>74.9</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Barrel 0</td>
<td>17.4</td>
<td></td>
</tr>
<tr>
<td>Barrel 1</td>
<td>16.6</td>
<td></td>
</tr>
</tbody>
</table>
5.4 Consolidation Testing

Waste rock, tailings, and two mixtures of waste rock and tailings were tested for consolidation behaviour in a large diameter consolidation cell described in Chapter Four. Specimens were subjected to a range of vertical applied stresses and observed for deformation and drainage. Loading rates are presented here, with compressibility and time-rate consolidation results.

5.4.1 Loading Rates

In practice, vertical stress was applied to specimens in the consolidation cell by opening a valve to increase the air pressure in the upper chamber of the consolidation cell. Delay in response time of the pressure transducer measuring air pressure in the cell also resulted in delayed fine-tuning of the pressure applied. Consequently, loads were increased over a period of minutes, rather than instantaneously, and specimens were occasionally overloaded during fine tuning of the applied load. Because the air pressure was applied from a building supply tank connected to a compressor, the applied loads also varied slightly through time with system pressure. Table 5.4 shows maximum, mean, and standard deviation of applied pressures.

Table 5.4 Variation in consolidation test applied pressure.

<table>
<thead>
<tr>
<th>Pressure Interval (kPa)</th>
<th>CIP Tailings Mean /Standard Deviation/ Maximum Pressures (kPa)</th>
<th>Waste Rock</th>
<th>4.4:1 Mixture</th>
<th>4.8:1 Mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>11/0.5/13</td>
<td>6.7/1.2/13</td>
<td>8.9/0.6/13</td>
<td>8.4/0.9/9</td>
</tr>
<tr>
<td>20</td>
<td>21/0.4/24</td>
<td>20/1.2/23</td>
<td>21/1.0/23</td>
<td>19/0.2/20</td>
</tr>
<tr>
<td>40</td>
<td>38/2.1/44</td>
<td>41/1.6/45</td>
<td>39/2.4/46</td>
<td>39/0.3/41</td>
</tr>
<tr>
<td>80</td>
<td>80/0.9/83</td>
<td>82/0.7/84</td>
<td>81/0.4/82</td>
<td>80/0.1/80</td>
</tr>
<tr>
<td>160</td>
<td>159/0.8/164</td>
<td>161/0.5/163</td>
<td>158/0.9/161</td>
<td>159/0.2/160</td>
</tr>
<tr>
<td>320</td>
<td>317/1.2/321</td>
<td>319/1.0/320</td>
<td>318/0.7/319</td>
<td>316/0.6/317</td>
</tr>
</tbody>
</table>
To generalize Table 5.4, the standard deviation in applied pressure was typically near 1 kPa, and as high as 2.4 kPa for the 40 kPa increment. Over-loading during step-up in applied pressure had most effect on waste rock specimens, which tended to deform instantaneously. Pressures applied during each increment were taken to be average measured pressures for tailings and mixture specimens and taken to be maximum measured pressures for the waste rock specimen.

Between 10 kPa and 15 kPa of applied pressure were required to mobilize the plunger in the consolidation cell and friction effects were therefore greatest relative to low pressures, particularly for applied stresses less than 20 kPa. Kinematic friction may have been lower than static values. No friction correction was applied to the data presented here.

Loading times of 24 hours were generally observed in order to provide a standard of comparison, but some variation of load times occurred.

5.4.2 Consolidation Parameters
Compressibility results for specimens tested in the consolidation cell are summarized as initial void ratios and final strains in Table 5.5, and plotted in Figure 5.12 as void ratio versus vertical effective stress. The tailings had much higher initial void ratios than the waste rock, while mixtures had the lowest initial void ratios. Total volume change or strain after the 320 kPa increment was an order of magnitude greater for the tailings than for waste rock, at 50.3% and at 4.7%, respectively. The mixtures had volume changes of 7.5% for the 4.4:1 mixture and 8.5% for the 4.8:1 mixture. The 4.4:1 mixture was
prepared with tailings at 51.3% solids and was observed to contain air voids. The volume of water leaving the 4.4:1 mixture specimen was collected during testing, and it was calculated that 2.1% of total volume change was due to air being compressed or expelled from the 4.4:1 mixture.

Table 5.5 Initial void ratio and total strain for consolidation test specimens.

<table>
<thead>
<tr>
<th></th>
<th>Void Ratio at 0 kPa</th>
<th>% Strain at 320 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP Tailings</td>
<td>3.2</td>
<td>50.3</td>
</tr>
<tr>
<td>Waste Rock</td>
<td>0.71</td>
<td>4.7</td>
</tr>
<tr>
<td>4.4:1 Mixture</td>
<td>0.49</td>
<td>7.5</td>
</tr>
<tr>
<td>4.8:1 Mixture</td>
<td>0.48</td>
<td>8.5</td>
</tr>
</tbody>
</table>

Figure 5.12 Summary of compressibility testing results.

Data in Table 5.5 indicate that the magnitude of volume change of the mixtures was more similar to the waste rock than to tailings.
Deformation versus log time data for the mixtures, waste rock, and tailings are plotted in Figures 5.13 through 5.18. The data in Figures 5.13 through 5.18 are normalized as strains with respect to initial sample height in order to allow comparison. The shape of the waste rock curves in Figures 5.13 through 5.18 was partly a function of loading. The irregular shapes of the waste rock curves in Figures 5.15 and 5.16 indicate relatively sudden changes in sample height that were possibly due to particle breakage or the crushing of asperities. Little data for waste rock is shown in Figures 5.13 and 5.14 because the tests were stopped due to a lack of movement. For higher pressure increments, the waste rock specimen underwent a sudden change in height corresponding to application of load, followed by a creep, where the sample deformed under constant load at a rate proportional to log-time.
Figure 5.13 Deformation versus log time for 0 kPa to 10 kPa stress interval.

Figure 5.14 Deformation versus log time for 10 kPa to 20 kPa stress interval.
Figure 5.15 Deformation versus log time for 20 kPa to 40 kPa stress interval.

Figure 5.16 Deformation versus log time for 40 kPa to 80 kPa stress interval.
Figure 5.17 Deformation versus log time for 80 kPa to 160 kPa stress interval.

Figure 5.18 Deformation versus log time for 160 kPa to 320 kPa stress interval.
The tailings data in Figures 5.13 through 5.18 have strain versus log time relationships with sigmoidal shapes indicating decay in pore water pressures induced by loading. The mixture data also had sigmoidal shapes indicating decay of pore water pressures, followed by a creep rate similar to the waste rock alone. Generally, the two mixtures behaved in a similar manner. Drainage of water was observed from the upper and lower ports of the consolidation cell for tailings and mixture specimens.

The data in Figures 5.13 through 5.18 indicate that the mixtures had similar degrees of deformation as the waste rock specimen. The tailings had more than double the deformation of waste rock and mixtures. The data also indicate that the tailings required more time than mixtures to consolidate. The time to dissipate pore-water pressures is a function of sample height, as well as coefficient of consolidation, \( c_v \). It should also be noted that the final height of the tailings specimen was smaller than mixture specimens due to a greater degree of volume change during the test.

Values of compression index, \( C_c \), are summarized in Table 5.6. The tailings had values of \( C_c \) ranging between 1.87 and 0.526, which were much greater than the waste rock, which had values of \( C_c \) ranging between 0.00495 and 0.0904. The mixtures had values of \( C_c \) that ranged from 0.161 to 0.0730. The value of \( C_c \) for the tailings decreased with increasing pressure. The value of \( C_c \) for the mixtures and for waste rock increased with pressure. As noted above, the plunger in the consolidation cell required 10 kPa to 15 kPa of applied pressure to overcome sidewall friction with no specimen in place.
Table 5.6 Compression indices, $C_c$.

<table>
<thead>
<tr>
<th>Pressure Interval (kPa)</th>
<th>CIP Tailings (lab)</th>
<th>Waste Rock (lab)</th>
<th>4.4:1 Mixture</th>
<th>4.8:1 Mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td>0.839</td>
<td>0.016</td>
<td>0.016</td>
<td>0.048</td>
</tr>
<tr>
<td>10-20</td>
<td>1.87</td>
<td>0.005</td>
<td>0.067</td>
<td>0.052</td>
</tr>
<tr>
<td>20-40</td>
<td>0.565</td>
<td>0.053</td>
<td>0.042</td>
<td>0.042</td>
</tr>
<tr>
<td>40-80</td>
<td>0.712</td>
<td>0.035</td>
<td>0.065</td>
<td>0.050</td>
</tr>
<tr>
<td>80-160</td>
<td>0.596</td>
<td>0.090</td>
<td>0.073</td>
<td>0.054</td>
</tr>
<tr>
<td>160-320</td>
<td>0.526</td>
<td>0.086</td>
<td>0.073</td>
<td>0.059</td>
</tr>
</tbody>
</table>

Consolidation parameters determined for the laboratory consolidation testing using the inflection point method (Robinson 1997, Mesri et al. 1999) are summarized in Tables 5.7 through 5.10. The values of mixture consolidation parameters calculated from data in Figures 5.13 through 5.18 are subject to interpretation. Data from analysis of Column 1 is included for comparison and is discussed in Chapters Six and Seven.

Table 5.7 Coefficients of consolidation, $c_v$.

<table>
<thead>
<tr>
<th>Nominal Pressure kPa</th>
<th>CIP tailings $c_v$ m^2/s</th>
<th>4.4:1 Mixture $c_v$ m^2/s</th>
<th>4.8:1 Mixture $c_v$ m^2/s</th>
<th>Column 1 (average $c_v$) m^2/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 10</td>
<td>2.0*10^-7</td>
<td>1.2*10^-6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 to 20</td>
<td>2.2*10^-7</td>
<td>1.5*10^-6</td>
<td>1.6*10^-6</td>
<td></td>
</tr>
<tr>
<td>20 to 40</td>
<td>3.2*10^-7</td>
<td>6.7*10^-6</td>
<td>2.2*10^-6</td>
<td></td>
</tr>
<tr>
<td>40 to 80</td>
<td>4.6*10^-7</td>
<td>1.6*10^-5</td>
<td>9.3*10^-6</td>
<td></td>
</tr>
<tr>
<td>80 to 160</td>
<td>6.6*10^-7</td>
<td>1.8*10^-5</td>
<td>1.5*10^-5</td>
<td></td>
</tr>
<tr>
<td>160 to 320</td>
<td>9.5*10^-7</td>
<td>2.3*10^-5</td>
<td>6.7*10^-5</td>
<td>8.8*10^-6</td>
</tr>
</tbody>
</table>

Values of coefficient of consolidation, $c_v$, in Table 5.7 were similar for the two mixtures, ranging from $1.2*10^{-6}$ m^2/s to $6.7*10^{-5}$ m^2/s. The value of $c_v$ for the CIP tailings
ranged from $2.0 \times 10^{-7}$ m$^2$/s to $9.5 \times 10^{-7}$ m$^2$/s, differing from the mixtures by more than an order of magnitude.

Coefficients of volume change, $m_v$, for laboratory test data and for Column 1 are summarize in Table 5.8.

Table 5.8 Coefficients of volume change, $m_v$.

<table>
<thead>
<tr>
<th>Pressure Range</th>
<th>CIP Tailings (lab) $m_v$</th>
<th>4.4:1 Mixture $m_v$</th>
<th>4.8:1 Mixture $m_v$</th>
<th>Waste Rock (lab) $m_v$</th>
<th>Column 1 (average $m_v$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 10</td>
<td>$1.8 \times 10^{-2}$</td>
<td>$6.2 \times 10^{-4}$</td>
<td></td>
<td>$3.9 \times 10^{-4}$</td>
<td></td>
</tr>
<tr>
<td>10 to 20</td>
<td>$1.9 \times 10^{-2}$</td>
<td>$8.0 \times 10^{-4}$</td>
<td>$1.0 \times 10^{-3}$</td>
<td>$7.0 \times 10^{-5}$</td>
<td></td>
</tr>
<tr>
<td>20 to 40</td>
<td>$2.8 \times 10^{-3}$</td>
<td>$3.8 \times 10^{-4}$</td>
<td>$4.0 \times 10^{-4}$</td>
<td>$4.1 \times 10^{-4}$</td>
<td></td>
</tr>
<tr>
<td>40 to 80</td>
<td>$1.2 \times 10^{-3}$</td>
<td>$2.1 \times 10^{-4}$</td>
<td>$2.0 \times 10^{-4}$</td>
<td>$1.4 \times 10^{-4}$</td>
<td></td>
</tr>
<tr>
<td>80 to 160</td>
<td>$4.6 \times 10^{-4}$</td>
<td>$9.4 \times 10^{-5}$</td>
<td>$9.5 \times 10^{-5}$</td>
<td>$1.9 \times 10^{-4}$</td>
<td></td>
</tr>
<tr>
<td>160 to 320</td>
<td>$1.9 \times 10^{-4}$</td>
<td>$8.5 \times 10^{-5}$</td>
<td>$5.5 \times 10^{-5}$</td>
<td>$9.4 \times 10^{-5}$</td>
<td>$8.5 \times 10^{-4}$</td>
</tr>
</tbody>
</table>

Values of $m_v$ in Table 5.8 were similar for the two mixtures, ranging between $5.5 \times 10^{-5}$ 1/kPa and $1.0 \times 10^{-3}$ 1/kPa. The tailings had values of $m_v$ that were higher than that of the mixtures, ranging between $1.9 \times 10^{-4}$ 1/kPa and $1.9 \times 10^{-2}$ 1/kPa. Waste rock had values of $m_v$ that were similar to the mixtures, ranging between $7.9 \times 10^{-5}$ 1/kPa and $4.1 \times 10^{-4}$ 1/kPa. Based on the values presented in Table 5.8, both mixtures had values of $m_v$ that were an order of magnitude lower than tailings and more similar to waste rock alone.
Values of secondary compression index, $C_\alpha$, in Table 5.9 were similar for the mixtures and waste rock, with mixtures having perhaps slightly lower creep rates. Insufficient data was collected to determine values of $C_\alpha$ for tailings alone. Values of $C_\alpha$ for the waste rock ranged between $3.1 \times 10^{-3}$ and $1.7 \times 10^{-2}$. Values of $C_\alpha$ for the mixtures ranged between $3.0 \times 10^{-4}$ and $8.5 \times 10^{-3}$. The 4.8:1 mixture had slightly lower values of $C_\alpha$ compared to the waste rock and 4.4:1 mixture. It is acknowledged that further volume change would have occurred had the tests been allowed to continue. However, loading times of approximately 24 hours were observed to provide a standard of comparison.
Table 5.10 Hydraulic conductivity from consolidation analysis, \( k \).

<table>
<thead>
<tr>
<th>Nominal Pressure kPa</th>
<th>CIP Tailings ( k ) (m/s)</th>
<th>4.4:1 Mixture ( k ) (m/s)</th>
<th>4.8:1 Mixture ( k ) (m/s)</th>
<th>Column 1 (average) ( k ) (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 10</td>
<td>3.5( \times 10^{-8} )</td>
<td>7.3( \times 10^{-9} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 to 20</td>
<td>4.2( \times 10^{-8} )</td>
<td>1.2( \times 10^{-8} )</td>
<td>1.6( \times 10^{-8} )</td>
<td></td>
</tr>
<tr>
<td>20 to 40</td>
<td>8.8( \times 10^{-9} )</td>
<td>2.5( \times 10^{-8} )</td>
<td>8.8( \times 10^{-9} )</td>
<td></td>
</tr>
<tr>
<td>40 to 80</td>
<td>5.4( \times 10^{-9} )</td>
<td>3.3( \times 10^{-8} )</td>
<td>1.8( \times 10^{-8} )</td>
<td></td>
</tr>
<tr>
<td>80 to 160</td>
<td>3.0( \times 10^{-9} )</td>
<td>1.7( \times 10^{-8} )</td>
<td>1.4( \times 10^{-8} )</td>
<td></td>
</tr>
<tr>
<td>160 to 320</td>
<td>1.8( \times 10^{-9} )</td>
<td>1.9( \times 10^{-8} )</td>
<td>3.6( \times 10^{-8} )</td>
<td>3.9( \times 10^{-8} )</td>
</tr>
</tbody>
</table>

Data presented in Table 5.10 indicate tailings had values of hydraulic conductivity, \( k \), ranging between \( 1.8\times 10^{-9} \) m/s and \( 3.5\times 10^{-8} \) m/s. The mixtures had values of \( k \) ranging between \( 7.3\times 10^{-9} \) m/s and \( 3.6\times 10^{-8} \) m/s.

5.4.3 Change in Structure

Volume change occurring due to applied pressures during consolidation testing are expressed as changes in waste rock skeleton and tailings matrix void ratios in Figures 5.19 and 5.20, respectively.
The data in Figure 5.19 indicate that the mixture specimens followed $e_r$ versus applied pressure relationships that were similar to the waste rock alone. However, the mixtures had a greater changes in $e_r$ and also had higher initial values of $e_r$ than the waste rock specimen. Values of $e_r$ for the mixtures were initially higher than for waste rock alone, but decreased to become similar to waste rock alone at higher pressures. In terms of particle packing theory, and the particle model presented in Chapter Three, the specimens change from a structure of waste rock separated by tailings particles to one with a load bearing waste rock skeleton. The waste rock in the mixtures appeared to dominate compressibility behaviour. The finding is significant with respect to the mechanical behaviour of mixtures.
Figure 5.20 Mixture tailings matrix void ratio, \(e_t\), versus applied pressure.

Figure 5.20 indicates that the initial values of tailings matrix void ratio, \(e_t\) for the mixtures were similar to, though slightly lower than, the initial \(e\) of the tailings only specimen. Figure 5.20 also indicates that values of \(e_t\) for the mixtures did not undergo the same degree of change as \(e\) for the CIP tailings only specimen. It is noted that because of strain compatibility the total volume of the tailings slurry is equal to the volume of the waste rock skeleton void space. The strain of the tails in the rock void space is related to total strain of the mixture by the initial waste rock skeleton porosity, \(n_{r0}\), using [5.3]:

\[
[5.3] \quad \text{Strain Rock Voids} = \frac{\text{Total Strain of Mixture}}{n_{r0}}.
\]

where

\[
[5.4] \quad n_r = \frac{e_r}{(1+e_r)}
\]
The similar responses of mixtures and waste rock alone with respect to the magnitude of volume change indicated a dominance of waste rock relative to the tailings in the mixtures. Examination of the change in structure of mixture specimens in terms of $e_r$ and $e_t$ indicated that mixtures behave more like waste rock than tailings. The presence of a continuous waste rock skeleton limits the degree of consolidation possible by the tailings in the waste rock void space. The relation between mixture structure and compressibility behaviour is explored further in Chapter Seven.

5.5 Hydraulic Conductivity

Detailed hydraulic conductivity testing results are presented here, and related to pre-consolidation pressure, hydraulic gradients, and specimen structure.

5.5.1 Summary

One CIP tailings specimen and two mixture specimens were tested for hydraulic conductivity by falling head test alternated with static loading. Hydraulic conductivity testing results are summarized in Table 5.11 and plotted in log-log scale with applied pressure in Figure 5.21. It should be noted that following application of loads, specimens were unloaded and allowed to expand prior to hydraulic conductivity testing. Hydraulic conductivity data in Table 5.11 and Figure 5.21 are geometrically averaged values of individual test results with outliers removed.
Table 5.11 Hydraulic conductivity test results.

<table>
<thead>
<tr>
<th>Preconsol. Pressure (kPa)</th>
<th>CIP Tailings</th>
<th>4.2:1 Mixture</th>
<th>4.3:1 Mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hydraulic Conductivity (m/s)</td>
<td>Hydraulic Conductivity (m/s)</td>
<td>Hydraulic Conductivity (m/s)</td>
</tr>
<tr>
<td>1</td>
<td>4.2*10^{-7}</td>
<td>1.6*10^{-8}</td>
<td>2.3*10^{-8}</td>
</tr>
<tr>
<td>12</td>
<td>2.1*10^{-8}</td>
<td>1.2*10^{-8}</td>
<td>1.4*10^{-8}</td>
</tr>
<tr>
<td>22</td>
<td>1.2*10^{-8}</td>
<td>1.6*10^{-8}</td>
<td>1.4*10^{-8}</td>
</tr>
<tr>
<td>175</td>
<td>7.3*10^{-9}</td>
<td>1.5*10^{-8}</td>
<td>8.3*10^{-9}</td>
</tr>
<tr>
<td>1160</td>
<td>1.8*10^{-9}</td>
<td>4.9*10^{-9}</td>
<td>4.3*10^{-9}</td>
</tr>
</tbody>
</table>

The CIP tailings had a higher initial value of hydraulic conductivity, $k$, and underwent a greater change in $k$ compared to the mixtures due to static loading. The value of $k$ of the CIP tailings specimen decreased by more than two orders of magnitude from $4.2*10^{-7}$ m/s in an unconsolidated state to $1.8*10^{-9}$ m/s after the 1160 kPa stress increment.
Similarly, the value of $k$ of the 4.2:1 mixture decreased from $6.5 \times 10^{-8}$ m/s to $4.9 \times 10^{-9}$ m/s, and the value of $k$ of the 4.3:1 mixture decreased from $2.3 \times 10^{-8}$ m/s to $4.3 \times 10^{-9}$ m/s.

5.5.2 Gradients

Maximum and minimum gradients used during the test are summarized in Table 5.12. The unconsolidated CIP tailings specimen was tested only at low gradients in order to prevent disturbance of the specimen, which was in slurry form.

Table 5.12 Gradients for hydraulic conductivity testing.

<table>
<thead>
<tr>
<th>Gradient</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP Tailings 4.2:1 Mixture</td>
<td>8.74</td>
<td>1.40</td>
</tr>
<tr>
<td>4.3:1 Mixture</td>
<td>5.90</td>
<td>1.38</td>
</tr>
<tr>
<td>5.60</td>
<td>2.09</td>
<td></td>
</tr>
</tbody>
</table>

Plots of hydraulic conductivity versus gradient are shown in Figures 5.22, 5.23, and 5.24. In general, $k$ was not found to be dependent on hydraulic gradient. Data in Figure 5.22 indicate that the value of $k$ for CIP tailings was not related to gradient for the CIP tailings. However, the $k$ data for CIP tailings following the 1160 kPa increment suggest a weak relationship between the value of $k$ and hydraulic gradient at high preconsolidation pressures. Data in Figure 5.23 and 5.24 do not show a conclusive relationship between the value of $k$ and gradient for the mixtures tested. Data presented in Figures 5.23, and 5.24 indicate that $k$ was not dependent on hydraulic gradient, for the ranges presented in Table 5.12, suggesting that Darcy's Law was valid for the mixtures examined.
Figure 5.22 Log of hydraulic conductivity versus gradient for CIP tailings.

Figure 5.23 Log of hydraulic conductivity versus gradient for the 4.2:1 Mixture.
5.5.3 Structure

Laboratory hydraulic conductivity testing results are examined here with respect to mixture structure. Mixture structure is described in terms of void ratio $e$ in Figure 5.25 and by tailings matrix void ratio, $e_t$, in Figure 5.26. Values of $e$ and $e_t$ were calculated from changes in specimen length relative to final specimen measurements. Specimen lengths were calculated from an average of five measurements taken over the specimen area. Differences in measurements of specimen length taken prior to and after each falling head test were less than 1.0%. Void ratios were calculated using final measurements of mass, water content, as well as specific gravity of waste rock solids, specific gravity of tailings solids, and mixture ratio (mixture ratio was calculated using the method described in Section 5.2.3).
Figure 5.25 Void ratio versus log of hydraulic conductivity.

Data in Figure 5.25 indicate that both the mixtures and the CIP tailings had log-linear relationships between hydraulic conductivity, \( k \), and void ratio, \( e \). The value of \( e \) of the tailings was much greater than the value of \( e \) for the mixtures, for the same value of \( k \). Values of \( k \) for the mixture specimens were more sensitive to changes in \( e \) than the CIP tailings.

The hydraulic conductivity of mixtures is examined here with respect to the conceptual model introduced in Chapter Three. Water flowing through a mixture must pass around the rocks, which are impermeable relative to the tailings matrix. All flow must therefore pass through the boundary layer at the surface of rocks and through the tailings matrix. The value of the tailings matrix void ratio, \( e_t \), will therefore influence the value of \( k \) of a homogeneous mixture. Values of \( e_t \) are plotted versus log of \( k \) in Figure 5.26.
In general, values of $k$ were higher for CIP tailings than mixtures at low preconsolidation pressures, and values of $k$ were lower for CIP tailings than mixtures at high preconsolidation pressures. The difference is attributed to the change in particle structure. The specimen of CIP tailings alone had a greater change in $e$ and $k$ than the mixtures subjected to the same applied stress. Presumably, the presence of waste rock in the mixture reduced the initial value of $k$ relative to tailings alone by reducing the area available for flow. The presence of waste rock also reduced total volume change in the mixtures. While the strain compatibility effect implied by Equation [5.3] magnifies the strain experienced by the tailings in the waste rock void space, the total strain experienced by the mixture was limited by the presence of a load bearing waste rock skeleton. The mixtures therefore had smaller changes in $k$ compared to tailings alone. The change in $k$ of the mixture with compression is therefore dependent on mixture...
particle structure. Figure 5.26 indicates that $k$ is a function of $e_t$ and it is suggested that $e_t$ provides a better indicator of $k$ than $e$ for mixtures of waste rock and tailings.

5.6 Soil-water Characteristic Curves

Section 5.6 presents soil-water characteristic curves (SWCCs) for a mixture of waste rock and tailings, a tailings only specimen, and a waste rock only specimen. Shrinkage curves are also presented for the mixture and tailings specimens. The SWCC data for the tailings and mixture include curves corrected for volume change, and also uncorrected curves. A comparison and summary of results is included.

The SWCC relates volumetric water content to matric suction. Volumetric water content is defined as volume of water in a specimen divided by the total volume of the specimen. In order to determine volumetric water content, the total volume of the specimen is determined at the beginning, or end, of the pressure plate test. Typically, the change in mass of water is measured as matric suctions are increased, and total volume is assumed to be constant for the duration of the test. When specimens undergo volume change during the pressure plate test, the assumption of constant total volume may generate a considerable error in the SWCC and the Air Entry Value (AEV) (Fredlund 1999).

During pressure plate testing, the mixture and tailings specimens were observed to undergo a large degree of volume change. Consequently, specimen dimensions were recorded in an effort to accurately determine SWCC curves. Shrinkage curves derived from volume measurements taken during the pressure plate tests are presented below with SWCC data corrected for volume change.
5.6.1 Mixture of Waste Rock and Tailings

The soil-water characteristic for a 4.4:1 mixture specimen is presented in Figure 5.27. The mixture was sampled from the concrete mixer truck during loading of Column 1 of the meso scale column study at 2.75 m, and then shipped to UBC for analysis. The mixture ratio was estimated to be 4.4:1 by particle size analysis of a duplicate sample. Values of volumetric water content at 1,000,000 kPa are assumed to be zero.

![Figure 5.27 Soil-water characteristic curve for mixture of waste rock and tailings.](image)

The data labelled “4.4:1 Mixture” in Figure 5.27 were corrected for volume change during the test. Uncorrected data are also included. The uncorrected data in Figure 5.27 were calculated with the assumption of a constant initial volume. The corrected data in Figure 5.27 are calculated with the assumption that specimen volume changed for each matric suction - with the volume change held equal to the volume of water lost.
(corrected data assume 100% saturation for the duration of the test with the exception of the 1,000,000 kPa suction point). Fredlund (1999) noted that volume change significantly affects the SWCC and AEV of specimens that change in volume during the pressure plate test, and suggested using a correction for volume change based on the shrinkage limit. No shrinkage limit tests were performed for this thesis. Instead, both uncorrected data and corrected data are presented in Figure 5.27. The void ratio versus water content relationship, or shrinkage curve, is presented in Figure 5.28.

The data points in Figure 5.28 were derived from measurements of sample volume and are plotted against the saturation line. The shrinkage curve presented in Figure 5.28 does not indicate an AEV. It is likely that the maximum suction applied during the test (which was 100kPa excluding the oven dry condition) did not exceed the AEV of the material. The curved shape of the SWCC in Figure 5.27 is attributed to changes in specimen...
volume, rather than desaturation. The method used here allows direct determination of volume for each pressure interval, but is subject to errors in measuring the specimen volume.

The 4.4:1 mixture material was observed to develop cracks on surface above 40 kPa matric suction. Cracks were attributed to shrinkage of the tailings matrix within the waste rock skeleton, which had a high initial water content.

5.6.2 CIP Tailings

The SWCC's for two specimens of CIP tailings are presented in Figure 5.29. Values of volumetric water content at 1,000,000 kPa are assumed to be zero.

![Graph showing Soil-water characteristic curve for CIP tailings.](image)

**Figure 5.29** Soil-water characteristic curve for CIP tailings.
As for the mixture described above, Figure 5.29 includes uncorrected data and data corrected for volume change. The tailings underwent a large volume change during the test, and volumes were calculated for each pressure interval by the method described above. Measurements of specimen volume are summarized as a void ratio versus water content, or shrinkage curve, in Figure 5.30.

The CIP tailings changed in volume prior to application of matric suctions, and continued to shrink during the test with application of matric suctions. The dimensions of CIP tailings specimens were measured at each pressure increment in an attempt to determine specimen volumes and AEVs. However, shrinkage of the specimen resulted in rounding and an irregular shape which prevented accurate measurements of volume at higher pressures. All water loss from the tailings specimens to at least 60 kPa matric suction was attributed to changes in volume rather than desaturation. The AEV was likely higher.
than 100 kPa. The initial curved shape of the SWCC in Figure 5.29 for CIP tailings is attributed to volume change, and does not indicate an AEV.

5.6.3 Waste Rock

The SWCC for altered sedimentary waste rock was not measured for this thesis. However, a material similar to the waste rock used in the study had been previously tested for SWCC. A black sedimentary rock with a similar grain size to the altered sedimentary rock used in the mixtures and majority of tests presented in this thesis. The black sedimentary rock was less competent than the altered sedimentary rock used in the mixtures, particularly when subjected to loading and wet/dry cycles. Particle size data are presented in Figure 5.31 for the black sedimentary waste rock, the 4.4:1 mixture and the mixture source materials (the 4.4:1 mixture SWCC is presented in Section 5.6.1).

Figure 5.31 Particle size distribution of mixture and black sedimentary waste rock.
The particle size distribution of the black sedimentary rock tested for SWCC was similar to the altered sedimentary waste rock used as a source for the 4.4:1 mixture, which is labelled “Source Rock” in Figure 5.31. The particle size distributions labelled “Black Sedimentary Waste Rock” and the “Source Rock” in Figure 5.31 were similar. On the basis of similar particle size distributions, the SWCC for the black sedimentary waste rock is included as an example of a waste rock that was similar to the altered sedimentary waste rock used to construct the mixture. The waste rock SWCC allows semi-quantitative comparison between material types. The SWCC for black sedimentary rock from the Porgera site is presented in Figure 5.32.

![SWCC for black sedimentary waste rock](image)

**Figure 5.32 SWCC for black sedimentary waste rock.**

The SWCC for black sedimentary rock presented in Figure 5.32 indicates an AEV of less than 1 kPa, with volumetric water contents less than 0.1 at suctions greater than 1 kPa. An air entry value of 0.023 kPa was calculated for the rock based on a mean particle
(D_{50}) radius, \( r \), of 15 mm and a surface tension \( T_s \), of 0.07275 N/m at 20°C, using equation [5.4] from Reinson et al (2005):

\[
[5.4] \ AEV = 2 \ T_s / \{ (r / (\cos45)) - r \}
\]

The SWCC for the black sedimentary waste rock did not undergo significant volume change during the test, and the data in Figure 5.32 were not corrected for volume change.

**5.6.4 Summary and Comparison of SWCC**

The SWCC's for the 4.4:1 mixture, the CIP tailings, and for black sedimentary waste rock are presented in Figure 5.33. Data for the 4.4:1 mixture and CIP tailings are corrected for volume change in Figure 5.33, while waste rock data are uncorrected.

![Figure 5.33 Comparison of SWCC for tailings, mixture, and waste rock.](image)
The data in Figure 5.33 indicate that the tailings and the mixture retained more water at high matric suctions than the waste rock alone, which desaturated at low matric suctions. The mixture was intermediate to the waste rock and tailings in terms of volumetric water content, with the exception of near zero matric suctions. The mixture retained slightly less than half the volume of water retained by the tailings, and more than three times the water retained by the waste rock, excluding near zero matric suctions. Changes in storage for the mixture and tailings were associated with shrinkage of total specimen volume, rather than desaturation.

The CIP tailings had the greatest initial volumetric water content, followed by the waste rock, then by the 4.4:1 mixture. The waste rock was observed to desaturate at matric suctions less than 1 kPa. Error in measuring specimen volume, and the maximum matric suction of the pressure plate prevented accurate determination of the AEV of the tailings and the mixture specimens. Volume change measurements and observations indicated that the AEVs of the tailings and mixture specimens were greater than 60 kPa. However, the mixture was observed to crack at suctions greater than 40 kPa. The mixture and tailings data in Figure 5.33 are corrected for volume change with the assumption that the specimens remained saturated. Error introduced by the assumption of saturation will affect the SWCCs at high matric suctions only, and was much less than errors introduced if measured specimen volumes were used to calculate specimen volumes. Irregular specimen shape, including internal cracks at higher suctions, prevented accurate determination of specimen volume.
The AEV is defined as "...the matric suction value that must be exceeded before air recedes into the soil pores..." (Fredlund and Rahardjo 1993). Desaturation occurs when the AEV is exceeded, allowing drainage pores of a continuous porous soil media. Cracking is a macroscopic phenomenon where the continuous porous media becomes non-continuous. Cracking does not imply desaturation of the porous media, but introduces air into the sample by changing the soil macrostructure. The change in macrostructure will drastically affect properties such as hydraulic conductivity and bulk air permeability, depending on the size and connectivity of the cracks. Volume change and cracking during the test were associated with high initial water contents.

5.7 Summary

Chapter Five included results of laboratory investigations of mixtures, tailings, and waste rock for particle size distributions, mixture ratios and structure, compressibility, hydraulic conductivity, soil-water characteristic curves, as well as tailings rheology, mixture trials, and Atterberg Limits.

Particle size distributions, water contents, mixture ratios were defined for laboratory specimens. Mixture ratios examined ranged from 4.2:1 to 6.4:1 waste rock to tailings by dry mass. The structures of mixtures examined in the laboratory were near the “just filled” mixture design criteria, with waste rock particles initially slightly separated by tailings slurry.
Rheology tests and mixture trials indicated that homogenous, saturated mixtures could best be constructed at Casson yield stress between 15 Pa and 30 Pa, corresponding to CIP tailings solids content between 45% and 50%. Yield stress is noted to provide a general criterion for behaviour of tailings during mixing that may be applied independently of material type.

Consolidation testing indicated that mixture compressibility behaviour was dominated by the waste rock skeleton. The finding is significant to the mechanical behaviour of mixtures. The magnitude of volume change at a compressive pressure of 320 kPa was 50% for the tailings, 5% for the waste rock, and between 7% and 9% for the mixture specimens. Compressibility parameters for the mixtures were more similar to the waste rock, and an order of magnitude different than the tailings. Mixtures had values of $c_v$, which were an order of magnitude greater than for tailings alone, indicating faster rates of consolidation, while the majority of consolidation for waste rock occurred instantaneously upon application of load. Creep rates, or values of secondary compression index $v\alpha$, were slightly higher for waste rock than for the mixtures.

Mixtures under compression had a change in structure whereby the waste rock and tailings void ratios decreased. By strain compatibility, the presence of a waste rock skeleton limited the degree of consolidation in of the tailings matrix.

The hydraulic conductivity of mixtures was similar to tailings, and much lower than for waste rock alone. The hydraulic conductivity of tailings alone changed by a greater
degree than mixtures subjected to similar preconsolidation pressures. The presence of a waste rock skeleton limited the consolidation of the tailings in the waste rock void space. Hydraulic conductivity was found to be independent of hydraulic gradient for the mixtures and gradients examined. Data indicate that changes in tailings matrix void ratio, $e_t$, provide a better indicator of hydraulic conductivity than $e$ for mixtures of waste rock and tailings.

Soil-water characteristic curves for tailings alone, a mixture, and waste rock alone were compared. Waste rock desaturated at low matric suctions, while the mixture and tailings maintain water saturation at higher matric suctions. Increases in matric suction were associated with volume change for the tailings and mixture specimens, but accurate determination of an AEV was not possible due to error in measuring specimen volumes. AEV for the tailings and mixture was likely greater than 60 kPa. The mixture was noted to develop cracks within the tailings matrix at suctions greater than 40 kPa.

Laboratory results are analyzed and compared to results from the Column study in Chapters Seven and Eight.
CHAPTER SIX. MESO SCALE COLUMN STUDY

Chapter Six presents results and observations from the meso-scale column study.

6.1 Introduction and Overview

Three mixture designs were prepared in a concrete transit mixer, then loaded into meso-scale columns, as described in Chapter Four. Once loaded, each column fill was monitored for drainage, differential and total settlement, and pore-water pressure response. A column containing waste rock only was constructed as a control and monitored for settlement. Observations made during construction and deconstruction indicate that one mixture was homogeneous, while the other two mixture profiles were segregated. The segregated column fills included saturated material overlain by unsaturated material consisting of larger cobble sizes coated in tailings paste. The saturated portions of mixture fills were observed to undergo double drainage, followed by single drainage and then development of negative pore-water pressures. The drainage of the fills was associated with dissipation of excess pore-water pressures generated by the self-weight of the fill upon placement. Following dissipation of excess pore-water pressures and drainage of ponded water, the fills developed negative pore-water pressures from the top down. The mixture columns were decommissioned approximately 24 months after construction. A summary of column fill behaviour is included in Table 6.1.
Table 6.1. Summary of column study.

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average mixture ratio for saturated portion of fill (waste rock to tailings by dry mass)</td>
<td>5:1</td>
<td>6:1</td>
<td>7:1</td>
<td>1:0</td>
</tr>
<tr>
<td>Initial length of fill (m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unsaturated</td>
<td>5.3</td>
<td>3.1</td>
<td>1.6</td>
<td>3.8</td>
</tr>
<tr>
<td>Saturated</td>
<td>5.2</td>
<td>2.2</td>
<td>3.8</td>
<td></td>
</tr>
<tr>
<td>Settlement (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>at 100 days</td>
<td>4.6</td>
<td>2.4</td>
<td>4.7</td>
<td>7.5†</td>
</tr>
<tr>
<td>at end of test (+740 days)</td>
<td>5.7</td>
<td>3.5</td>
<td>5.7</td>
<td>9.6</td>
</tr>
<tr>
<td>Time to dissipation of excess pore water pressures (days)</td>
<td>&gt;27</td>
<td>&gt;23</td>
<td>&gt;1.4</td>
<td>-</td>
</tr>
<tr>
<td>Settlement during dissipation of excess pore-water pressures (%)</td>
<td>3.8</td>
<td>1.5</td>
<td>2.9</td>
<td>-</td>
</tr>
<tr>
<td>Hydraulic Conductivity (m/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>oedometer / consolidation</td>
<td>3.9*10^-8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>drainage of ponding</td>
<td>1.1*10^-8</td>
<td>5.4*10^-8</td>
<td>1.3*10^-7</td>
<td>0.3</td>
</tr>
<tr>
<td>constant head test</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

† base drain closed and profile flooded after 13 days and 5.6 % total settlement

6.2 Column Fill Structure

Section 6.2 presents a summary of observations of column fill structure, data for particle size distributions, water contents, mixture ratios, analysis of column mixture design versus as-built profiles, and description of mixture fill structure in terms of the particle model presented in Section 3.4. The purpose of examining fill structure was to provide a description of the material, to determine reasons for variation from design mixture ratios, and to provide a quantitative description of mixture structure that may be related to geotechnical behaviours.
6.2.1 Summary and General Observations

The column fills were designed to be homogeneous, with mixture ratios of 7.5:1, 10:1 and 12:1. As-built profiles varied considerably from design. Of three mixture designs constructed, Column 1 had a homogeneous profile with a mixture ratio that was slightly lower than design, while Columns 2 and 3 had segregated profiles due to mixture design and sorting by the concrete transit mixer. The mixture designs for Columns 2 and 3 did not provide enough tailings slurry to fill the volume of the void space in the waste rock. The concrete transit mixer acted to sort saturated finer materials from unsaturated coarser materials coated in tailings paste, resulting in profiles consisting of a lower, saturated portion of mixture material overlain by unsaturated coarse material. The as-built mixture ratios in Columns 2 and 3 were lower than design values in the lower portion of the fills, and higher than design values in the upper unsaturated portions of the fills.

The water contents of the saturated portions of the fills were lower than expected from mixture inputs. Higher tailings solids contents (lower mixture ratios) did not correspond to a proportional increase in mixture water content. Instead, water contents of as-placed mixtures suggest a loss of water from the tailings slurry during the mixing process.

Movement of particles within the column fills was found to be limited to consolidation settlements. The particle size distributions of samples taken during deconstruction of Column 1 indicate that larger particles remained in the upper parts of the fills, and did not settle through the tailings. The magnetic settlement targets placed in all four columns were similar in size to waste rock particles, and did not appear to change in orientation,
dip, strike, etc., from time of placement to time of excavation, approximately two years later. Observation of ponding in Column 1 did not indicate upward particle movement due to surface drainage of excess pore-water pressures. The maximum pressure observed in Column 1 was 80 kPa at 1 m elevation. The resulting upward gradient was 1.8, calculated from a change of pressure of approximately 80 kPa over 4.53 m. Drainage from the columns was not observed to contain major quantities of tailings particles, which were bright red and easily identifiable.

In terms of the particle model presented in Section 3.4, the lower, saturated portions of the column fills had initial structures near the "just filled" ideal mixture ratio, but were slightly tailings dominated. In contrast, the upper, unsaturated portions of Columns 2 and 3 had rock-dominated structures with tailings particles, water and air occupying the waste rock void space. Particle size distributions, water contents, as-built mixture ratios, and particle structures were examined to provide a quantitative description of particle structure in terms of waste rock skeleton void ratio $e_r$, tailings matrix void ratio, $e_t$, and total void ratio, $e$.

6.2.2 Particle Size Distribution

Particle size distribution of samples of source materials for the column fills, including the waste rock stockpile, drainage sand, and CIP tailings are shown in Figure 6.1. Particle size distributions of samples of the column fills are shown in Figure 6.2, 6.3, 6.4, and 6.5. Mixture samples taken during construction and during column decommissioning are included in Figures 6.2, 6.3, and 6.4.
Figure 6.1 includes the particle size distribution of a composite of samples taken from the source waste rock stockpile. Samples taken from the waste rock stockpile used as a source for column fill construction had a maximum particle size of 150 mm with approximately 3% clay and silt sizes. The CIP tails used were typically clay to sand size, with the distributions shown in Figure 6.1 provided by the mill, and also determined by hydrometer in the laboratory. The washed drainage sand in Figure 6.1 was chosen for its high permeability and low volume change properties and was intended to act as a drainage layer rather than as a filter.
The data in Figure 6.2 confirm observations made during loading that Column 1 contained a homogeneous mixture profile. Samples taken during construction had similar particle size distributions to samples taken during decommissioning. Sample mixture ratios are included in the legend and are examined below.
Data in Figures 6.3 and 6.4 confirm observations made during column loading that Column 2 and Column 3 contained non-homogeneous mixture profiles. The profiles in Column 2 and 3 included zones of saturated mixture material in the base of each column, overlain by zones of coarser sizes coated in tailings paste. The upper portions of the fills were observed to include an air filled void space. Column 2 had a greater length of unsaturated fill compared with Column 3, as detailed in Table 6.1. Column profiles are discussed below.
Figure 6.4 Particle size distributions, Column 3.

Figure 6.5 Particle size distributions, Column 4.
The data in Figure 6.5 confirm observations made during loading of Column 4 indicating segregation of the waste rock only fill. The data and observations indicate that the waste rock profile in Column 4 contained larger sizes in the base of the column overlain by increasingly finer sizes. The sorting of the Column 4 fill was attributed to the concrete transit mixer, which typically handles materials with a top size of less than 25 mm.

6.2.3 Initial Water Contents

Averaged water contents of materials used for construction of column fills are included in Table 6.2. CIP tailings samples taken during mixture construction for Columns 1 and 2 had solids contents near 44%, while the source tailings for Column 3 had solids contents near 40%. The waste rock stockpile had higher initial water contents than samples taken during column construction. It should be noted that the stockpile was sampled prior to column construction, and may have drained and/or dried prior to use.

Table 6.2. Average initial water contents of column source materials.

<table>
<thead>
<tr>
<th>Sample</th>
<th>CIP Tailings</th>
<th>Waste Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P$ (%)</td>
<td>$w$ (%)</td>
</tr>
<tr>
<td>Column 1</td>
<td>43.7</td>
<td>128.7</td>
</tr>
<tr>
<td>Column 2</td>
<td>44.4</td>
<td>125.4</td>
</tr>
<tr>
<td>Column 3</td>
<td>39.6</td>
<td>152.5</td>
</tr>
<tr>
<td>Column 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waste rock stockpile</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Water contents of mixture samples taken during column filling and decommissioning are shown in Figure 6.6. Initial water contents of samples of the mixture fills were greater than for samples of the waste rock only fill in Column 4. Data in Figure 6.6 indicated that the mixtures underwent a change in water content from approximately 16% to less than 10% during the test. The samples taken during decommissioning of Columns 1, 2...
and 3 indicate lower water contents, and therefore drier conditions, in the upper parts of the column fills. Water contents of samples taken during construction of Column 4, the waste rock only column, increase with elevation, which is expected from the fining upward profile indicated by particle size analyses presented in Figure 6.5.

![Figure 6.6 Water contents of column samples.](image)

### 6.2.4 Mixture Ratios

Average mixture ratios for the column fills are presented in Table 6.1, and detailed data are presented in Figure 6.7. Mixture ratios were calculated from the particle size distribution data presented in Section 6.2.2 using Equation [5.1] presented in Section 5.2.3.
Data in Figure 6.7 indicate that Column 1 had a fairly uniform mixture ratio, near 5:1. Columns 2 and 3 had lower mixture ratios overlain by higher mixture ratios in the upper parts of the columns. It should be noted that the method used to calculate mixture ratios presented in Section 5.2.3 assumes that mixtures are homogeneous, and also assumes that the waste rock contributes a constant proportion of fine materials to the mixture. The apparent segregation of column fills may have introduced an error to calculated mixture ratios for samples with high mixture ratios.

The apparent segregation of mixture and waste rock materials in Columns 2 and 3 resulted in non-homogeneous profiles including saturated mixture fill overlain by cobble sizes coated in tailings paste with an air filled void space. The geometry of as-placed column profiles is illustrated in Figure 6.8. The lengths of saturated and unsaturated
material were determined from observations during loading and deconstruction, and by filling the columns with water after the intensive 100 day monitoring period.

Following the 100 day monitoring period, water was added to Columns 2 and 3 in order to estimate the length of unsaturated fill in the upper portion of the columns. The volume of water added to each column was recorded, and used with estimated values of porosity of approximately 0.3 to determine the length of unsaturated fill in the upper portions of the mixture fills. Estimates of length of unsaturated material in the top of each fill were confirmed by observations made during column deconstruction.

The initial porosity of the waste rock control fill in Column 4 was determined by adding water to the column on day 13 following column loading. The initial porosity of the waste rock fill was determined from the volume of water added to the fill, a correction for the initial water content of the rock, and a correction for total settlement during the first 13 days of the test.
Figure 6.8 shows that Column 1 contained 5.3 m of saturated, homogeneous mixture material. Column 2 contained 2.2 m of saturated mixture material overlain by 3.1 m of unsaturated cobble size material coated in tailings paste. Column 3 contained 3.8 m of saturated mixture material overlain by 1.6 m of unsaturated material. Column 4 contained 5.2 m of waste rock.
The mixture ratios of the column fills varied considerably from design values described in Chapter Four. Design mixture ratios included 7.5:1, 12.5:1, and 10:1, while as-built mixture ratios for saturated portions of column fills were near 5:1, 6:1, and 7:1, respectively. Mixture ratios for the unsaturated portions of the fills were as high as 30:1.

In order to understand the variation in mixture ratio, the volume of void space in the waste rock may be compared to the volume of source tailings slurry, as in Table 6.3. According to the particle packing theory reviewed in Chapter Three, the volume of waste rock voids in a mixture should stay constant until the volume of tailings slurry introduced was greater than the initial volume of voids for the waste rock alone. The theoretical volume of waste rock void space in each column fill was calculated by assuming a waste rock void ratio of 0.772, which was measured for the waste rock control fill in Column 4. The initial void ratio of the waste rock in Column 4 was determined from the volume of water required to fill the void space of the rock at 13 days, corrections for settlement and initial waste rock water content, and the specific gravity of the waste rock.
Table 6.3 Volume relations for column fills.

<table>
<thead>
<tr>
<th></th>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mixture ratio (by mass)</td>
<td>7.5:1</td>
<td>12.5:1</td>
<td>10:1</td>
</tr>
<tr>
<td>Rock solids (l)</td>
<td>3160</td>
<td>3310</td>
<td>3000</td>
</tr>
<tr>
<td>Rock voids (l)</td>
<td>2440</td>
<td>2560</td>
<td>2320</td>
</tr>
<tr>
<td>Rock water (l)</td>
<td>220</td>
<td>260</td>
<td>210</td>
</tr>
<tr>
<td>Tailings slurry (l)</td>
<td>1680</td>
<td>970</td>
<td>1430</td>
</tr>
<tr>
<td>Air (calculated) (l)</td>
<td>540</td>
<td>1330</td>
<td>680</td>
</tr>
<tr>
<td>Total (l)</td>
<td>5600</td>
<td>5870</td>
<td>5320</td>
</tr>
<tr>
<td>Air (calculated)</td>
<td>9.6%</td>
<td>22.7%</td>
<td>12.8%</td>
</tr>
<tr>
<td>Water content</td>
<td>16.2%</td>
<td>10.7%</td>
<td>15.5%</td>
</tr>
</tbody>
</table>

As-Placed

<table>
<thead>
<tr>
<th></th>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixture ratio (saturated)</td>
<td>5:1</td>
<td>6:1</td>
<td>7:1</td>
</tr>
<tr>
<td>Total fill initial (l)</td>
<td>3680</td>
<td>3640</td>
<td>3770</td>
</tr>
<tr>
<td>Upper unsaturated zone:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Air by volume</td>
<td>-</td>
<td>17.3%</td>
<td>8.9%</td>
</tr>
<tr>
<td>Water added (l)</td>
<td>-</td>
<td>600</td>
<td>200</td>
</tr>
<tr>
<td>Volume change* (l)</td>
<td>-</td>
<td>47</td>
<td>20</td>
</tr>
<tr>
<td>Drainage* (l)</td>
<td>-</td>
<td>77</td>
<td>154</td>
</tr>
<tr>
<td>Water content (average)</td>
<td>15.7%</td>
<td>11.8%</td>
<td>17.7%</td>
</tr>
</tbody>
</table>

* following 100 day intensive monitoring period, prior to adding water

Comparison of volumes of the waste rock voids with tailings slurry added indicated that the column mixture designs did not include enough tailings slurry to fill the voids of loosely placed waste rock. Table 6.3 indicates that the proportions of air in the as-built fills were lower than values calculated from the mixture design. Column 1 had a design air content of approximately 10% by volume, but no air voids were observed in the placed fill. Columns 2 and 3 had design air contents of 22.7% and 12.8%, and as-built air contents measured at 17.3% and 8.9%, respectively. As an explanation, it is noted that the volume of material produced during construction was greater than the volume required to fill the columns, and that excess materials remaining in the mixer truck after filling likely consisted of unsaturated cobble sizes coated in tailings paste. Leaving
coarser, unsaturated mixture materials in the concrete mixer truck had the effect of reducing the mixture ratio and proportion of air in the as-placed mixtures. Segregation of the Column 2 and 3 mixture fills is therefore attributed to mixture design and action of the concrete transit mixer.

Mixture handling and segregation in the concrete transit mixer appeared to be dependent on the relation between tailings slurry volume and waste rock void volume. Despite a large difference in design mixture ratio, the concrete transit mixer produced tailings saturated portions of the Column 1 and 2 fills at mixture ratios near 5:1 and 6:1, as indicated in Figure 6.7. Column 3 had slightly higher mixture ratios that were attributed to a lower initial tailings solids content. The fills of Columns 1 and 2 were constructed with tailings slurry at solids contents near 44%, while Column 3 was constructed with tailings near 40%, as described in Table 6.2. The concrete transit mixer acted to sort saturated finer materials from unsaturated coarser materials.

6.2.5 Particle Structure

Section 6.2.5 defines the particle structure of the column fills in terms of waste rock skeleton void ratio $e_r$, tailings matrix void ratio, $e_t$, and total void content, or porosity, $n$. Knowledge of particle structure is useful because it may be related to geotechnical behaviours such as hydraulic conductivity, compressibility, and consolidation. Structure cannot be directly measured, but must be deduced from volumetric relations, specific gravity, water content, etc. Mixture structure descriptors of $e_t$ and $e_r$ were derived using equations presented in Section 3.4 and then compared with the initial void ratios of the waste rock control column, and the source tailings.
Section 6.2.5 examines the initial structure of samples taken from the lower, saturated portions of the column fills. Where possible, mixture sample structure is related to the structure of source materials.

6.2.5.1 Waste Rock Skeleton

The initial waste rock skeleton of mixtures in the columns had closer particle packing arrangements than the initial condition of the waste-rock-only control column fill. The initial values of $e_r$ determined for mixture samples taken during column loading were, on average, slightly smaller than the initial average value of $e$ for the waste-rock-only fill. Values of $e_r$ for the mixture samples taken during construction are presented in Figure 6.9 with the average value of $e$ determined for the waste rock control fill in Column 4.

![Figure 6.9 Initial waste rock skeleton void ratios for column fills.](image)
Figure 6.9 indicates that values of $e_r$ for saturated mixtures were generally smaller than for the waste-rock-only fill. The values of $e_r$ were calculated from water content, mixture ratio, specific gravity, and with the assumption that mixture samples were initially saturated. Air contents were not measured during the testing, but the effect of air content was examined. The effect of an assumed 3% air content is included in Figure 6.9. The average initial value of void ratio, $e$, of the waste-rock-only control fill in Column 4 was determined from the volume of water required to fill the column, with corrections for initial water content and settlement.

Factors increasing values of void ratio, $e$, of waste-rock-only over values of $e_r$ for mixtures may have included greater self-weight of the mixture, and sorting of the rock particles during loading of the waste-rock-only fill. Greater self-weight may have acted to increase initial dry density and decrease $e_r$ for the mixtures. The observed “fining upwards” profile in the waste-rock-only fill of Column 4 would theoretically have a lower density than a well mixed profile because smaller particles would not fill the void space of larger particles in a sorted profile. If air is introduced to the mixture, then values of $e_r$ increase and become more similar to the initial Column 4 waste rock void ratio of 0.772. It should also be noted that value of $e$ for the waste-rock-only fill is an average, and likely varied with depth and particle size distribution.

6.2.5.2 Tailings Matrix

The initial tailings matrix particle structure of the column mixture fills had a closer packing than expected from source tailings. Values of initial mixture tailings matrix void
ratios, $e_t$'s, for saturated column samples are presented in Figure 6.10 with values of $e$ calculated for the source tailings.

![Figure 6.10 Initial tailings matrix void ratios for column fills.](image)

Values of $e_t$ were calculated from mixture water content, mixture ratio, specific gravity, and the assumption that samples were saturated. Air contents were not measured during the testing, but the effect of 3% air by volume illustrated in Figure 6.10. Entrained air would increase the value of $e_t$ for the mixtures examined.

Values of $e_t$ presented in Figure 6.10 were lower than the values of $e$ for the source tailings slurry, indicating a loss of water during construction of the mixtures. The water content of mixtures was lower than values predicted from as-built mixture ratios and
source tailings solids contents. While as-built mixtures contained an excess of tailings solids relative to the design, the mixtures did not contain a proportional excess of water. Instead, initial water contents of samples taken during construction of Column 1 had an average of 15.7%, which was slightly lower than design values of 16.2%.

6.2.5.3 Total Porosity

Initial mixture porosities are presented in Figure 6.11 with theoretical curves calculated from an assumed mixture waste rock skeleton void ratio of 0.68 and a range of initial tailings solids contents. The porosity diagram is similar to Figure 3.4, with each line representing a binary mixture with infinite particle size ratio. Initial porosity of samples taken during column construction are plotted with the assumption that samples were initially saturated. Data are also plotted to illustrate the effect of 3% air content for Column 1 samples.
Data in Figure 6.10 and 6.11 indicate that back-calculated source tailings solids contents were higher than expected, or more simply, the mixtures did not contain as much water as expected from inputs. The primary source of water in the mixtures was the input tailings slurry. One explanation for low mixture water contents is that the measurements of initial solids contents were in error. For Columns 1 and 2, the source tailing solids contents for the saturated portions of the fills were back-calculated using mixture ratio and mixture water contents to be approximately 58%, which is higher than the measured value of 44%. Similarly, the lower saturated portion of the mixture fill in Column 3 was back-calculated to have a source tailings solids content of near 49%, which was higher than the measured value of 40%. However, CIP tailings at solids content greater than 50% become too thick to handle, a condition that was not observed in the field during column mixture construction. Other explanations for lower mixture water content may
be that water was lost during the mixing process, or remained in the concrete transit mixer.

The theoretical minimum porosity curves shown in Figure 6.11 were calculated using equations presented in Section 3.4 with the assumption that the initial porosity of the waste rock in the mixture does not change with the addition of tailings slurry until the volume of slurry is greater than the volume of waste rock voids. The lines in Figure 6.11 were also calculated with the assumptions that the source tailings slurry contained no air, and that the mixture waste rock void ratio was 0.68. Calculations assuming higher initial waste rock porosities produced theoretical curves with higher mixture porosities. Similarly, introducing air caused an increase in theoretical mixture porosity.

6.2.5.4 Summary of Particle Structure
In terms of the particle model presented in Section 3.4, the saturated mixtures in the column profiles had as-built mixture designs were near the minimum “just filled” porosity and mixture ratios calculated from mixture inputs. Values of $e_t$ were lower than expected from mixture inputs, and values of $e_r$ were lower than for values of $e$ for waste rock alone.

6.3 Pore-Water Pressure Response and Total Stress Profiles
Pore-water pressure response of the column mixture profiles is described here and related to the total stress generated by the self-weight. Pore-water pressures were monitored in the fills of Columns 1, 2, and 3 for a period of 100 days following column loading. The pore-water pressure distribution of each profile was observed to decay with time and
drainage of the fills. Following the 100 day intensive monitoring period, tensiometers were installed through the column walls to measure negative pore-water pressures (or matric suctions). By the sign convention used here, negative pore-water pressures are equal to positive matric suctions.

6.3.1 Pore-water Pressure Response

Selected pore-water pressure measurements taken from vibrating wire and pneumatic piezometers and tensiometers installed in the mixture profiles are shown in Figure 6.12, 6.13 and 6.14. Self-weight loading created excess pore-water pressures in all three mixture profiles during column loading. Induced pore-water pressures then dissipated as the profiles drained and consolidated. Profiles with shorter lengths of saturated fill had lower initial pore-water pressures. Time to dissipation of excess pore-water pressures varied with mixture ratio and length of saturated fill. Calculated hydrostatic profiles are shown in Figures 6.12, 6.13, and 6.14.

Figure 6.12 shows select pore-water pressures from pneumatic piezometers and tensiometers installed in Column 1. Measurements from vibrating wire piezometers in Column 1 are not shown, but confirm the trend observed in the pneumatic piezometer data. Column 1 had an initially triangular pressure profile distribution. The maximum measured pore-water pressure in Column 1 was 80 kPa at 1 m elevation, with pore-water pressure measurements decaying with time to below 10 kPa after approximately 27 days. With further drainage, the water table dropped below the surface of the mixture material, and pore-water pressures were observed to become negative near the top of the profile.
Negative pore-water pressures measured using tensiometers are included in Figure 6.12, 6.13, and 6.14 and are discussed below.

![Figure 6.12 Summary of pore-water pressure measurements, Column 1.](image)

Figure 6.12 shows select pore-water pressure measurements from vibrating wire piezometers and tensiometers installed in Column 2. The maximum pore-water pressure measurement in Column 2 was 21 kPa at 2 m elevation, and measurements decayed to near zero kPa after approximately 23 days. The shape of the pore-water pressure distribution shown in Figure 6.13 may indicate dissipation of excess pore-water pressures during column construction, or some form of preferential flow allowing pore pressure dissipation. The greatest pore-water pressure due to self-weight was expected at the bottom of the fill, near 0.6 m elevation. However, the largest observed pressure occurred
at 2 m elevation, and may indicate a dissipation of pore-water pressure by drainage through the sand and base drain below the mixture fill.

Figure 6.13 Summary of pore-water pressure measurements, Column 2.

Figure 6.14 shows select pore-water pressure measurements from pneumatic piezometers and tensiometers installed in Column 3. The maximum measured pore-water pressure in Column 3 was 66 kPa at 2 m elevation, and measurements decayed to less than 10 kPa after approximately 1.4 days. The shape of the profile in Figure 6.14 also indicates that some dissipation of pore-water pressure may have occurred during construction of Column 3. Possible preferential flow in Column 3 may have included radial flow. For double drainage, the pattern of expected dissipation would be similar to that observed for Column 1 in Figure 6.12, with high pressures draining immediately at the base of the profile, resulting in a rounding of the profile. For radial drainage, the path of drainage
allows all elevations to drain at the same time, and may create the pattern of pore pressure response illustrated in Figure 6.14.

![Figure 6.14 Summary of pore-water pressure measurements, Column 3.](image)

The pneumatic piezometers in Column 1 and Column 3 were accurate to within ± 0.4 kPa, with a manufacturer’s recommended lower range of 20 kPa. In use, readings as low as 10 kPa were considered reliable. The manufacturer’s stated accuracy for the vibrating wire piezometers installed in Column 2 was ±10 kPa, with readings subject to a temperature calibration. Measurements of temperature at the surface of the mixture fills varied between 14°C and 18 °C during most of the 100 day testing period. The operating temperature was assumed constant at 16°C, introducing an additional error of less than ±1 kPa. Piezometer readings were also taken at the same time of day to minimize error due to thermal variations. The plotted elevations of piezometers in Figure 6.8 are initial
positions calculated from the final excavated position of each piezometer tip with a correction for settlement.

6.3.2 Total Stress Profiles

Total stress and pore-water pressure profiles in the column fills are useful for understanding the response to self-weight. Figures 6.15, 6.16, 6.17, and 6.18 include initial and final pore-water pressure measurements, designated as "u," and also theoretical "Total Stress" profiles calculated from self-weight and the length of each fill. Final pore-water pressure measurements were taken just prior to column deconstruction. Tensiometers were installed through the column walls to measure soil matric suctions, labelled as "u final" in Figures 6.15, 6.16, and 6.17. It should be noted that Columns 1, 2 and 3 were flooded and allowed to drain six months prior to deconstruction and final measurement of negative pore-water pressure profiles. The Columns were flooded to bring the columns to a comparable condition. Several attempts to fix the seals at the base of Columns 2 and 3 had involved flooding of individual columns. Flooding all three columns allowed six months of drainage, or similar conditions favourable for comparison of individual behaviours between the three fills.
Data in Figure 6.15 indicate that the magnitude of the initial pore-water pressure distribution induced by self weight of the fill in Column 1 was lower than expected for a fluidized fill by an average of approximately 8 kPa. Pore pressures measurements in Figure 6.15 are initial readings from vibrating wire piezometer measurements, with the assumption of zero pore-water pressure at the elevation of the base drain invert, and at the column surface. All pressures are stated relative to atmospheric pressure.

The pore-water pressure in Column 1 was near zero after approximately 70 days. The water table was observed to coincide with the surface of the fill at 85 days, after which time the pore-water pressures became negative from the top down. Figure 6.15 includes final pore pressure measurements from tensiometers indicating a near hydrostatic distribution. Measurements of negative pore-water pressures near the surface of the fill
were slightly lower than the hydrostatic distribution, likely due to evaporation at the surface of the profile.

![Graph showing total stress profile for Column 2.](image)

**Figure 6.16** Total stress profile for Column 2.

The total stress profile calculated from the density of the Column 2 fill is plotted in Figure 6.16 with initial and final pore-water pressure measurements. The magnitude of the initial pore-water pressure distribution induced by self-weight of the fill in Column 2 was much lower than expected. However, Column 2 took approximately 20 days to dissipate excess pore-water pressures, indicating a low value of hydraulic conductivity. The final pore pressure distribution in Column 2 was near hydrostatic, with lower pore pressures near the surface of the fill indicating evaporation at the profile surface.
The total stress profile for the Column 3 is plotted in Figure 6.17 with initial and final pore-water pressure measurements. The magnitude of the initial pore-water pressure distribution induced by self-weight of the fill in Column 3 was slightly lower than expected, indicating entrained air. Drainage of pore-water pressures in Column 3 occurred within approximately 3 days of filling, and indicating a higher value of hydraulic conductivity compared to Columns 1 and 2. The final pore pressure distribution in Column 3 varied slightly from hydrostatic, likely due to measurement error and surface drying.
The total stress and final pore-water pressure profile in Column 4 are included in Figure 6.18. The total stress was calculated from initial bulk density and includes the mass of the near 3% water content. The "u hydrostatic" was calculated based on the length of saturated profile and the unit weight of water. Column 4 was flooded and remained saturated for the majority of the test, producing a water pressure profile similar to the one illustrated in Figure 6.18. Initially, Column 4 was unsaturated, but was flooded on day 13 of the test, and remained saturated for the rest of the two-year test.

6.4 Drainage

Selected measurements of drainage from the column fills are included in Figures 6.19, 6.20, and 6.21. All three mixture profiles followed a pattern of drainage including a phase of double drainage associated with high internal pore-water pressures followed by
a phase of single, or downward, drainage associated with lower pore-water pressures. Water was expelled from both the top and bottom of the saturated portion of each profile during the double drainage phase due to hydraulic gradients created by self-weight loading. The transition from double drainage to downward drainage occurred when pore-water pressures had dissipated enough that the downward hydraulic gradient due to gravity exceeded the upward hydraulic gradient induced by self weight loading. The process is similar to that described by Terzaghi (1943) for placement of hydraulic fills with a pumped underdrain.

![Diagram](image)

Figure 6.19 Summary of drainage measurements, Column 1.

Surface drainage was observable as ponding on Column 1 and is plotted in Figure 6.19. The volume of ponded water in Column 1 reached a maximum of approximately 1% of the volume of the fill at 20 days, marking the end of the double drainage phase. The transition from double drainage to downward drainage can also be seen as a point of
inflection on the plot of volume change at day 20 in Figure 6.19. During the downward drainage phase, ponded water re-entered the profile. Observations indicated no ponded water after approximately 85 days. The error in the water balance was -6% at 20 days, and -12% at 85 days, with the negative sign indicating that less water was collected than expected. Errors in the water balance are attributed to leaks, measurement errors, expansion of the column wall, and also to evaporation due to air flow under the column lid. Several minor leaks were observed and patched during the early stages of the test for the mixture columns. It should also be noted that the apparent error is reduced by water expelled during compression and consolidation of the sand and lower base mixture layers.

Figure 6.20 Summary of drainage measurements, Column 2.
No ponding was observed for Column 2 or Column 3 due to the unsaturated zones in the upper part of the profiles. However, the difference between the volume of water reporting to the base drain and the volume change of the fill indicates a double drainage phase, shown in Figures 6.20 and 6.21. Water expelled from the upper surfaces of the saturated zones of the profiles was likely stored within the void space of the upper unsaturated portions of the profiles. Based on data shown in Figures 6.20 and 6.21, the double drainage phases are estimated to have lasted for approximately 2.6 days and 0.5 days for Column 2 and Column 3, respectively. In each case, the end of the double drainage phase is taken as the maximum difference between the volume change of the fill and the volume of water reporting to the base drain, which corresponds to the greatest amount of water stored, or "ponded" above the saturated fill. Once the double drainage phase ended, the start of the single phase of drainage occurs as "ponded" water re-enters
the fill to exit the base drain. The maximum volume of “ponded” water therefore coincides with the transition from double to single drainage.

Volume change equalled volume of water reporting to the base drain at approximately 7 days for Column 2 and Column 3. The desaturation of the upper cobble portions of the Column 2 and Column 3 profiles likely contributed water to the base drains and reduced the time required for the volume of water reporting to the base drain to equal volume change of the fill. Total water collected from the base drains during column loading is not shown in Figure 6 and included 17.5 litres from Column 1, 4.5 litres from Column 2, and 30.5 litres from Column 3. The minor volumes of drainage collected during the loading period (eight hours for Column 1, four hours each for Columns 2 and 3) were attributed to compression of the sand layer, drainage of excess water on top of the sand layer, and also consolidation of the lower parts of the mixture profiles. A small volume of water exited the base drain of Column 4 during column loading, and was attributed to drainage of excess water initially on top of the sand layer, and also to compression of the sand drainage layer.

6.5 Settlement

The majority of settlement in the mixture columns occurred during dissipation of excess pore-water pressures, with additional volume change attributed to shrinkage associated with negative pore-water pressures and mechanisms of secondary settlement. Total settlements at 100 days and at the end of test are included in Table 6.1 as percent change in length relative to the initial length of fill above the drainage sand for each profile. Differential settlements of each column fill profile are plotted in Figures 6.22, 6.23, 6.24,
and 6.25. Total settlement for each mixture profile was less than 5% after 100 days and less than 6% at the time of column decommissioning. The majority of settlement occurred in the upper parts of each profile, with the exception of Column 2. The magnitude of settlement in Column 2 was approximately half that of the other mixture profiles, and is attributed to a shorter length of saturated fill. Total settlement for Column 4 with waste rock only was 7.4% after 100 days, and 9.6% at the end of the test. It should be noted that the base drain of Column 4 was closed and the profile was flooded with water after 13 days, at which time 5.6% total settlement had occurred.

Displacements of the surface and magnetic targets buried within the Column 1 fill relative to the top of the column are shown in Figure 6.22. The majority of settlement in
Column 1 occurred in the upper 3.8 m, with more than double the degree of settlement measured for the lower 1.5 m of the profile.

Displacements of the surface and magnetic targets buried within the Column 2 fill relative to the top of the column are shown in Figure 6.23. The settlement in Column 2 appears to have occurred relatively evenly, with slightly less settlement occurring in the mid-upper unsaturated portion of the profile, as shown in Figure 6.23.
Displacements of the surface and magnetic targets buried within the Column 3 fill relative to the top of the column are shown in Figure 6.24. The majority of settlement in Column 3 occurred in the middle of the profile, which had more than double the percentage of settlement calculated for the lower 1.5 m of the profile.
Displacements of the surface and magnetic targets buried within the Column 4 fill relative to the top of the column are shown in Figure 6.25. The majority of settlement in Column 4 occurred in the middle and upper parts of the profile, followed by the upper portion of the profile, and finally by the lower portion of the profile.

The magnetic settlement system worked reasonably well, but did produce a few anomalies, such as the apparent oscillation of the targets at 4 m in Columns 1, 2, and 3. The 5 m magnetic target placed in Column 1 moved laterally during column loading, and could not be detected from the probe guide tube. Errors in settlement readings are attributed mainly to thermal expansion and contraction of the columns, but also to movement of the probe guide tube on the outer column wall. The column walls were observed to change in length with the daily thermal cycle by as much as 5 mm. All
settlement measurements were referenced to the top of the columns, and the volume change estimates included in Figure 6.19, 6.20, and 6.21 are based on these settlement measurements. Thermal expansion and contraction of the column walls (mentioned below in Section 6.9) introduces an error of approximately ± 5 mm into settlement readings, and ± 3.4 litres to volume change estimates. The lower portions of the probe guide tubes were trimmed to bring them flush with the column walls after the columns had been filled. Trimming should have most influence on the 0.5 m magnetic target readings. The guide tubes were also observed to show some thermal expansion that caused a small amount of curvature in alignment.

6.6 Analysis of Consolidation

According to conventional consolidation theory, the degree of compression should increase with effective stress. The stress profiles presented in Section 6.3.2 indicate increasing effective stress with depth, and it was expected that the settlement in the column profiles would be greatest at the base of each column. However, the differential settlement measurements of the column fills presented in Section 6.5 indicated less settlement in the lower parts of the profiles, and more settlement in the upper parts of the column profiles. The smaller-than-expected observed settlements in the lower parts of the profiles may have been due to consolidation occurring during column loading, or may been due to column wall effects, which are discussed below. Ignoring loading schedules and column wall effects, consolidation parameters were determined, and discussed below.
The pattern of drainage and volume change of column fills was complex, and was divided into three phases for analysis, including:

i) double drainage phase,

ii) single drainage phase, and

iii) phase of development of negative pore pressures.

The phases of drainage were defined primarily by drainage pattern, but also by pore-water pressure response. The double drainage phase had high excess internal pore-water pressure created by self-weight, and drainage of water from the top and bottom of the saturated fill. The single drainage phase had lower excess pore-water pressures, and drainage of water from the base drain only. The latter phase included development of negative pore-water pressures from the top down as the water table dropped below the surface of the saturated portion of a profile, but also included drainage from the base drain only. The phases of drainage were examined independently.

The double drainage phase was examined using conventional consolidation theory, and also by numerical simulation with a finite element model. The single drainage phase was examined by closed form solution, as well as a finite element seepage model. The development of negative pore pressures was examined both qualitatively and quantitatively. The following analyses focus primarily on Column 1, which had the greatest length of saturated homogeneous fill and the most complete data set. Degree of settlement for each phase is examined first.
6.6.1 Degree of Settlement

The times and degree of total settlement occurring during each phase of consolidation in the column fills is shown in Table 6.4.

Table 6.4 Summary of settlement in column profiles.

<table>
<thead>
<tr>
<th>Column</th>
<th>Time (days)</th>
<th>Strain (%)</th>
<th>Proportion (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 to 20</td>
<td>3.5</td>
<td>59.9</td>
</tr>
<tr>
<td></td>
<td>20 to 85</td>
<td>1.0</td>
<td>16.7</td>
</tr>
<tr>
<td></td>
<td>85 to 700</td>
<td>1.4</td>
<td>23.4</td>
</tr>
<tr>
<td></td>
<td>700</td>
<td>5.9</td>
<td>Total</td>
</tr>
<tr>
<td>2</td>
<td>2.6</td>
<td>0.8</td>
<td>22.3</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>0.3</td>
<td>8.5</td>
</tr>
<tr>
<td></td>
<td>700</td>
<td>2.5</td>
<td>69.1</td>
</tr>
<tr>
<td>3</td>
<td>0.5</td>
<td>2.2</td>
<td>36.6</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>1.2</td>
<td>19.3</td>
</tr>
<tr>
<td></td>
<td>700</td>
<td>2.7</td>
<td>44.0</td>
</tr>
<tr>
<td>4</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>700</td>
<td>9.7</td>
<td>Total</td>
</tr>
</tbody>
</table>

Approximately 77% of settlement observed in the Column 1 profile occurred during drainage of excess pore-water pressures. In contrast, approximately 69% of settlement observed in the Column 2 profile occurred during the phase of negative pore-water pressures. Approximately 56% of settlement observed in the Column 3 profile occurred during the drainage of excess pore-water pressures. It should be noted that the Column 1 profile was initially fully saturated and the phases of drainage were readily distinguishable. However, Columns 2 and 3 contained saturated portions overlain by
unsaturated portions, and the difference between phases of drainage was less distinct. The unsaturated portions of the upper profiles of Columns 2 and 3 immediately began to drain due to gravity induced gradients after placement. Consequently, water collecting on top of the saturated portions of the Column 2 and 3 profiles was due to gravity induced downward flow from the unsaturated portions of the profiles, and also to self-weight induced upward flow out of the saturated portions of the profiles. The delay of pore pressure dissipation in Column 2 may be partly due to the addition of water draining from the upper unsaturated parts of the fills. Table 6.1 lists the time to pore pressure dissipation as 23 days for the Column 2 profile, while time to completion of single drainage included in Table 6.4 is listed as 7 days. Values listed in Table 6.4 were determined from drainage data in attempt to quantify consolidation parameters of the lower saturated portions of each profile. The waste rock only profile in Column 4 was not observed to develop excess pore-water pressures during settlement.

6.6.2 Phase of Double Drainage

Immediately follow column construction, the fills were observed to develop pore-water pressures due to self-weight. Drainage and associated dissipation of excess pore pressures resulted in water exiting from surface of the Column 1 profile as well as the base drain.

6.6.2.1 Numerical Analysis by Terzaghi’s Method

Conventional consolidation theory was applied to the double drainage phase of the Column 1 fill. Conventional theory implies that the progress of consolidation corresponds to the dissipation of excess pore-water pressures. Double drainage conditions imply an effective specimen height equal to one half of the height of the
profile of mixture material in Column 1. For the present analysis, the effects of secondary consolidation, or creep, occurring during the double drainage phase is ignored. In contrast to Terzaghi’s (1943) approach, the final pore pressure profile is assumed to be 0 kPa through the entire length of the fill.

Based on conventional analysis of dissipation of pore-water pressures from a double drained layer (Terzaghi (1943), Lambe and Whitman (1969)), the average coefficient of consolidation, \(c_v\), of the Column 1 fill was \(8.8 \times 10^{-6} \text{ m}^2/\text{s}\), coefficient of volume change, \(m_v\), equal to \(8.5 \times 10^{-4} (1/\text{kPa})\), and hydraulic conductivity, \(k\), was \(3.9 \times 10^{-8} \text{ m/s}\). The analysis assumes an initial maximum pore-water pressure of 93 kPa at the base of the fill, with all excess pore-water pressures equal to zero at 70 days.

In his treatment of hydraulic fills, Terzaghi (1943) assumed that no water exited the surface of the fill, and also allowed pore-water pressures to decay below zero kPa. The Column 1 fill was observed to have ponded water on surface until approximately 85 days, taking approximately 15 days to drain ponded water. The presence of ponded water prevents application of the solution presented by Terzaghi (1943), and a finite element model was therefore selected to confirm consolidation parameters of hydraulic conductivity and coefficient of volume change.

6.6.2.2 Finite Element Modeling

The consolidation of the Column 1 fill was modeled for first for settlement only to confirm a coefficient of volume change, \(m_v\). Next, the consolidation of the Column 1 fill was modeled for seepage and settlement to confirm a value of hydraulic conductivity, \(k\),
for the double drainage phase. Settlement-only modeling was done using SIGMAW V5.20, and seepage-settlement modelling was done using SEEPW V5.20 and SIGMAW V5.20, a commercially available stress-strain finite element modeling package.

Inputs for the settlement-only model included an axi-symmetric mesh geometry described in Section 4.5 and a body load of 21.7 kPa/m. Boundary conditions included a horizontally constrained lateral boundary representing the column wall, and a horizontal and vertically constrained base. The mixture was modelled as a linear elastic material with a Young's Modulus, \( E \), of 785 kPa, and Poisson's ratio, \( v \), of 1/3. The value of \( E \) was determined from the value of \( m_v \) determined in Section 6.6.2.1 using [6.3]:

\[
[6.3] \quad E = \frac{(1 + v) (1 - 2v)}{(m_v^* (1 - v))}
\]

Load deformation analysis (with zero pore-water pressures) resulted in total surface settlements equal to 4.9%, which were similar in magnitude to that observed for the Column 1 profile during the drainage of excess pore-water pressures (the stress-strain analysis assumes a static condition of zero excess pore pressures). However, the model indicated a linear strain distribution of near zero at surface of the profile, increasing with depth to a maximum strain of 9.8% near the base of the fill. Measured displacements of magnetic targets in Figure 6.22 indicated that the lower portion of the Column 1 fill had a lower degree of settlement than the upper portion of the fill. While a linear elastic model provides a reasonable estimate of the magnitude of volume change, predicted differential settlements within the model profile were not representative of measured settlement patterns.
In a more complex simulation of the double drainage phase behaviour, the Column 1 fill was modelled using a fully coupled seepage-stress-strain analysis of consolidation. The finite element mesh geometry used is described in Section 4.5. Initial heads for the simulation were generated with SeepW for the same mesh with boundary conditions of pore-water pressure equal to zero at the column surface, and head equal to the elevation of the drain invert at the base of the profile. Values of parameters for the coupled SeepW model included a saturated hydraulic conductivity, $k$, of $3.9 \times 10^{-8}$ m/s with a value of the slope of the SWCC at zero kPa pressure of $3.27 \times 10^{-4}$ (1/kPa). Boundary conditions for the SEEPW portion of the coupled SEEPW-SIGMAW analysis included a no-flow lateral boundary representing the column walls, an upper boundary or surface condition of head equal to elevation, and a lower boundary condition of head equal to elevation of the base drain invert. The SIGMAW component of the coupled model used the same properties, boundary conditions, and body load as the settlement-only analysis described above. The coupled consolidation model was run for approximately 100 days.

Output data from the coupled consolidation model examined here includes settlement and pore-water pressure response. The pattern and degree of settlement in the coupled model was similar to the response of the settlement-only analysis described above. Pore-water pressures generated in the coupled consolidation model are shown in Figure 6.26.
The maximum pore-water pressures predicted by the model were near 110 kPa, which was higher than measurements from the Column 1 profile. The coupled model for Column 1 followed a similar pattern of pore-water pressure dissipation observed in the field study, presented in Figure 6.12. The models serve to confirm the values of mixture hydraulic conductivity and coefficient of volume change determined by analytical and laboratory methods.

6.6.3 Phase of Single Drainage

Following the double drainage phase, the water ponded on the surface of Column 1 began to drain back through the fill to the base drain. Ignoring internal pore-water pressures, the process was analogous to a falling head test. The values of $k$ for mixture materials were determined from drainage of ponded water assuming no excess internal pore-water pressures, and are presented both in Table 6.1 above, and in Table 6.5 below. The phase
of single drainage for Column 1 was also modeled using SEEPW. Results are discussed below in Section 6.6.5.

Equation [6.4] was used to determining the hydraulic conductivity of the column profiles from the time and volume of downward drainage.

\[ k = \frac{(a L \ln(h_2 / h_1))}{(A \, dt)}, \]

where \( a \) is area of the standpipe, \( L \) is the length of column fill, \( h_1 \) and \( h_2 \) are heads on the column at times \( t_1 \) and \( t_2 \), respectively, \( A \) is the area of the specimen, and \( L \) is the length of saturated profile. For Column 1, \( a \) and \( A \) were equal.

The phase of single drainage was also modelled using SEEPW for Column 1 using the finite element mesh described in Section 4.5 with assumptions of no deformation and no initial excess pore-water pressure distribution. Boundary conditions included a no-flow boundary at the sides representing the column walls, a lower boundary condition of head equal to the elevation of the drain invert, and an upper boundary condition of a head versus volume function representing ponded storage above the profile. The hydraulic conductivity of the fill material was assumed to be \( 3.9 \times 10^{-8} \) m/s. The time for ponded water to drain into the fill was approximately 20 days, which was shorter than the 65 days of observed drainage of ponded water for the surface of Column 1. Differences in time of drainage between the model and the column may be due to a lower/higher hydraulic conductivity, evaporation during the drainage period from the surface of the fill.
the same model with a value of saturated hydraulic conductivity to $1.2 \times 10^{-8}$ m/s resulted in a time to drainage of ponded water of between 65 and 70 days.

6.6.4 Phase of Negative Pore-Water Pressures

As stated in Table 6.4, the degree of volume change occurring during the phase of negative pore-water pressures accounted for 23% of total settlement for Column 1, and near 69% and 44% for Column 2 and 3 profiles, respectively. Volume change due to the development of negative pore-water pressures was therefore significant. The proportion and degree of total settlement during the phase of negative pore-water pressures appear to correlate with the proportion of unsaturated material in the Column profiles. Part of the settlement observed during the phase of negative pore-water pressures was attributed to the development of matric suctions and also to creep, or secondary settlement in the Column profiles. It is not likely that the drainage sand and lower mixture layer desaturated during the 100 day monitoring period, but partial desaturation may have occurred during later stages of the test.

6.6.5 Summary of Column Consolidation Parameters

Mixture consolidation parameters from Column 1 are summarized in Table 6.5.
Table 6.5 Summary of column consolidation parameters.

<table>
<thead>
<tr>
<th>Column</th>
<th>Mixture Ratio</th>
<th>( k ) consolidation (m/s)</th>
<th>( k ) drainage (m/s)</th>
<th>( c_v ) (m(^2)/s)</th>
<th>( m_v ) (1/kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5:1</td>
<td>3.9 \times 10^{-8}</td>
<td>1.1 \times 10^{-8}</td>
<td>8.8 \times 10^{-6}</td>
<td>8.5 \times 10^{-4}</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.2 \times 10^{-8}†</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>6:1</td>
<td></td>
<td>5.4 \times 10^{-8}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>7:1</td>
<td></td>
<td>1.3 \times 10^{-7}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1:0</td>
<td></td>
<td></td>
<td>0.3††</td>
<td></td>
</tr>
</tbody>
</table>

† SeepW simulation, ††constant head test

Examining data in Table 6.5, mixture profiles with higher mixture ratios had higher values of \( k \). The mixture in Column 3 had higher values of \( k \) and mixture ratio than the mixture in Columns 1 and 2, and this may also be due to the higher initial water content of the source tailings. The highest value of \( k \) for waste rock was measured by constant head test to be 0.3 m/s under a hydraulic gradient of 0.002, after 13 days and 5.6% settlement. The values of \( k \) for mixtures were found to range between 1 \times 10^{-8} m/s and 1 \times 10^{-7} m/s. For Column 1, the value of \( k \) determined by analysis of consolidation, 3.9 \times 10^{-8} m/s, was higher than that estimated from drainage of ponded water, 1.1 \times 10^{-8} m/s. The difference was expected due to excess pore pressures remaining in the Column 1 profile following the double drainage phase. Excess pore-water pressures remaining in the profiles after double drainage may have reduced the gradient and the values of mixture \( k \) in Table 6.5 determined from drainage of ponded water may be lower than actual values.
6.6.6 Column Loading Schedule Effects

By consolidation theory, effective stress should increase with depth, and more consolidation was expected in the lower parts of the profiles. As noted above, the majority of settlement in column fills occurred in the upper parts of each profile, with the exception of Column 2. One possible explanation for less-than-expected settlement in the lower profiles is that consolidation occurred during column loading. For Column 1, the volume of drainage collected during column loading was approximately 18 litres, and was made up of water draining from and overlying the drainage sand and lower mixture filler layer, as well as from consolidation of the mixture profile immediately overlying the drainage sand. Loading of Column 1 took approximately eight hours. Assuming that the maximum rate of consolidation due to self-weight would occur immediately after filling, the maximum rate of settlement can be determined from initial settlement data. The amount of settlement in the eight hours immediately following loading of Column 1 was equivalent to a volume of 18 litres. It is expected that a lower rate of consolidation would have occurred during loading because less self-weight would be available to drive consolidation. Therefore, the volume of water produced prior to the start of monitoring is not accountable solely by volume change during column loading. However, it is likely that the lower portion of the Column 1 fill did undergo some settlement during the loading period, and resulted in lower measured strains for the lower parts of the profiles in the later parts of the test. Columns 2, 3, and 4 are also assumed to have undergone some consolidation during column loading.
6.7 State of Water Saturation and Matric Suctions

The state of water saturation in the column profiles was investigated by direct observation of soil conditions, examination of drainage and volume change data, and by in-situ matric suction measurements. The zones of the mixture profiles that were initially saturated with tailings slurry were found to remain water saturated during the intensive 100 day monitoring period. The waste rock only profile in Column 4 remained unsaturated until flooded at 13 days. Desaturation in Columns 2 and 3 was attributed to the upper unsaturated zones of those profiles rather than the lower zones of mixture material. Observations made following the 100 day monitoring period include cracking and a lighter colour at the surface of the Column 1 mixture profile, as well as crusting of the coating of tailings paste on the larger rock particles at the surface of Columns 2 and 3. Surface drying of the profiles was attributed to air flow under the column lids. Comparison of the volume of drainage and the volume of settlement in Figure 6.19 indicate that the mixture in Column 1 did not undergo major desaturation during the initial 100 days following column loading. Data in Figures 6.20 and 6.21 indicate that desaturation of the profiles in Columns 2 and 3 did occur, with volume change equal to 73% and 59% of volume of drainage at 100 days in Columns 2 and 3, respectively.

Tensiometer measurements taken after day 100 of the test are included in Figures 6.12, 6.13, and 6.14. At 125 days the pore-water pressure values in Column 1 were measured at −14 kPa near the surface, and approximately 0 kPa in the lower part of the profile, shown in Figure 6.12. Lower values of pore pressures likely occurred at the upper surface layer of Column 1, but the layer was too thin to isolate with tensiometers. It
should be noted that Column 1 was saturated with ponded water at the surface of the fill until day 85 of the test, leaving approximately 40 days of free drainage without water on surface and thus the appropriate conditions to generate the negative pore-water pressures reported here. At 120 days, the pore-water pressures in Column 2 were measured at −46 kPa near the surface, increasing to −19 kPa at 5 m elevation and approximately 0 kPa at 0.6 m elevation near the base of the profile, as shown in Figure 6.13. At 116 days, the pore-water pressures in the Column 3 profile were measured to be −54 kPa at the surface and greater than −10 kPa in the lower portion of the profile, as shown in Figure 6.14.

Final tensiometer measurements are included in Figures 6.15, 6.16, and 6.17. Observations made during column deconstruction confirm that cracking related to shrinkage of the tailings matrix did occur in the upper portions of the Column 1, 2, and 3 fills. It should be noted that Columns 1, 2, and 3 were filled with water and then allowed to drain for approximately six months prior to final tensiometer measurements. The final negative pore-water pressures therefore developed over six months of free drainage, without access to rainfall or moisture.

The estimated lengths of the mixture profiles that remained continuously saturated over the 100 day intensive monitoring period were 5.3 m, 2.3 m, and 4.5 m, for Columns 1, 2, and 3, respectively. Negative pore-water pressure measurements are compared to laboratory derived SWCC’s in Chapter Seven.
6.8 Final Density Measurements
Attempts to calculate the density of samples taken during decommissioning were unsuccessful due to significant error in measurements of sample volume by the method described in Chapter Four. Consequently, the data are not presented here.

6.9 Pressure Plate Data and Column Wall Movement
Data collected from the pressure plates installed on the sand layer in Column 1 are presented in Figure 6.27.

![Figure 6.27 Pressure plate data from Column 1.](image)

The data in Figure 6.27 show considerable scatter, but generally indicate a decrease in pressure with time. The maximum pressures measured during the initial stages of the test were 86 kPa and 82 kPa for the centre and edge plates, respectively. The decrease in measured pressure at the base of the column fill with time may indicate wall effects, or
perhaps bridging of the fill over the plates. The plates were placed near the centre and near the edge of the column. It may be that the column walls acted to laterally support the lower portion of the fills.

The columns were observed to expand and contract with the daily thermal cycle. Measurements from the top of Column 1 to ponded water on the surface of the fill were noted to vary by several millimetres with the time of day. The cause of the variation was likely differential expansion of the HDPE pipe due to solar heating. A typical daily thermal cycle included cool nights, followed by sun and warming in the morning and early afternoon, followed by cooling and rain by mid afternoon. Nightly minimum air temperatures were noted to be as low as 12°C, while maximum daily air temperatures were as high as 27°C. Column wall expansion was measured in an effort to quantify thermal effects. From approximately 6:45 am to 10:30 am, column walls expanded by an average of 2.5 mm in length. Measurements of outer column wall surface temperature varied from 14°C to as high as 32°C in direct sunlight. The average change in column wall surface temperature was approximately 9°C. The column fills at surface maintained an average temperature of between 14°C and 16.5°C. The coefficient of thermal expansion for HDPE pipe is approximately 120 x 10⁻⁶ m/m°C. Using the coefficient of thermal expansion, a change of 9°C is calculated to change a 6 m HDPE column wall by 6.5 mm. The upper 1.5 m of the columns facing a northern aspect had most exposure to sunlight, and it is expected that the columns would not expand evenly in length, but would curve slightly with thermal expansion due to solar heating. The measurements and
estimates of change in column wall length during part of a daily thermal cycle indicate that column wall expansion and contraction may have affected the column fills.

Pressure plate data from Column 1 are plotted in Figures 6.28 and 6.29 for the centre and edge plates, respectively. The data are divided into morning and afternoon readings.

Figure 6.28 Centre pressure plate data from Column 1.
Data in Figures 6.28 and 6.29 indicate lower measured pressures in the afternoon, as compared to the morning, for both the edge and centre pressure plates. The apparent decrease in pressure in the afternoon may be due to column wall effects. Hoop stresses in the column walls were not measured in the experiment. As mentioned, data indicate that the column walls were subject to expansion and contraction due to the daily thermal cycle. The effect of expansion and contraction of the column walls on the column fills is unknown, but support by the column walls could act to reduce total stress experienced in the lower parts of the profiles, and may provide an additional reason for the lower degree of consolidation in the lower part of the column fills. Ultimately, the significance and reliability of pressure plate data are questionable, and column wall effects were noted, but ignored for analysis.
6.10 Summary

Data and analysis from the meso-scale column study are presented in Chapter Six. Column profile composition and initial structures were related to the particle model presented in Section 3.4. Column 1 had a homogeneous profile with a structure near the "just filled" optimum, but slightly tailings dominated. Columns 2 and 3 had upper, unsaturated zones containing cobble coated in tailings paste, and lower, saturated zones of slightly tailings dominated material. Pore-water pressure response, drainage, and settlement of the column fills were presented, and indicate that the saturated portions of the column fills acted as hydraulically placed fills. Saturated portions of the fills underwent phases of double drainage, followed by a phase of single drainage and the development of negative pore-water pressures. Values of consolidation parameters were determined by closed form solution, and verified with a finite element model. State of water saturation and matric suction data were also presented. Column data and the results of analyses are compared with laboratory test data in Chapter Seven.
CHAPTER SEVEN. ANALYSIS

7.1 Introduction

Chapter Seven presents analyses and comparisons of laboratory and column study results. Specimen structure, volume change behaviour, hydraulic conductivity, and unsaturated behaviours are evaluated. A relation for hydraulic conductivity of mixtures is proposed, and a method for predicting volume change is also presented.

7.2 Initial Specimen Structure

Initial waste rock skeleton void ratios from laboratory and column specimens are plotted in Figure 7.1, and initial tailings matrix void ratios are plotted in Figure 7.2. The term "initial" indicates as-placed void ratios, prior to consolidation and settlement. Specimen names refer to test type and mixture ratio. Laboratory specimens are listed in Table 5.1.
Figure 7.1 Initial waste rock skeleton void ratio for laboratory and column specimens.

Figures 7.1, 7.2, and 7.3 were created from a spreadsheet by assuming a range of mixture ratios, fixed tailings solids contents, and fixed waste rock water contents using equations presented in Section 3.4. Assumed values are labelled on the Figures, and are detailed below. Figures 7.1, 7.2 and 7.3 are similar to Figure 3.4, but show porosity with respect to $e_r$, $e_t$, and $n$, rather than $n$ alone.

In general, laboratory mixture specimens had higher initial waste rock skeleton void ratios, $e_r$'s, than column specimens, as illustrated in Figure 7.1. The initial value of $e$ for the waste rock only compressibility specimen (labelled Rock Compressibility in Figure 7.1) was slightly lower than that of the waste rock in Column 4, both of which are also included in Figure 7.1. Laboratory mixture specimens had initial values of $e_r$ that were
greater than $e$ for the waste rock compressibility specimen, while the column mixture specimens generally had values of $e_r$ that were lower than the initial value of $e$ for the waste rock in Column 4. Figure 7.1 includes $e_r$ versus $R$ relations predicted for a mixture constructed of source waste rock with an initial $e$ of 0.7, water content of 1%, combined with source tailings at 40%, 50%, and 60% solids. Accounting for the source tailings solids content for each mixture presented in Tables 5.2 and 6.2, the laboratory and column study specimens fall to the left of the predicted relations, indicating mixture ratios that were slightly lower than the "just filled" optimum. If the initial source waste rock void ratio used to predict the theoretical curves in Figure 7.1 is increased above 0.7, then the predicted $e_r$ increases proportionally.

Figure 7.2 Tailings matrix void ratio for laboratory and column specimens.
Initial tailings matrix void ratios, $e_t$'s, for column specimens were slightly lower than laboratory specimens, as shown in Figure 7.2. The laboratory specimens had values of $e_t$ that were similar to source tailings $e$, while the column specimens had a large difference between $e_t$ and source tailings $e$. Figure 7.2 includes $e_t$ versus $R$ relations predicted for mixtures constructed of source waste rock with an initial $e$ of 0.7 and a water content of 1%, and source tailings at 40%, 50%, and 60% solids content. Accounting for source tailings solids content of each mixture presented in Tables 5.2 and 6.2, the theoretical curves indicate that the column and laboratory specimens had mixture ratios that were slightly lower than the “just filled” mixture ratio (indicating an excess of tailings).

Porosities, $n$'s, of laboratory and column specimens are plotted in Figure 7.3 with respect to mixture ratio.
Values of $R$ of saturated laboratory and column specimens ranged between 4 and 6, and values of $n$ ranged between 0.28 and 0.36. Initial porosities of laboratory test specimens were slightly higher than those of specimens taken during column loading. Figure 7.3 includes predicted $e$ versus $R$ relations for mixtures constructed of source waste rock with $e$ of 0.7, and water content of 1%, and of source tailings at 40%, 50%, and 60% solids. The theoretical curves in Figure 7.3 indicate that all specimens had mixture ratios slightly lower than the "just filled" optimum.

In terms of the particle model presented in Chapter Three, specimens from the laboratory had initial structures that were slightly tailings dominated, with values of $e_f$ that were slightly higher than $e$ for waste rock alone. Mixture specimens had values of $R$ that were slightly lower than the "just filled" optimum, while waste rock only and tailings only
specimens correspond to maximum and minimum values of mixture ratio, respectively. More simply, the waste rock particles in each laboratory mixture specimen were initially spaced further apart than similar waste rock particles without tailings slurry. The unsaturated upper portions of the fills of Columns 2 and 3 were waste rock dominated and contained air in the void space. The specimens tested in the laboratory were similar to the lower, tailings-saturated portions of the column fills, but had slightly larger porosities and similar mixture ratios.

7.3 Hydraulic Conductivity

Values of hydraulic conductivity, $k$, from laboratory falling head tests, analysis of consolidation of laboratory specimens, and analysis of the column profile behaviours are summarized in Figure 7.4. Values of hydraulic conductivity measured in the laboratory varied between $1.8 \times 10^{-9}$ m/s and $4.2 \times 10^{-7}$ m/s for tailings, and between $4.3 \times 10^{-9}$ m/s and $7.6 \times 10^{-8}$ m/s for the mixtures. Values of hydraulic conductivity determined from analysis of column data were between $1.1 \times 10^{-8}$ m/s and $3.9 \times 10^{-8}$ m/s for the 5:1 mixture in Column 1, $5.4 \times 10^{-8}$ m/s for the 6:1 mixture in Column 2, $1.3 \times 10^{-7}$ m/s for the 7:1 mixture in Column 3, and 0.3 m/s for the waste rock in Column 4.
Figure 7.4 indicates that the value of \( k \) for waste rock was several orders of magnitude greater than that of the mixtures and tailings. The mixtures and tailings had similar values of \( k \) but different values of void ratio, \( e \). The value of \( k \) of the mixtures appears to be more sensitive to changes in \( e \) than tailings. It should be noted that Figure 7.4 refers to void ratio, \( e \), and does not refer to values of \( e_r \) or \( e_t \).
7.3.1 Hydraulic Conductivity of Mixtures Related to Tailings Alone

Laboratory test data presented in Chapter Five indicated that the hydraulic conductivity of mixtures can be related to tailings matrix void ratio, $e_t$. The data in Figure 5.21 indicated log-linear relationships between $e_t$ and $k$ for the mixtures examined. Kumar and Muir Wood (1997) proposed a relation for sand-bentonite mixtures to relate $k$ and $e$ for the clay portion [7.1]:

$$[7.1] \quad k = A e_{clay}^B$$

where $A$ and $B$ are constants. A similar relation may be used for mixtures of waste rock and tailings as in [7.2].

$$[7.2] \quad k = A e_t^B$$

Fitted relations (not shown) using [7.2] had $R^2$ values of 0.97 for the tailings, 0.91 for the 4.2:1 mixture, and 0.86 for the 4.3:1 mixture. Comparison of $e_t$ versus $k$ relationships for tailings and mixtures in Chapter Five (Figure 5.20) indicated that the mixtures had lower values of $k$ than tailings with the same value of $e_t$. Although the slopes of the $e_t$ versus log $k$ lines were similar, the value of $k$ for the mixtures was lower than that of the tailings with the same value of $e_t$.

The reduction in $k$ of the tailings due to the addition of waste rock is attributed to a reduction in the area available for flow, and also to increased tortuosity of the flow path. In a study of bentonite-sand mixtures, Mollins et al. (1996) used the approach of Porter (1960) to relate the value of $k$ of a mixture sand and clay to the value of $k$ of clay alone by [7.3]
\[ 7.3 \] \quad k_{\text{mix}} = k_{\text{clay}} \cdot n_s \cdot T_s \\

where \( k_{\text{mix}} \) is the hydraulic conductivity of the mixture, \( k_{\text{clay}} \) is the hydraulic conductivity of the clay, \( n_s \) is the porosity of the sand, and \( T_s \) is the tortuosity of the sand.

Inspection of Equation [7.3] indicates that an increase in the tortuosity of the flow pathway cannot be attributed to the sand component alone. Rather, the increase in tortuosity must result from interactions between the sand and clay particles. Container wall effect will affect the packing of clay particles at the surface of sand particles. While not strictly correct, Equation [7.3] does provide a useful means of estimating mixture hydraulic conductivity.

A similar relation to Equation [7.3] is proposed here for mixtures of waste rock and tailings as per Equation [7.4]:

\[ 7.4 \] \quad k_{\text{mix}} = k_{\text{tailings}} \cdot n_r \cdot T \\

where \( k_{\text{mix}} \) is the hydraulic conductivity of the mixture, \( k_{\text{tailings}} \) is the hydraulic conductivity of the tailings, \( n_r \) is the porosity of the rock, and \( T \) is the increase in tortuosity of the flow path. The distinction is made that the term \( T \) is a function of the interaction of the waste rock and tailings, rather than waste rock alone.

The data presented here suggests that \( k_{\text{tailings}} \) is the dominant variable in Equation [7.4], while \( n_r \) is not expected to vary significantly for mixtures near the 'just filled' optimum. The tortuosity term, \( T \), is expected to vary with particle size ratio and waste rock hydraulic conductivity. Using \( k \) versus \( e \) relationships fitted to the mixture data in the
form of [7.2], equation [7.4] indicates that the 4.2:1 mixture had values of $T$ near 0.35, and the 4.3:1 mixture had values of $T$ near 0.6.

It is of interest to note that Borgesson et al. (2003) found that values of $k_{mix}$ for mixtures of bentonite and crushed rock were significantly higher than expected based on predictions ignoring tortuosity, shown in Equation [7.5]:

\[ [7.5] \quad k_{mix} = k_{clay} * n_b \]

where $n_b$ is the porosity of crushed rock. The finding was attributed to non-homogeneity or uneven distribution of bentonite powder and the formation of preferential flow pathways through the mixture. The difference in hydraulic conductivity may also be due, in part, to the lack of a correction for changes in flow path tortuosity.

Other studies of co-disposal noted changes in $k$ with $R$, but did not relate the values of $k$ of tailings to $k$ of mixtures. Leduc et al. (2004) found that $k$ increased with $R$ for from mixtures ratios of 0:1 (tailings alone) to 4:1 but did not provide testing methods or particle sizes. Williams and Kuganathan (1992b) attributed an increase in $k$ with $R$ to preferential seepage along the relatively flat surfaces of coarse particles for co-disposed coal washery wastes. In terms of particle packing theory, the increase in permeability is due to container wall effect, where flat surfaces prevent fine particles from packing tightly together and thus create preferential flow paths. The increase in $k$ with $R$ suggests that the value of $T$ was greater than 1 for the mixtures examined by Leduc et al. (2004) and Williams and Kuganathan (1992b).
Williams et al. (2003b) presented data indicating that values of $k$ for mixtures were lower than for tailings alone for uncompacted mixtures with $R$ values of 5:1 and 10:1 and for compacted mixtures with $R$ values of 15:1. Values of $k$ for mixtures with $R$ values of 20:1 were near that of rock alone. Mixtures of 5:1 fresh rock to tailings had values of $k$ that were also near that of rock alone. It is noted that maximum particle size in mixture specimens was 75 mm and that the testing mold was 152 mm in diameter and 178 mm high (Williams et al. 2003b). However, it should be noted that the resulting ratio of specimen diameter to maximum particle size was 2:1, which is much lower than the ASTM D5868-95 recommended minimum ratio of 6:1.

Aside from changes in $T$, other reasons for the observed decrease in $k$ of mixtures relative to tailings could be the entrainment of air. No attempt was made to back-pressure saturate the hydraulic conductivity test specimens. If occluded air remained within the specimens during the falling head tests and static loading, then air bubbles could act as barriers to flow in a manner similar to additional rock particles. If bubbles were concentrated at one plane in the specimen, then the area available for flow would be reduced.

### 7.3.2 Hydraulic Conductivity Related to Change in Structure

In general, and as expected, values of hydraulic conductivity, $k$, for mixtures and tailings were observed to decrease with static loading. However, the compressibility of the mixtures was less than the tailings alone due to the presence of the waste rock skeleton. Consequently, the tailings in the mixture underwent less volume change than tailings alone subjected to similar pressures. The change in the value of $k$ for a given pressure
was therefore greater for tailings than for the mixtures. While mixtures had lower values of $k$ than tailings with the same value of $e_t$, the tailings had much lower values $e_t$ than mixtures compacted at the same stress. As a result, the tailings had higher values of $k$ at low preconsolidation pressures, and lower values of $k$ at high preconsolidation pressures, than the mixtures. Strain compatibility and the magnitude of volume change governed changes in hydraulic conductivity due to changes in $e_t$.

### 7.4 Unsaturated Behaviour

Pressure plate tests for soil-water characteristic curves (SWCC’s) presented in Section 5.6, Figures 5.21 through 5.27, did not indicate a well defined air entry value (AEV) for mixtures or for tailings. Instead, water losses at low suctions were accountable by volume change. Cracks were observed on the surface of the mixture specimen at pore-water pressures below -40 kPa. Cracks represent a secondary structural feature or macrostructure with respect to the unsaturated soils model described by Fredlund and Rahardjo (1993). Cracks may increase hydraulic conductivity and air permeability by several orders of magnitude, depending on size and connectivity.

Comparison of the laboratory derived SWCC (Figure 5.27), and column study tensiometer measurements (Figure 6.12) indicates that the Column 1 profile should not have desaturated or cracked at 125 days after loading, and should have had cracks in the upper portions of the column profile after 700 days. Tensiometer measurements at 125 days indicated a minimum pressure of greater than -20 kPa at the surface of the Column 1 profile, as shown in Figure 6.12. Final tensiometer measurements, taken after 700 days were near -60 kPa as shown in Figure 6.15. Observations made during the pressure plate
test of a sample taken during loading of Column 1 indicated that the negative pore pressure required to induce cracking was -40 kPa. Observations confirm that no surface cracking was visible on the Column 1 profile at 125 days, while cracks were found within the Column 1 profile at the time of decommissioning. During deconstruction after 700 days, cracks were observed at the surface and in the upper 2.5 m of the Column 1 profile. If no surface evaporation had been allowed in Column 1, then the final pore pressure profile would be expected to be negative and hydrostatic, as illustrated in Figure 6.15. Evaporation from the surface of the fills likely caused pore-water pressures to decrease below the hydrostatic distribution. The sign conventions used here equates negative pore-water pressures with positive matric suctions.

The upper profiles in Columns 2 and 3 were initially unsaturated and stayed unsaturated for the duration of the test. The terms "unsaturated" and "saturated" are used with respect to water content of the granular pore space. Tensiometer measurements taken after the first 100 days of the test are shown in Figure 6.13 and 6.14. Final tensiometer measurements are shown in Figures 6.16 and 6.17. Observations made during decommissioning of Column 2 indicated fissures or cracks from the surface of the profile down to an elevation of approximately 2.5 m. Observations made during decommissioning of Column 3 indicated cracks above 4 m elevation. Although the specimen tested for SWCC was taken from Column 1, observations appear to correlate cracking within the tailings matrix at pore-water pressures below approximately -40 kPa for the Column 2 and 3 profiles.
The waste rock only profile in Column 4 was initially unsaturated, and remained unsaturated until the column was flooded at day 13 of the test.

In general, comparison of mixtures in the Column profiles indicated that mixture ratios lower than six were subject to cracking at pore-water pressures lower than -40 kPa. It should also be noted that a prolonged period of drainage without access to rainfall was required to develop the negative pore-water pressures and associated cracking.

### 7.5 Volume Change Behaviour

Consolidation parameters presented in Chapters Five and Six are summarized in Table 7.1, as ranges where possible.

<table>
<thead>
<tr>
<th></th>
<th>CIP Tailings</th>
<th>Waste Rock</th>
<th>4.4:1 Mixture</th>
<th>4.8:1 Mixture</th>
<th>Column 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_v$ (m$^2$/s)</td>
<td>2.0*10^{-7}</td>
<td>1.2*10^{-6}</td>
<td>1.6*10^{-6}</td>
<td>8.8*10^{-6}</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9.5*10^{-7}</td>
<td>2.3*10^{-5}</td>
<td>6.7*10^{-5}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$m_v$ (1/kPa)</td>
<td>1.9*10^{-4}</td>
<td>7.0*10^{-5}</td>
<td>8.5*10^{-5}</td>
<td>5.5*10^{-5}</td>
<td>8.5*10^{-4}</td>
</tr>
<tr>
<td></td>
<td>1.9*10^{-2}</td>
<td>4.1*10^{-4}</td>
<td>8.0*10^{-4}</td>
<td>1.0*10^{-3}</td>
<td></td>
</tr>
<tr>
<td>$k$ (m/s)</td>
<td>1.8*10^{-9}</td>
<td>0.3†</td>
<td>7.3*10^{-9}</td>
<td>8.8*10^{-9}</td>
<td>3.9*10^{-8}</td>
</tr>
<tr>
<td></td>
<td>4.2*10^{-8}</td>
<td>3.3*10^{-8}</td>
<td>3.6*10^{-8}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_c$</td>
<td>0.526</td>
<td>0.005</td>
<td>0.016</td>
<td>0.042</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.87</td>
<td>0.090</td>
<td>0.073</td>
<td>0.059</td>
<td></td>
</tr>
<tr>
<td>$C_\alpha$</td>
<td>0.0031</td>
<td>0.0068</td>
<td>0.0003</td>
<td>0.0037</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.017</td>
<td>0.014</td>
<td>0.067</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

† from Column 4

In general, both laboratory and column study data indicated that mixtures had similar compressibility to waste rock alone, and similar hydraulic properties to tailings alone.
The laboratory and Column 1 mixtures had similar values of coefficient of consolidation, \( c_v \); coefficient of volume change, \( m_v \); hydraulic conductivity, \( k \); and secondary compression index, \( C_\alpha \). Values of \( c_v \) for the tailings were an order of magnitude lower than that of the mixtures. The laboratory waste rock and mixture specimens had similar values of \( m_v \), \( C_C \), and \( C_\alpha \). Tailings had higher values of \( m_v \) than the mixtures and waste rock. Values of \( k \) are discussed below, but it is noted here that values of \( k \) for the mixtures were more similar to the tailings than the rock.

Laboratory testing results presented in Section 5.4.3 indicate that mixture structure changes with compression. However, the individual structures of waste rock and tailings components changed differently, as described by changes in \( e_t \) and \( e_r \). Changes in the value of \( e_r \) were similar to changes in the value of \( e \) for waste rock subjected to similar loads. Changes in the value of \( e_t \) were much smaller than changes in the value of \( e \) for the tailings only specimen subjected to the same stress. The observations confirm the idea that the presence of waste rock in the mixture governs the degree of compression of the tailings void space, as defined by equation [5.3]. The change in mixture structure during compression will affect other geotechnical properties that are dependent on the values of \( e_r \) and \( e_t \), such as hydraulic conductivity.

### 7.6 Predictions of Density Related Properties

In order to provide basis for evaluation of mixtures as a mine waste disposal technique, it is useful to illustrate the geotechnical properties of mixtures for a range of mixture ratios.
The compressibility testing results presented in Chapter Five indicate that mixtures had volume changes that were more similar to waste rock than to tailings. Data indicated that mixture compressibility behaviour was more similar to waste rock, in terms of magnitude, than to the tailings alone. The data raise the question of the degree of influence of the waste rock versus the tailings for mixture compressibility behaviour. The primary mechanism for volume change at the range of stresses applied during laboratory compressibility testing was most likely the compression of particle structure, with particles moving closer together as water is expelled. Some particle breakage was noted to occur during compression of specimens containing waste rock, but did not appear to significantly change the particle size distributions in Figures 5.2 and 5.3.

In terms of the particle model presented in Chapter Three, and the specimen structures presented above, it is assumed that compression of a mixture will change the mixture particle structure such that $e_r$ and $e_t$ both decrease during compression as water and air are expelled from the mixture. The change in structure due to compression will presumably increase dry density. Because most soil behaviour is a function of structure, changes in structure due to compression will affect other geotechnical behaviour including permeability, shear strength, and potential for liquefaction. Consequently, the ability to predict changes in mixture structure is useful for the evaluation of other mixture geotechnical behaviours. Studds et al. (1998) presented a method for predicting the change in structure of bentonite sand mixtures from the compression curves of sand alone and bentonite alone. A similar method is re-formulated here to predict the volume
change of mixtures of waste rock and tailings and tested using the laboratory data presented in Chapter Five.

7.6.1 A Conceptual Model for Compressibility of Mixtures

A conceptual model for mixture compressibility behaviour is introduced here as a basis for predicting void ratio versus effective stress relationships for mixtures. The model is similar, if not identical to the model proposed by Studds et al. (1998) for mixtures of sand and bentonite. Basic assumptions for the model include:

1. the mixture is homogeneous,
2. compression is one dimensional and monotonic,
3. effective stresses are implied, excess pore pressures are ignored,
4. waste rock, tailings particles, and water are inelastic, or incompressible, and
5. creep effects are ignored.

Mixtures are conceptualized in terms of the particle model presented in Section 3.4, with a waste rock skeleton and a tailings matrix, described by $e_r$ and $e_t$, which each experience a portion of applied vertical stress. It is assumed that the sum of the load carried by the waste rock skeleton and the tailings matrix is equal to the applied load, as illustrated by the spring model shown in Figure 7.5, and by Equation [7.10], which is presented below.

The compressibility of the waste rock portion of the mixture is assumed to be identical to waste rock alone. Similarly, the compressibility of the tailings portion of the mixture is assumed to be identical to tailings alone. Strain compatibility is assumed, and implies that a change in the volume of void space in the waste rock skeleton is equal to the change in the volume of tailings slurry.
7.6.2 A Method for Predicting Mixture Compressibility

The spring model provides a basis predicting mixture compressibility. Specifically, a void ratio versus applied stress relationship may be predicted from mixture ratio, $R$, density of waste rock and tailings, and measured compression curves for waste rock alone and tailings alone. The method predicts changes in applied stress from changes in void ratio by following these steps:

1. The mixture is assumed to have an initial void ratio that may be calculated from the mixture design, and assumptions from particle packing theory.

2. From the initial mixture void ratio, a successively smaller mixture void ratio is arbitrarily assumed, as if the mixture had been subjected to incremental loading and compression.

3. Each arbitrary void ratio is used to derive waste rock skeleton and tailings matrix void ratios.

4. The value of the waste rock skeleton void ratio is referenced to the compression curve for waste rock alone in order to predict of the stress carried by the waste rock portion of the mixture. The compressibility curve for the waste rock
skeleton in the mixture is assumed to be identical to that of the parent waste rock, and all effects of the tailings matrix on the waste rock skeleton are assumed to be negligible. The stress required to compress the parent waste rock to a void ratio equivalent to that of the mixture waste rock skeleton void ratio is taken as the stress carried by the mixture waste rock skeleton.

5. Similarly, the tailings matrix void ratio is referenced to the compression curve for tailings alone to predict the stress carried by the tailings portion of the mixture. The compressibility curve of the tailings matrix is assumed to be identical to that of the parent tailings, and all effects of the waste rock skeleton on the tailings matrix are ignored. The stress required to compress the parent tailings to a void ratio equivalent to the mixture tailings matrix void ratio is taken as the stress applied to the tailings matrix alone.

6. The stresses calculated for waste rock skeleton in Step 4, and for the tailings matrix in Step 5 are summed. The sum is taken as the stress required to compress the mixture to the void ratio assumed in Step 2.

7. By repeating steps 2 through 6 for a set of arbitrary void ratios, a set of stresses may be calculated, resulting in a void ratio versus applied stress function.

The method described above is tested using laboratory test data:

Step 1) The initial void ratio, \( e \), of the mixture can be predicted from phase relations and source material properties using equation [3.4]. The method is similar to that described by Morris and Williams (2000b).
Step 2) An arbitrary sequence of mixture void ratios is selected based on the initial predicted void ratio calculated in Step 1).

Step 3) The tailings matrix void ratio, $e_t$, can then be derived using [3.8], or from each void ratio, $e$, calculated in Step 1) along with mixture ratio, $R$, and the density of the tailings solids $\rho_t$ and waste rock, $\rho_r$ using [7.6],

$$[7.6] e_t = e + (e)(R)(\rho_t/\rho_r)$$

Similarly, the waste rock skeleton void ratio, $e_r$, can be derived using [3.6] or [7.7].

$$[7.7] e_r = e + (\rho_r/\rho_t)(1/R)(e + 1)$$

Step 4 and 5) Next, it is assumed that the waste rock skeleton and tailings matrix portions of the mixture follow void ratio, $e$, versus applied vertical effective stress, $\sigma_v'$, relationships described for the parent waste rock and tailings. The $e_r$ versus $\sigma_v'$ relationships of the waste rock and tailings specimens shown in Figure 5.19, may be described by equations [7.8] and [7.9], which were fitted to the data (with $R^2$ values greater than 95%) and adopted here for demonstration purposes.

$$[7.8] \sigma_{v'r} = 36276e_r^2 - 52318e_r + 18871$$

$$[7.9] \sigma_{v't} = 518.78e_r^{5.125}$$

Step 6) In order to determine the stress at each global void ratio, $e$, it is first assumed that the stress applied to the mixture, $\sigma_v'$, may be partitioned to waste rock as $\sigma_{v'r}$, and to tailings as $\sigma_{v't}$, using [7.10].

$$[7.10] \sigma_v' = \sigma_{v'r} + \sigma_{v't}$$
Step 7) Equations [7.6], [7.7], [7.8], [7.9], and [7.10] are sequentially solved for a series of arbitrary initial void ratios to construct a predicted $e$ versus $\sigma_{v'}$ relationship for the mixture.

The technique assumes that compression and volume change occur only with an increase in $\sigma_{v'}$, and that the $e$ versus $\sigma_{v'}$ relationships for the parent waste rock and tailings are unique. Mixture compression is assumed to occur under monotonic loading such as an increase in effective stress that would occur during self-weight loading.

Predictions of $e$ versus $\sigma_{v'}$ for the 4.8:1 and 4.4:1 compressibility test specimens are plotted with measured data in Figure 7.6. The average absolute error in $\sigma_{v'}$ predicted for a given $e$ is 90% for the 4.4:1 prediction, and 69% for the 4.8:1 prediction. The prediction seems to be a poor fit, but the shapes of the curves are similar. The large variation in predicted values of $\sigma_{v'}$ may be due to variations in the initial packing and values of $e_r$ of the waste rock portions of each mixture. Predicted values of Compression Index, $C_c$, are included in Table 7.2. The average absolute error for the predicted $C_c$ values for both the 4.4:1 and 4.8:1 mixtures was 17% relative to the measured values, excluding the 0 kPa to 10 kPa interval.
Figure 7.6 Measured and predicted void ratio versus applied stress relationships.

Table 7.2. Summary of predicted Compression Indices, \( C_f \)’s.

<table>
<thead>
<tr>
<th>Nominal Pressure Interval (kPa)</th>
<th>CIP Tailings Only ( C_f )</th>
<th>Waste Rock Only ( C_f )</th>
<th>4.4:1 Mixture measured / predicted ( C_f )</th>
<th>4.8:1 Mixture measured / predicted ( C_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td>0.839</td>
<td>0.016</td>
<td>0.016 / 0.062</td>
<td>0.048 / 0.043</td>
</tr>
<tr>
<td>10-20</td>
<td>1.87</td>
<td>0.005</td>
<td>0.067 / 0.047</td>
<td>0.052 / 0.033</td>
</tr>
<tr>
<td>20-40</td>
<td>0.565</td>
<td>0.053</td>
<td>0.042 / 0.043</td>
<td>0.042 / 0.038</td>
</tr>
<tr>
<td>40-80</td>
<td>0.712</td>
<td>0.035</td>
<td>0.065 / 0.050</td>
<td>0.050 / 0.045</td>
</tr>
<tr>
<td>80-160</td>
<td>0.596</td>
<td>0.090</td>
<td>0.073 / 0.053</td>
<td>0.054 / 0.050</td>
</tr>
<tr>
<td>160-320</td>
<td>0.526</td>
<td>0.086</td>
<td>0.073 / 0.073</td>
<td>0.059 / 0.073</td>
</tr>
</tbody>
</table>

The prediction technique does not appear to work well at pressures less than 10 kPa, but data for measured curves at low pressures were not considered reliable due to friction in the tester and variation in applied pressure. The technique implies that the waste rock
particles do not interact until \( e_r \) for the mixture is equal to the initial void ratio, \( e \), of the waste rock only specimen, as illustrated in Figure 7.5. Until \( e_t \) for the mixture equals \( e \) for the waste rock, it is assumed that all load is carried by the tailings particles. The assumption does not necessarily represent what actually occurs, but does provide a basis for predicting volume change behaviour.

### 7.6.3 Participation Factors

Because the technique described above predicts the individual loads carried by the waste rock and tailings, it is possible to partition the predicted load between the waste rock skeleton and tailings matrix. A participation factor for rock, \( \alpha_r \), is introduced here as the proportion of the applied stress carried by the waste rock skeleton of the mixture. Where \( \alpha_r \) is equal to one, the stress applied to the mixture is carried entirely by the waste rock skeleton. Where the \( \alpha_r \) is zero, the applied stress is carried entirely by the matrix of tailings particles (which includes the "floating" waste rock particles). A plot of predicted \( \alpha_r \) versus \( \sigma_{v'} \) for the 4.4:1 and 4.8:1 mixtures is shown in Figure 7.7. Figure 7.7 indicates that 90% of the pressure applied to each mixture is carried by the rock skeleton of each mixture at pressures greater than 100 kPa. The reader should be aware that Figure 7.7 illustrates a predicted relation, and the author knows of no way to verify the actual stress carried by the waste rock or tailings portions of the mixtures. However, the idea that the rock may carry a significant portion of the load is significant to many behaviours, including compressibility, shear strength, and liquefaction.
7.7 Summary

Initial particle structure of mixture specimens was classified with respect to the conceptual particle model presented in Section 3.4. The laboratory specimens had initial structures that were slightly tailings dominated, as did the lower, saturated portions of the column fills. Hydraulic conductivity of field and laboratory specimens were found to be similar. The hydraulic conductivity of mixtures was related to the hydraulic conductivity of tailings alone, and to changes in particle structure. Finally, volume change of mixtures was examined, and a method to predict void ratio versus effective stress relationships for mixtures was presented.
CHAPTER EIGHT. DISCUSSION

Chapter Eight presents a discussion of the rational framework, methodology and findings of investigations and analyses.

8.1 Rational Framework

In order to evaluate mixing as a mine waste disposal technique, it is important to understand the effect that design variables, such as mixture ratio, have on behaviour. Particle packing theory provides a fundamental theoretical basis for relating mixture design to behaviour through structure. The rational framework or approach used in the evaluation of mixtures is made up of the following theorems:

1. particle packing theory provides a basis for the design of mixture structure,
2. mixture structure governs mixture behaviour, and
3. mixture behaviour provides a basis for evaluating mixtures as a mine waste disposal technique.

The literature review in Chapter Three indicated that existing studies of co-disposal have not linked mixture design to behaviour, other than through empirical testing. In order to provide a fundamental theoretical basis for understanding mixture design and behaviour, particle packing theory and studies of soil mixture geotechnical behaviour were reviewed.

The review of particle packing theory provided a basis for understanding the effect of mixture design variables on mixture structure, and is discussed below in Section 8.2. The literature review of studies of soil mixtures indicated that the structure of binary mixtures
with a large particle size difference is quantifiable in terms of the packing of large and small particle components. Large particle void ratio of soil mixtures has been related to mechanical behaviours including shear strength, static and cyclic liquefaction, and compressibility. Small particle void ratio of soil mixtures has been related to hydraulic conductivity and compressibility. This thesis describes the structure of mixtures of waste rock and tailings in a similar manner, using the terms of a waste rock skeleton void ratio, $e_r$, and a tailings matrix void ratio, $e_t$. Mixture-specific structural descriptors such as $e_r$ and $e_t$ were found to be better indicators of geotechnical behaviour for the mixtures than traditional descriptors such as void ratio and porosity.

Practical investigations of the behaviours of mixtures of waste rock and tailings included laboratory-scale studies of volume change and permeability behaviour, as well as a meso-scale column study of self-weight consolidation. One type of waste rock, one type of tailings, and mixtures of the same waste rock and tailings were investigated for volume change, hydraulic conductivity, and water retention. Due to the dependence of mixture behaviours on mixture design, a comprehensive investigation of mixture properties for all designs was unfeasible. However, this thesis has attempted to provide theory that is fundamental to understanding mixture design, structure, and behaviour, with the intention that the same theory may be applied to other materials. Mixture theory is used with laboratory testing results to predict particle structure, compressibility, and hydraulic conductivity of mixtures with a range of mixture ratios. Analysis of the results allows evaluation of mixing waste rock and tailings as a mine waste disposal technique.
8.2 Design

Design variables investigated for mixtures of waste rock and tailings included mixture ratio, $R$, and initial tailings solids content, $P$. The effect of mixture ratio was investigated through particle packing theory to predict initial structure, and also through mixture trials. The effect of tailings solids content was investigated through mixture trials and tailings rheology.

The work presented in this thesis with respect to the use of particle packing theory for mixture design follows the method for predicting mixture porosity proposed by Morris and Williams (2000b). The current work uses the theory initially presented by Furnas (1928). Specifically, the predictions of mixture porosity from source material properties and mixture ratio provide accurate indications of initial mixture porosity. The properties of the waste rock and tailings used in the 4.8:1 laboratory test specimen were used to create Figures 3.3, 3.4, and 3.5. Based on particle size data shown in Figure 5.3, the ratio of average particle sizes (i.e. $D_{50}$) was greater than 1000 for the waste rock and tailings used to create the 4.8:1 mixture. The 4.8:1 mixture was not observed to contain any large air voids. Calculated and experimentally determined particle size distributions were similar. The initial dry density of the 4.8:1 specimen was calculated to be 1.842 g/cm$^3$ based on laboratory measurements, and was predicted to be 1.814 g/cm$^3$ based on mixture design, for a difference of 1.5%. The initial water content of the 4.8:1 mixture was calculated to be 17.7% and predicted to be 18.5%, for a difference of approximately 4.5%. The higher density and lower water content of the mixture may indicate water loss during the mixing process. In practice, the mixtures were prepared with an excess of
tailings to try to account for losses to the mixing equipment. The initial void ratio of the waste rock was also expected to vary. In general, the laboratory mixture trials confirmed that particle-packing theory is suitable for predicting the initial structure of mixtures of waste rock and tailings.

Other researchers have previously used particle-packing theory to predict mixture porosity. Furnas (1928) was the first to develop particle-packing theory for binary mixtures. Powers (1964) used the theory presented by Furnas (1928) for the design of concrete aggregates. Al Jarallah and Tons (1981) used the same theory to predict the porosity of binary mixtures of compacted aggregate. Morris and Williams (2000) presented a method for predicting the porosity of coal washery wastes deposited by pumped co-disposal. Morris and Williams (2000b), and also Vallejo (2001) presented simplified versions of Figure 3.4 (without particle size ratio effects or reference to Furnas 1928), in order to predict mixture porosity. The use of the same concepts to predict porosity of mixtures composed of different material types (ore, shot, beads, coal tailings and reject, etc.) demonstrates the fundamental nature of particle packing theory for binary mixtures.

Mixture ratio, $R$, has a design optimum for maximum density at the “just filled” point. Source tailings solids content, $P$, should be as large as possible to reduce porosity and produce a maximum density. While maximum density mixtures are desirable it appears that there is an upper limit to the value of source tailings solids content, $P$. Practical observations of increasing tailings solids content, and quantification of tailings rheology
presented in Chapter Five demonstrated that yield stress could provide an indicator of suitable tailings solids content for mixtures created by hand. In order for the tailings to flow into the void space of the rock during mixing, there is a limit to the tailings yield stress. Attempts to increase tailings solids content during mixture trials resulted in mixtures that were stiff and contained air voids. The yield stress suitable for mixing by hand was found to be between 15 and 30 Pa. Similar trials could be carried out to determine solids contents and yield stresses suitable for a mechanical mixing process. Vane shear tests or slump tests are recommended for future work in characterizing tailings yield stress, as per Morris and Williams (2000), and Clayton et al. (2003).

8.3 Structure

Structure of mixtures was defined in terms relevant to particle packing theory. For binary mixtures, the packing of large particle and small particle components may be described separately. The benefit to defining separate structures is that the structure of each component may be related to different physical behaviours.

8.3.1 Variables for Defining Structure

Traditionally, soil structure is described in terms of porosity, \( n \); void ratio, \( e \); and relative density, \( D_r \). The literature reviews presented in Chapter Three indicated that binary soil mixtures have a more complex structure than “single size” soils, and also that the structure is quantifiable. The conceptual particle model presented in Chapter Three defined mixture structures including waste rock dominated, tailings dominated, and an intermediate structure where the tailings “just fill” the voids of the rock. Most importantly, mixture structure is quantifiable in terms of the packing arrangements of
small and large particle components. The waste rock skeleton void ratio $e_r$, and tailings matrix void ratio, $e_t$, are used to describe the structure of mixtures investigated in this thesis. Studies of geotechnical investigations of soil mixtures reviewed in Chapter Three have indicated that the structure of binary mixtures governs geotechnical behaviours. The terms $e_r$ and $e_t$ are suitable for the work presented in this thesis, and may be used to relate design variables to structure, and therefore to geotechnical behaviours.

8.3.2 Relation Between Design Variables and Structure

The design variables of mixture ratio and tailings solids content are directly related to mixture structure, as described by $e_r$ and $e_t$, respectively. Mixture ratio, $R$, is defined as the ratio of waste rock to tailings by dry mass. The value of $R$ is inversely related to $e_r$: high values of $R$ result in low values of $e_r$, and low values of $R$ result in high values of $e_r$. Tailings solids content, $P$, is defined as mass of tailings solids divided by total mass of tailings slurry. The value of $P$ is inversely related to $e_t$, and increases in $P$ will directly decrease the value of $e_t$. The relationships between mixture design variables, $R$ and $P$, and the variables used to describe mixture structure, $e_r$ and $e_t$, allow mixture design to be related to behaviour.

When $e_r$ of a mixture is related to the void ratio of waste rock alone, it provides a measure of whether the waste rock particles in the mixture are in contact and form a continuous, load bearing skeleton, or are separated and “floating” in a matrix of tailings. Consequently, large particle void ratios, analogous to $e_r$ for other soil mixtures, have been related to mechanical behaviours such as shear strength and compressibility.
Similarly, the value of $e_t$ provides an indication of the density of the finer particle matrix in the mixture. Where the matrix is continuous, $e_t$ governs hydraulic conductivity. Past investigations of soil mixtures have indicated that behaviour can be interpreted and even predicted with respect to structure. The present investigations of the mixtures of waste rock and tailings support the idea.

8.3.3 Effects of Mixing on Initial Structure

The saturated portions of the three column mixture profiles described in Chapter Six all had mixture ratios near 6:1. The mixtures were constructed using the same mixing and placement process with rock from the same waste rock stockpile, and with the same tailings with slightly different values of $P$. The mixture designs were all different, but two columns had a tailings shortfall where the volume of tailings slurry was less than the volume of the waste rock void space. It is of interest that the as-produced mixture ratios were similar, near 6:1, despite design mixture ratios of 7.5:1, 10:1, and 12.5:1. The similarity of resulting mixture ratios is attributed partly to the mixing process.

For a given mixing process it should be possible to define a mixing void ratio. If the mixing of waste rock is considered, it is logical to assume that in order for mixing to occur, the waste rock particles must not interlock, but must separate enough to allow movement of particles relative to other particles. In geotechnical terms, the process of mixing creates localized zones of shearing where the particle structure dilates to allow movement. For any given mixing process, such as the mixing of the waste rock in the concrete transit mixer in the column study, there will be localized zones of shearing within the mixture mass. The void ratio will be higher in zones of shearing, and may be
subject to collapse once out of the zone of shearing. For the dry waste rock, the expanded structure produced by shearing should collapse under self-weight to a value of void ratio that is near the maximum observed for waste rock. The collapse is due to gravity, and will depend on the geometry of the mixing process. For saturated materials, the rate of collapse following shearing will be limited by the hydraulic conductivity and compressibility of the material being mixed. For mixtures of waste rock and tailings, the waste rock skeleton structure produced by mixing will not collapse instantaneously due to the low hydraulic conductivity of the tailings matrix. Consequently, the initial structures of mixtures of waste rock and tailings will tend to be tailings dominated, with a waste rock skeleton void ratio that is near the maximum for waste rock alone.

Further inspection of the mixing process indicates that while the waste rock void ratio must be large enough to allow rock particles to move relative to one another, the same must be true of tailings particles. The rheology testing presented in Chapter Five indicates that the stress required to shear the tailings is dependent on solids content. Although untested, it is feasible that a larger researcher or a mechanical process could more effectively mix tailings at higher solids contents with higher yield stresses. However, the entrainment of air voids may also be an issue. The energy required to mix waste rock and tailings is a function of mixture structure and it should therefore be possible to calculate the energy requirements for mixture construction from the mixture design. Calculations of energy requirements for mixing and further investigations of the characteristics of different mixing techniques are beyond the scope of this thesis and are left for future investigations.
8.4 Volume Change, Hydraulic Conductivity and SWCC Behaviour

As stated in Chapter Three, the evaluation of mixture geotechnical properties and behaviours is complicated by the dependence of behaviour on mixture design. Studies of soil mixtures reviewed in Chapter Three indicated a strong dependence of geotechnical properties and behaviours on the proportions of mixture components. The same is true for mixtures of waste rock and tailings. Once a mixture design has been selected, the evaluation of geotechnical properties is straightforward. In lieu of an exhaustive empirical approach of testing the full range of possible mixture ratios, this thesis has investigated mixtures with mixture ratios near the ideal "just filled" case, and also at the limits of maximum and minimum mixture ratio. The initial structure of the remaining range of mixture ratios was predicted in Chapter Three, and a method to predict changes in mixture structure with compression was presented in Chapter Seven. The predicted mixture structures allow prediction of other behaviours with known relationships to structure for the intermediate range of mixture ratios. For example, the hydraulic conductivity of a range of mixtures may be predicted from structure and the properties of the parent materials.

Mixture behaviours that were directly examined included volume change, hydraulic conductivity, and soil-water characteristic curves. It is important to note that the results of laboratory and column investigations presented in Chapters Five and Six are unique to the waste rock and tailings examined. Other types of waste rock and tailings will have different values of hydraulic conductivity, compressibility, and soil-water characteristic curves. Consequently, the following discussion attempts to generalize the findings into
underlying theory or universal concepts that may be applied to other types of waste rock and tailings. The results also allow a direct comparison of properties for the waste rock and tailings investigated.

8.4.1 Volume Change of Mixtures

8.4.1.1 Compressibility and Degree of Volume Change

Mixtures investigated for compressibility in the laboratory and column studies had similar degrees of volume change compared to waste rock alone. Laboratory testing indicated the tailings were much more compressible than waste rock and mixtures of waste rock and tailings. In terms of strains, at an applied stress of 320 kPa the tailings compressed by 50%, while the waste rock and mixtures compressed by between 5% and 10%. Expressed in terms of coefficient of volume change, \( m_v \), tailings had higher values of \( m_v \) than waste rock and mixtures, which had similar values of \( m_v \).

The similar compressibility of mixtures compared to waste rock is attributed to the presence of the mixture waste rock skeleton. The initial structure of mixtures tested was slightly tailings dominated, with respect to the particle model presented in Chapter Three. With compression, the waste rock particles come together to form a continuous load-bearing skeleton. The waste rock skeleton then limits the degree of volume change with further loading, with strain compatibility acting to govern the degree of volume change of the tailings in the waste rock void space. Predicted waste rock influence factors presented in Chapter Seven indicated that more than 90% of an applied stress of 100 kPa is carried by the waste rock skeleton component of the mixtures examined.
If a minimum of volume change is desirable, then mixture ratio, $R$, and tailings solids content, $P$, may be chosen to reduce mixture compressibility. The laboratory testing results presented in Chapter Five showed a decrease in mixture compressibility with a decrease in values of $e_r$ and $e_t$. In order to minimize the initial value of $e_r$, the value of $R$ should be greater than or equal to the “just filled” optimum. In order to minimize the initial value of $e_t$, the value of $P$ should be as high as possible. Consideration of the effect of $R$ indicates that the ideal “just filled” case should theoretically be less compressible than waste rock alone because the tailings particles carry part of the applied load. When $R$ is greater than the “just filled” case the mixture will contain air and the tailings particles will not carry applied loads. Where $R$ is less than the “just filled” case, the structure will be tailings dominated, and therefore subject to a greater degree of volume change. The optimum design for a minimum of volume change therefore includes a value of $R$ equal to the “just filled” case, and a maximum value of $P$.

The above design options consider initial mixture structure only. If loading conditions are known, then it is possible to account for compression to produce mixtures with rock-to-rock contact, but with lower values of $e_t$. By adding an excess of tailings, or choosing an initial value of $R$ with an excess of tailings, subsequent compression will change the structure from one that is tailings dominated to one that is rock dominated. However, no such deliberate attempts were made during laboratory or column investigations. By coincidence, the mixtures investigated in the laboratory and column studies were initially slightly tailings dominated. The discussion of the effect of the mixing process in Section 8.3.3 indicated that the physical mixing process will tend to produce mixtures with initial
structures that are slightly tailings dominated. Results of compression testing indicated a shift from tailings dominated to rock dominated behaviour.

The model for mixture compressibility presented in Chapter Seven is a simplified representation of a physical process. The compressibility model implies that the transition from a waste rock dominated structure to a tailings dominated structure is distinct, and occurs with an abrupt change in behaviour. The assumption implies uniform particle shapes and absolute homogeneity. Investigations of other soil mixtures reviewed in Chapter Three have indicated that the transition for other soil mixtures is not distinct, and that transitional structures occur. More detailed models describing transitional structures with equivalent large and small particle void ratios have been proposed for evaluating the liquefaction potential and undrained response of mixtures of sand and silt, e.g. Thevanayagam (2000), Thevanayagam et al. (2000), and Thevanayagam et al. (2002). However, it was the intent of the model presented in Chapter Seven to provide a simplified representation of behaviour for static conditions.

Treating the waste rock and tailings in a mixture independently is a simplification that is useful for predicting and interpreting behaviour. Predictions from the compressibility model were most accurate at higher stresses, shown in Figure 7.6. Predictions had greater error at lower stresses, and may be due to the model's exclusion of transition effects. The transition in structure from tailings dominated to waste rock dominated occurred at low stresses, where compressibility test data were considered unreliable due to friction in the testing equipment. At the least, the simplifying assumptions used in the
model provide a basis for discussion and for the development of a more complex model of behaviour that could include transition behaviours.

8.4.1.2 Rate of Consolidation of Mixtures

The rate of consolidation of mixtures was an order of magnitude faster than tailings. The majority of consolidation of waste rock tested for compressibility in the laboratory occurred almost instantaneously upon loading. The decrease in time to completion of consolidation was attributed to differences in value of hydraulic conductivity, $k$, of mixtures, compared to tailings alone. As noted in Chapter Five, the value of $k$ decreased with the value of $e_t$ for saturated mixtures. It was also noted that the decrease in $e_t$ due to compression was limited by strain compatibility to the change in void space of the waste rock skeleton. Because the waste rock skeleton has low compressibility, the change in $e_t$ during mixture compression is limited compared to tailings alone under the same load. The smaller change in $e_t$ for the mixture compared to large changes in $e$ for tailings alone implies that conventional consolidation theory is better suited to mixtures than tailings, which have large strains and require more complex formulations to determine the rate of consolidation (e.g. Schiffman et al. 1988, Gibson et al. 1967, 1981). Although the strains for mixtures are small, the change in $e_t$ is magnified by porosity of the waste rock, as related by Equation [5.3].

Tailings disposed as slurry has a low density compared to rock or mixtures and therefore will have less self-weight to drive consolidation. At a solids content of 48%, the density of CIP tailings slurry is 1.46 g/cm$^3$, which means that less self-weight is available to drive consolidation than mixtures or waste rock, which have higher densities.
Thixotropic effects counteract the effect of gravity on the solids in tailings slurry. As mentioned in Chapter One, certain types of tailings may remain semi-fluid for years (e.g. Scott et al. 1985). Consequently, the rate of consolidation of tailings is difficult to predict using conventional consolidation theory (Schiffman et al. 1988). The consolidation of mixtures appears to occur with the dissipation of excess pore-water pressures and the development of effective stresses within the mixture mass. The majority of consolidation of waste rock occurred instantaneous upon application of load followed by a prolonged period of creep, or secondary consolidation. Rates of creep were similar for laboratory specimens of waste rock and mixtures tested for compressibility.

For design, more rapid consolidation could be made to occur by increasing the value of mixture hydraulic conductivity.

8.4.2 Hydraulic Conductivity of Mixtures

Laboratory and column studies both indicated that the mixtures tested had values of hydraulic conductivity, $k$, that were similar to tailings alone, and much lower than waste rock alone. The mixtures tested in the laboratory for hydraulic conductivity followed Darcy’s Law for the gradients used during the test. The values of $k$ for mixtures were related to the tailings matrix void ratio, $e_t$. Chapter Seven also included methods to relate the value of $k$ for a mixture from the value of $k$ for tailings alone, as well as the porosity, $n_r$, and tortuosity, $T$.

Changes in the value of $k$ due to compression were greater for tailings than for mixtures subjected to the same preconsolidation pressure. As mentioned above, the presence of a
waste rock skeleton limits the amount of the applied load that is transferred to the tailings matrix. Because the waste rock skeleton is less compressible than the tailings matrix, the waste rock effectively prevents compression of the tailings matrix. As a result, tailings had higher values of $k$ at low preconsolidation pressures, and lower values of $k$ at higher preconsolidation pressures than mixtures subjected to the same pressures.

For a given value of $e_t$, mixtures had lower values of $k$ than the tailings alone. Mechanisms to account for the decrease in the value of $k$ of mixtures relative to tailings alone were examined in Chapter Seven and include reduction in area available for flow due to the presence of waste rock, increased tortuosity of the flow pathway, and entrainment of air in the mixture, which will also reduce the area available for flow. Mechanisms for increasing the value of $k$ of mixtures compared to tailings alone include preferential flow along the surface of waste rock particles as a manifestation of container wall effect, and heterogeneity in the mixture. As mentioned above, the mixtures of waste rock and tailings examined by this thesis had lower values of $k$ compared to tailings with similar values of $e_t$. Previous investigations of co-disposal of coal washery wastes attributed an increase in values of $k$ for mixtures over tailings alone to container wall effect, but did not attempt to quantify mixture structure, or values of $e_t$.

Once constructed, it is expected that the value of $k$ of mixtures will decrease with compression and consolidation due to self-weight loading. Possible failure mechanisms that would increase the value of $k$ include cracking due to desiccation, piping due to high
hydraulic gradients, and perhaps internal erosion of the tailings particles from the waste rock void space due to hydraulic transport under high hydraulic gradient.

In terms of design, a low value of $k$ may be obtained by selecting mixture ratio, $R$, and tailings solids content, $P$. As described above, mechanisms for reducing the value of $k$ with respect to tailings alone include reduction in area available for flow by inclusion of waste rock particles, increased flow path tortuosity, and reduction in $e_t$. While a maximum value of $R$ appears to be desirable to reduce the area available for flow, mixtures with mixture ratios higher than the “just filled” case will contain air and the value of $k$ will increase dramatically due to heterogeneity and preferential flow through the resulting voids. At mixture ratios less than the “just filled” case, the mixture will contain less rock, and more area will therefore be available for flow. Increasing $R$ will also have the effect of increasing preferential flow due to container wall effect, which will tend to increase values of $k$. Dominance of container wall effects versus area available for flow will depend on the particle size ratio, particle shape, and on absolute particle sizes. It is therefore likely that the tortuosity term is a function of not only the waste rock particle size ratio and particle shape, but also of the particle size ratio and shape of the tailings particles. For the mixtures of waste rock and tailings examined, the ideal value of $R$ for low permeability is the “just filled” case. For the lowest value of $k$ the value of $e_t$ should also be as low as possible, and therefore $P$ should be as high as possible.
8.4.3 Soil-water Characteristic Curves of Mixtures

The soil-water characteristic curves of tailings and waste rock are opposite extremes. At low matric suctions, tailings typically have higher water contents than waste rock alone. CIP tailings slurry thickened to 48% solids had a volumetric water content of 76%, and a gravimetric water content of 108%. In contrast, measurements of water content of the waste rock stockpile used for column fill construction indicated that, when dripping wet, the waste rock had a gravimetric water content near 4%. Mixtures had an initial water content that was closer to that of the waste rock. The initial condition of the lower, saturated portions of the column study mixture profiles had gravimetric water contents near 16%, or volumetric water contents near 36%. The upper unsaturated portions of the profiles had lower water contents. The proportions of rock and the tailings have opposite effects on the capacity of a mixture to store water. The addition of waste rock to tailings alone has the effect of increasing solids volume and therefore reducing the volume of water stored in a mixture. The addition of tailings to waste rock increases fine particle content and therefore increases the capacity of a mixture to store water by capillary effects. The fine particle content also influences the ability of a mixture to retain water under matric suction, which is described by the soil-water characteristic curve (SWCC).

One of the key points on the SWCC is the air entry value (AEV), which is defined as "matric suction value that must be exceeded before air recedes into the soil pores" (Fredlund and Rahardjo 1993). The waste rock examined had a value of AEV that was, by observation, less than one kPa. The values of AEV for the mixture and CIP tailings
specimens were difficult to determine due to the volume change accompanying increases in matric suctions, but were likely greater than 60 kPa.

What is more important than the AEV for mixtures studied here is the fact that the mixtures in the columns were observed to crack at matric suctions greater than 40 kPa. Cracking is a change in macro structure that dramatically increases hydraulic conductivity and air permeability. Mechanisms that will close or heal cracks include an increase in total stress, clogging of fractures by particles, and swelling of clay particles (Eigenbrod 2003). It is unlikely that an increase in effective stress would close cracks in the tailings matrix of a mixture because the stress applied to a mixture would be carried primarily by the waste rock skeleton. Clogging could partially seal the cracks, but is unlikely to restore a low value of $k$. The closing of cracks by swelling of clay particles depends on the clay content in the tailings. For the CIP tailings examined, the change in mixture structure resulting from desiccation was considered irreversible. With loss of water the tailings shrink within the void space of the rock. The volume of the waste rock skeleton changes slightly, but the volume change of the tailings matrix is much greater. Observations from the upper portion of Column 1 after a prolonged drying period indicated a structure with a waste rock skeleton with voids containing both air and tailings. It is noted that long periods of drainage (months) without access to rainfall were required to generate higher matric suctions and associated cracks.

Wilson et al. (2003a) stated that key design criteria for control of oxidation included a high AEV and low value of $k$. The AEV for the mixtures examined here was high, and
the value of \( k \) was low. However, the volume change of the tailings matrix can induce cracking that will render a high AEV insignificant by greatly increasing the value of \( k \).

For design of the SWCC of mixtures of waste rock and tailings, it is assumed that a high AEV and low susceptibility to cracking are desirable. Both properties will depend on the type and density of the tailings used. In terms of the particle model presented in Chapter Three, susceptibility to cracking will be worst at high values of \( e_t \). The value of \( e_t \) may be decreased by raising the value of \( P \) and by consolidating the mixture. Mixtures with values of \( R \) that are less than the "just filled" will be unsaturated. Mixtures with values of \( R \) that are greater than the "just filled" case will be saturated, but will have tailings dominated structures. Again, compression of a mixture that is initially slightly tailings dominated may provide a lower value of \( e_t \), which will increase the matric suction required to induce cracking in the tailings matrix. In summary, for high AEV and low susceptibility to cracking, values of \( R \) should correspond to the "just filled" case, or be slightly lower, and values of \( P \) should be maximized.

It should be noted that tailings will undergo severe volume change and cracking upon desiccation. Also, waste rock is typically initially unsaturated and remains unsaturated unless placed under water. Mixtures offer superior water retention compared to waste rock, are significantly less susceptible to cracking than tailings alone.

**8.5 Summary and Engineering Significance**

The logical framework for evaluating mixtures as a mine waste disposal technique involves relating mixture design variables to behaviour through structure. Particle
packing theory forms a basis for relating mixture design to structure, and hence to behaviour. Initial structure may be designed through selection of values of mixture ratio, \( R \), and initial tailings solids content, \( P \). Table 8.1 presents a summary of optimum values for design criteria.

Table 8.1 Optimum values for mixture design variables.

<table>
<thead>
<tr>
<th>Property</th>
<th>Mixture ratio, ( R )</th>
<th>Source Tailings Solids Content, ( P )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressibility</td>
<td>- minimum volume “just filled” case or slightly higher</td>
<td>maximize</td>
</tr>
<tr>
<td></td>
<td>- change</td>
<td></td>
</tr>
<tr>
<td>Hydraulic Conductivity</td>
<td>- minimum ( k ) “just filled” case or slightly lower</td>
<td>maximize</td>
</tr>
<tr>
<td>SWCC</td>
<td>- high AEV “just filled” case or slightly lower</td>
<td>maximize</td>
</tr>
<tr>
<td></td>
<td>- minimum cracking</td>
<td></td>
</tr>
</tbody>
</table>

In terms of compressibility, hydraulic conductivity, and SWCC, the ideal mixture design appears to have a mixture ratio near the just filled case or slightly lower, and maximum source tailings solids content.

Mixtures had lower compressibility than tailings alone due to the presence of a load bearing waste rock skeleton. In comparison, tailings alone were highly compressible. Mixtures had low values of hydraulic conductivity and were able to maintain saturation due to the presence of a tailings matrix. In comparison, waste rock has a high hydraulic conductivity and is almost always unsaturated.
Using the above framework, key issues that further require research include:

i) acid rock drainage,

ii) potential for liquefaction,

iii) trafficability and reclamation issues,

iv) operational methodology for construction and mixing,

v) costs, and

vi) other potential failure mechanisms such as piping, desiccation cracking, soil dispersion, erosion, and slope stability.
CHAPTER NINE. CONCLUSIONS

9.1 Overview

This thesis evaluates an alternative mine waste disposal technique where mine waste rock and tailings are combined as homogenous mixtures for disposal. Select geotechnical behaviours of mixtures were compared to tailings alone and waste rock alone. The thesis provides a fundamental theoretical basis for understanding mixture design and for interpreting behaviour with respect to particle structure.

The thesis is divided into three parts. The first part is primarily composed of literature review and includes Chapters One, Two and Three. Chapter One defines problems resulting from current methods of mine waste disposal. Chapter Two reviews past work on co-disposal of mine wastes. Chapter Three reviews particle packing theory for mixtures of waste rock and tailings, reviews studies of soil mixtures for relationships between mixture structure and behaviour, and introduces a conceptual particle model for mixture structure. The particle model provides a basis for understanding the design and behaviour of mixtures through structure.

The second part of the thesis, including Chapters Four, Five, and Six, presents practical investigations and results. Chapter Four describes the experimental methodology used to evaluate mixture behaviour. One type of waste rock, one type of tailings and mixtures of the same waste rock and tailings were investigated. The waste rock and tailings were taken “as-produced” from an operating metal mine, and were not altered with the exception of scalping the waste rock of larger sizes to allow testing. Tailings solids
content was also examined as a design variable. Chapters Five presents results from laboratory investigations including consolidation testing, hydraulic conductivity testing alternated with static loading, pressure plate tests for soil-water characteristic curves, mixture trials and also investigation of tailings rheology. Chapter Six presents results from a two year meso-scale column study of self-weight consolidation investigating the volume change, pore-water pressure response, and settlement of mixtures of waste rock and tailings.

The third part of the thesis, including Chapters Seven, Eight and Nine presents analyses, discussion, and a summary of findings and conclusions. Chapter Seven includes a comparison of laboratory and column study findings. Where possible, the findings have been generalized to principles that may be applied to other types of waste rock and tailings. A method to predict compressibility is presented with an equation relating hydraulic conductivity to parent material properties through particle structure. Chapter Eight includes a discussion of the results and analysis with recommendations for the values of design variables of mixture ratio and source tailings solids content. Chapter Nine presents the conclusions of the work.

9.2 Conclusions

Conclusions of the thesis are drawn from both the literature review and from analysis of experimental evidence and observations from practical investigations. General conclusions include:
1. No theoretical basis for linking mixture design to behaviour has previously been proposed for mixtures of waste rock and tailings. Little work has been done to investigate mixtures of mine waste rock and tailings.

2. Mixtures of coarse waste rock and fine tailings are well represented by particle packing theory for binary mixtures presented by Furnas (1928). Mixtures of coarse waste rock and fine tailings are typically gap-graded with stepped particle size distributions. The large difference in average particle diameter allows application of particle packing theory for binary mixtures. Mixtures can be conceptualized in terms of a structural particle model described quantitatively by a waste rock skeleton void ratio, $e_r$, and tailings matrix void ratio, $e_t$.

3. The particle model for mixtures of waste rock and tailings is useful for design. Design variables for mixtures of waste rock and tailings include mixture ratio, $R$, defined as waste rock to tailings by dry mass, and tailings solids content, $P$, defined as mass of tailings solids divided by mass total. Particle packing theory indicates that a design optimum for initial maximum density occurs at a value of $R$ where the tailings "just fill" the void space of the waste rock. The porosity and water content of mixtures constructed in the laboratory were successfully predicted using the model.

4. While particle packing theory indicates a maximum value of $P$ for maximum mixture density, laboratory mixture trials and measurements of tailings rheology indicate that there are limits to the value of $P$ that may be constructed. The limits to $P$ suitable for preparing mixtures by hand can be defined independently of material type by a rheological yield stress.
5. The particle model for mixtures of waste rock and tailings is useful for interpreting and predicting geotechnical behaviours. Compressibility of mixtures was found to be largely dependent on changes in waste rock skeleton void ratio. The hydraulic conductivity of mixtures was found to be largely dependent on tailings matrix void ratio. Both the compressibility and hydraulic conductivity of mixtures may be predicted using the particle model and the properties of the parent waste rock and tailings.

6. Concepts of particle packing theory may be applied universally to soil mixtures. The review of studies of the geotechnical behaviours of soil mixtures indicated that behaviour is dependent on mixture particle structure. Most investigations were found to be specific to one area of research, e.g. shear strength, or permeability and volume change behaviour. The majority of investigations examined two-component soil mixtures with a large particle size ratio, e.g. sand-bentonite mixtures. Although different behaviours of different materials were independently examined, investigators commonly invoked mixture structure to provide mechanistic explanations for complex behaviours.

Conclusions specific to the tailings and waste rock examined include:

7. The mixtures examined had the compressibility of waste rock alone and were much less compressible than tailings alone. The finding is attributed to the presence of a load bearing waste rock skeleton. Mixture structure was noted to change with compression from one that was initially tailings dominated to one that was waste rock dominated.
8. Mixtures had values of hydraulic conductivity that were similar to tailings alone. Waste rock had a hydraulic conductivity that was several orders of magnitude higher than the tailings or mixtures.

9. Mixtures and had a high air entry value (AEV), similar to tailings, while waste rock desaturated at low matric suctions. The finding is attributed to the presence of a fine-grained tailings matrix.

With respect to existing methods of mine waste management:

10. Mixtures will reduce the potential for ARD relative to waste rock alone due to a low hydraulic conductivity and high air entry value. Mixtures had hydraulic conductivities that were orders of magnitude lower than rock alone and remained saturated for significant periods of time relative to waste rock.

Significant issues identified include:

11. Mixtures constructed of tailings at high initial water contents may be prone to cracking under prolonged drying conditions. With prolonged drainage and desiccation, mixtures were observed to develop cracks in the tailings matrix. Cracking has the potential to increase hydraulic conductivity and air permeability.

12. Mixture construction and handling is a significant issue. Two of three mixtures were found to segregate within a concrete transit mixer during construction of the column study. Waste rock alone was sorted by the action of the same process. Further research is required to provide a suitable construction technique.
9.3 Future Investigations

The research of mixtures of waste rock and tailings is a new field. The investigations presented in this thesis have focused on volume change, permeability and soil-water retention. Other geotechnical behaviours requiring investigation include susceptibility to liquefaction failure, shear strength, susceptibility to piping failure, geochemistry, freeze-thaw and climate interaction, cracking, and any anticipated modes of failure. Cost benefit analysis, and methods of implementation are also crucial to the development of a new mine waste disposal technique.
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