MECHANICAL RESPONSE OF HIGHLY GAP-GRADED MIXTURES OF WASTE ROCK AND TAILINGS (PASTE ROCK)

by

ALI KHALILI

B.Sc., Sharif University of Technology, 2001
M.Sc. Sharif University of Technology, 2003

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF
THE REQUIREMENTS FOR THE DEGREE OF

DOCTOR OF PHILOSOPHY

in

THE FACULTY OF GRADUATE STUDIES

(Civil Engineering)

THE UNIVERSITY OF BRITISH COLUMBIA
(Vancouver)

June 2009

Abstract

The mixing of mine tailings and waste rock to form "paste rock" prior to disposal is now receiving significant attention from the point of view of sustainable mine waste management practice. This approach has been viewed as a favourable alternative to traditional methods of mine waste disposal because paste rock has the potential to overcome deficiencies, such as acid rock drainage and mechanical instability, associated with traditional methods of mine waste disposal. In consideration of the current limited understanding of the fundamental mechanical response, a systematic laboratory triaxial testing research program was undertaken on paste rock specimens prepared such that the tailings would "just fill" the void spaces between the coarse-particle skeleton. A new "slurry displacement" method was developed for reconstitution of saturated, uniform/homogeneous specimens of highly gap-graded paste rock for triaxial testing. Undrained cyclic triaxial tests indicated that reconstituted paste rock displayed "cyclic-mobility-type" strain development. Strain-softening accompanied by loss of shear

strength did not manifest regardless of the applied cyclic stress ratio (CSR). The results suggest that the material is not likely to experience flow deformation under monotonic (static) and/or cyclic loading conditions at least up to the tested initial effective confining stress conditions ≤ 400 kPa. The behaviour of paste rock was noted to be more similar to the behaviour of rock-only material than that of tailings-only material indicating that the rock skeleton mostly controls the shear resistance in "just filled" paste rock. This finding is in accord with the behaviour of paste rock observed from one-dimensional consolidation tests. In relative terms, paste rock has a higher potential for strain development under a given cyclic stress ratio and number of load cycles in comparison to tailings-only and rock-only materials. The presence of tailings in the pore space between the rock particles appears to decrease the ability of the rock particles to engage contact and develop inter-particle stresses in comparison to the case with rock-only material.

Table of Contents

List of Tables

List of Figures

Acknowledgements

The research work presented in this thesis was funded by the Natural sciences and Engineering Research Council (NSERC) of Canada, Barrick Gold and Inco Ltd. Their support is greatly appreciated.

I wish to thank my supervisor Dr. Wijewickreme for his great encouragement, support and guidance and my co-supervisor Dr. Wilson for his valuable inputs and comments.

Special thanks go to all the graduate students at UBC graduate geotechnical laboratory for their support and valuable discussions.

Dedication

To my parents and family whose constant love and support made this thesis possible.

Statement of Co-Authorship

I was responsible for performing all laboratory related research work including all laboratory tests and data reduction work. I was also responsible for initially writing all the chapters in this thesis. The contents of chapters two to five were reviewed by Dr. Wijewickreme and based on the comments received, these chapters were edited by me prior to submission.

Chapter 1.

Introduction and Background

1.1. Background

Mineral resources have played a major role in shaping the human life style for many millennia, and the expectation is that this role will continue with increasing intensity in the future. Minerals of various kinds can commonly be found in the earth's crust and mining/extraction processes involve physical and chemical modification of the natural ore containing the minerals. Usually the extracted minerals form only a very small percentage, in turn, leaving behind large quantities of waste in various forms at the mine site. This clearly has made the management of waste a key component in a mining operation.

The two most common mine waste streams arising from mining operations are coarse grained waste rock and relatively fine-grained tailings. Commonly used mine waste disposal methods, involve storing each waste stream separately. Waste rock is hauled by trucks and stored in surface dumps or stockpiles and tailings are pumped in the form of slurry through pipelines to surface tailings impoundments. Each of the waste storages is commonly associated with the potential for specific environmental damages or mechanical instability, which might result in loss of resources and human life.

1.1.1 Waste rock

In the surface mining process, reaching the ore containing the commercial minerals typically requires the removal of overburden rock using blast-excavation techniques. Fractured rock from such blasting usually consists of angular particles of varying size; particle sizes of up to one meter are common among mine waste rock. The porous nature of waste rock allows oxygen and the water (from precipitation) to flow relatively freely between the particles. In the case of sulphide bearing minerals, chemical weathering occurs due to the reaction between water, oxygen, and the sulphide, further catalyzed by naturally occurring bacteria to produce usually toxic effluent called acid rock drainage (ARD). ARD is a metal rich, low pH liquid, which has or can have severe environmental consequences.

Various methods have been proposed to overcome ARD effects ranging from the addition of base materials to neutralize the ARD (Lapakko et al. 2000), or the use of bactericides to slow down the reaction leading to ARD in the first place (Olson et al. 2003), to the use of barriers to limit the oxygen entry into the dump and reduce the reaction rate (Lundgren 2001). Considering the amount of waste rock generated at mine sites, some of the proposed methods such as addition of bactericides or neutralizers can be very expensive and other techniques such as cover systems can lose their efficiency over time as a result of their deterioration or development of cracks. Currently, there are no proven methods to eliminate the ARD problem completely although most of the methods are known to be effective to some extent (Wickland 2006).

1.1.2 Tailings

Tailings, the other common waste stream generated by mine operations, are the remainder of the ore and chemical additives after the valued mineral(s) have been removed. Tailings are fine-grained and their particle sizes range from clay to sand size. Because of the nature of deposition, tailings are well known for their high degree of saturation and long consolidation times, leading to relatively weak shear strengths.

Tailings impoundments commonly built from the coarser fraction of the tailings material itself. The tailings particles are separated into fine and coarse fractions on site (typically by use of cycloning methods) and the larger particles are used for construction of the

retaining dykes/dams. In this method, only a small starter dam is built from non-tailings material and the rest of the impoundment is constructed completely from coarse tailings, thereby, resulting in cheaper and more economic tailings storage solutions.

Combined with the high degree of saturation and relatively loose densities, the "as-placed" tailings material is generally considered susceptible to liquefaction. Liquefaction of tailings can lead to instability of the retaining dykes and failure of tailings impoundments, with potentially severe consequences including loss of life and multi-faceted environmental, social, and economic impacts. The chemistry of tailings depends on the type of mineral, extraction type and neutralizing process (treatment) but it usually consists of the ore particles, water and reagents from the extraction process (Wickland 2006).

There have been more than 70 tailings impoundment failures reported since 1950 (http://www.wise-uranium.org/mdaf.html). Although many of those are reported to have failed as a result of static liquefaction, cyclic behaviour of tailings is also of great interest as many of the impoundments located in seismic areas. The nature of deposition, combined with the almost-saturated conditions of tailings makes them susceptible to liquefaction. On failure of a tailings impoundment, the generally saturated tailings may flow for kilometers, sometimes covering areas measured in hectares and cause environmental damages and loss of human life (Wagener et al. 1997, Wickland 2006).

1.1.3 Waste rock / tailings mixtures

To overcome the difficulties and reduce the potential environmental damages associated with traditional methods of dealing with mine waste, new methods are continually being explored by the mining industry. Several methods have been proposed and tried, each having its own merits and deficiencies. One of the methods that has recently received significant attention is to blend waste rock and tailings to form a homogeneous mixture (Wickland 2006). It has been suggested that by doing so, tailings particles fill the voids of the waste rock which would reduce oxygen from flowing between the waste rock particles and consequently has the potential to reduce ARD problems. In addition, waste rock particles would form a load bearing matrix as part of the mixture which potentially can improve the mechanical behaviour of the mixture. It has been proposed that this mixture (i.e. also termed "paste rock", Junquiera et al. 2007) can be used as a cover or form the body of the waste deposit systems. Potential advantages of paste rock include the generation of less waste volume for disposal, overall cost reduction, and simpler waste management systems from an overall view point. Paste rock would generally form a highly gap-graded material which can be classified as silty gravel or gravelly silt. So far, only limited effort has been made to study the preparation of homogenous mixtures of waste rock and tailings. Understanding of the mechanical behaviour of such mixtures forms a critical part in applying this method of disposal to practical applications. Although the available information from past work pertaining to gap-graded soils provides some insight, little information is available directly on paste rock.

With this background, a research program was proposed to study the mechanical behaviour of paste rock under shear loading conditions. In order to provide a background and basis for the proposed work, previous studies conducted on gap-graded soils in general and tailings/waste-rock mixtures in particular are presented in this chapter. Firstly, common approaches used for mixing waste rock and tailings are discussed. This is followed by a presentation on the current state of knowledge on the mechanical properties of the mixtures of waste rock and tailings (e.g. consolidation, monotonic and cyclic shear loading behaviour). The mechanical response of mixtures is expected to be influenced by the mechanical behaviour of their constituents (i.e., the behaviour of waste rock and tailings as separate materials). The present understanding of the mechanical response of gravels and tailings is also summarized. Finally, some practical considerations regarding laboratory testing of gravels and the post-cyclic residual strength of soils are discussed.

1.2. Mixtures of Waste Rock and Tailings

Various methods practically considered/adopted for disposal of mine waste as mixtures of waste rock and tailings, presented in relation to the degree of mixing, are presented in Table 1.1. Studies have shown that increased degree of mixing reduces the environmentally harmful effects including potential for acid rock drainage (Wickland 2006, Johnson et al 1995, Williams 1997). In addition, mixing waste rock and tailings

would also offer other advantages such as elimination or reduction in the size of tailings impoundment which, in turn, can lead to potential financial savings.

1.2.1 Existing approaches to design of mixtures of waste rock and tailings

Interaction between individual coarse and fine particle fractions plays an important role in determining the overall mechanical behaviour of a waste rock and tailings mixture; therefore, it is reasonable to expect the mixture to inherit some characteristics from each of the parent constituents in addition to the dependency of its mechanical behaviour to the mixture ratio.

The approaches that have been proposed for determining the appropriate waste-rock/tailings for mixture designs include:

1- Preliminary empirical approaches developed without detailed property characterization of the materials;
2- Procedural approaches similar to those used in asphalt and/or concrete technology; and
3- Approaches partly based on particle packing theory.

Most of the preliminary work on mixtures of tailings and waste-rock falls into the first approach above, which can more accurately be classified as studies highlighting some beneficial aspects of mixing waste-rock and tailings rather than design specific

recommendations. Brawner (1978), for example, proposed mixing of waste rock and tailings as a beneficial procedure and highlighted the potential advantages and disadvantages over traditional methods. Eger et al. (1984) studied the potential ARD reduction in mixtures of waste rock and tailings and recommended the use of mixtures as a means to reduce metal leaching and ARD without specific guidelines on the optimum mixture ratio. No specific design recommendations could be derived from studies involving this first approach.

Wilson et al. (2003a, 2003b) developed mixing procedures based on the techniques used in concrete technology. The method involved mixing tailings and waste rock with the ratios which yield gradation curves similar to particle size distribution of glacial till, which is known to have near ideal properties in terms of strength and hydraulic conductivity. Fines et al. (2003) and Williams et al. (2003) also used similar approaches to develop mixtures of waste rock, tailings, slag and clay to produce well-graded particle size distribution although Fines et al. (2003) did not suggest any design criteria or gradation.

As acknowledged by Wickland (2006), approaches developed based on concrete mixing processes, usually require an extensive amount of pre-mixing preparation of the aggregates (such as sorting, weighing, etc.). Also, the presence of batching plants for crushing, sorting and re-mixing of rock particles may be needed as is common for concrete mixing facilities. In addition, it is very common for the waste rock and tailings

to have completely different particle sizes ranges leading to mixtures not having a specific particle size range; in such instances, addition of external soil particles of specific size may be necessary to produce a well graded mixture, all leading to procedural complexities and associated costs.

Considering the significant difference in size between waste rock and tailings particles in most instances, from a point of view of simplification, a mixture of waste rock/tailings can be assumed to comprise of essentially two average particle sizes. Such mixtures are commonly referred to as binary mixtures in particle packing theory. The mixture ratio in a binary mix will determine the status of voids among bigger particles and the percentage filled with smaller particles. If the ratio of tailings to waste rock is smaller than a certain amount, the waste rock particles will be in contact with each other, with the tailings particles partially filling some of the voids while leaving some others unfilled. Increasing the tailings portion will eventually lead to a state at which tailings particles fill all the voids between waste rock particles while still keeping them in contact with each other. Such a state is referred to as "just filled" state by Williams et al. (1995). An additional increase in the tailings portion leads to creation of a tailings matrix in which rock particles are "floating".

Williams et al. (1995) performed a series of trial-and-error mixings during pumped co-disposal of coal mine waste, and concluded that the ideal mix ratio is the one that leads to a just-filled state. They identified some factors such as low mixture ratio,

gap-graded-ness of the mixture, etc. that would affect the degree of segregation during the mixing process. Compressibility of the waste rock matrix is commonly much lower than the compressibility of the tailings fraction and with a special reference to particle packing theory, Wickland (2006) confirmed that "just filling" the waste rock particle matrix with tailings would result in a mixture with low compressibility (close to that of the waste rock matrix) while giving rise to the maximum tailings storage and reduced potential for ARD.

1.2.2 Current knowledge on the mechanical behaviour of binary mixtures of geo-materials

1.2.2.1 Consolidation characteristics and hydraulic conductivity

Fukue et al. (1986) studied the consolidation response of mixtures of sand and clay and concluded that the clay matrix dominates the consolidation properties of the mixtures until the sand fraction reaches a point where sand particles come in contact with each other. Beyond this point, a low rate of consolidation was observed, which was attributed to the frictional behaviour of the sand particle matrix. The void ratio of the sand skeleton in this state was defined "threshold void ratio", which varied from 1.25 to 1.4 for different mixture ratios. Although Fukue et al. (1986) did not explicitly discuss complications of particle packing theory with respect to their research, it is clear that the threshold void ratios were slightly higher than the maximum void ratio of the sand alone

(without any clay) which corresponds to clay contents slightly higher than those needed for "just-filling" the voids between the sand particles.

Wagg and Konard (1990) studied the hydraulic conductivity and consolidation behaviour of mixtures of silt and clay and found that, for the material used, a clay content of 30% or more is the threshold between silt-dominated and clay-dominated behaviour. A binary mixture model was adopted to explain the behaviour of the mixture. Wagg and Konard (1990) postulated two void types representing the clay and silt, and they also recognized that some large pore spaces between the particles might not be filled with soil particles for low clay contents, in turn, leading to silt-dominated behaviour.

Mollins et al. (1996) performed a series of special swelling tests on mixtures of sand and bentonite clay and conceptualized the behaviour using a binary mixture model. It was noticed that above a certain vertical effective stress, sand particles would dominate the behaviour of the mixture. This threshold vertical effective stress was found to be dependent on the clay content of the mixture which increased with increasing clay to sand ratio.

Kumar and Muir Wood (1997) conducted a series of consolidation and triaxial tests on mixtures of kaolin clay and coarse sand. They found that for clay contents of 40% to 100%, the response of the specimens was almost identical (in both consolidation and triaxial testing) irrespective of the clay content. As the sand content increased above

60%, the behaviour of the mixture changed significantly both in consolidation and monotonic shear, which was concluded to be the result of sand particles interacting with each other. Although Kumar and Muir Wood (1997) suggested relationships for the calculation of the sand and clay void ratios, they did not address the performance of the mixtures with respect to particle packing theory or the binary mixture models.

Studds et al. (1998) proposed a theoretical model to predict the void ratio vs. applied stress for a mixture of bentonite clay and sand based on the assumption that the applied stress will be equal to the sum of stresses absorbed by the clay and the sand separately. They further assumed that the initial porosity of the sand matrix does not influence the load deformation characteristics of the sand fraction; this was not true when sand particles lose contact with each other. Studds et al. (1998) also studied the effect of various permeants on the hydraulic conductivity of the mixtures.

Stewart et al. (2003) used the model proposed by Studds et al. (1998) to study the swelling properties of the mixtures of sand and bentonite clay. They concluded that the addition of 20% sands to the bentonite clay can reduce the swelling of the mixture significantly. It was proposed that for sand ratios of less than 20%, the granular particles would be separated from each other as a result of the swelling clay between the particles.

Wickland (2006) performed a series of laboratory consolidation and hydraulic conductivity tests and meso-scale column studies on some sedimentary rock and carbon

in pulp (CIP) tailings and various mixtures of the materials (the same material used for this research) with the intention of finding the most suitable mixture ratio for practical applications. (Note: CIP tailings are considered to be fine grained relative to other types of tailings; they are produced by crushing, grinding, flotation, pyrite concentrate pressure oxidation and carbon-in-pulp gold recovery of the ore. Activated carbon used for gold adsorption is removed during processing, but minor amounts of carbon particles remain in the tailings. Tailings are treated with calcium hypochlorite to destruct any cyanide remaining from processing).

A binary model approach was introduced for explaining the volume change characteristics and hydraulic conductivity of mixtures of various ratios. Wickland (2006) concluded that up to a certain ratio, the tailings particles will passively fill the voids between the rock particles and do not contribute to any load transfer between them. If this condition prevails, volumetric deformation of the mixture during consolidation would be very close to that of the rock skeleton with limited volumetric strain taking place during consolidation. Tailings paste that fills the voids would stay unconsolidated even after exposing the mixture to high loads. The hydraulic conductivity of the mixture in this case would depend on the mixture ratio, and it would become closer to that of the tailings slurry as the rock to tailings ratio decreases. If the tailings content increases beyond a certain threshold, the rock particles would “float” into the tailings mixture without being effective in transferring the load. In this case, consolidation behaviour of the mixture would be very similar to that of the tailings slurry alone, with large

volumetric strains occurring over relatively long time durations. The hydraulic conductivity of the mixture would also be very close to that of the tailings slurry, since the contribution of the rock particles to the overall permeability is relatively less significant.

Wickland (2006) used the just-filled concept previously proposed by Williams et al (1995) to explain some mechanical properties of the mixtures. It was concluded that the just filled paste rock can offer the advantages of the low hydraulic conductivity of tailings alone and the low compressibility characteristics of waste rock alone, and therefore is most desirable for practical applications in order to minimize the space required for the disposal of these waste materials.

1.2.2.2 Monotonic shear response

Numerous studies have been completed to advance the knowledge of the monotonic shear response of sands and clays (e.g., Rowe 1962; Schofield and Wroth 1968; Ladd and Foote 1974; Vaid and Chern 1985). This knowledge-base also provides the fundamental basis for understanding the shear behaviour of mixtures of geomaterials. Mixtures and granular material in general, can show responses similar to those of sands or clays, or both depending on the composition of the mixtures and interaction between their components. Studies on the shear strength characteristics of various soil mixtures started to receive attention during the 1960s.

Holtz and Ellis (1961) studied the monotonic shear response of mixtures of clay and gravel and mixtures of sand and gravel using data from triaxial testing of large unsaturated specimens. They found that an increase of gravel content beyond a certain threshold would lead to an increase in the shear strength of the mixture. Holtz and Ellis (1961) suggested that at a certain gravel content, further addition of gravel did not affect the shear characteristics of the mixture in a significant manner; this observation was attributed to the inability of gravel particles to interact with each other at such low gravel contents. The threshold at which the transition occurs between clay-like behaviour and gravel-like behaviour was found to be around 50% gravel content. Furthermore, they found that the failure envelope of clay-gravel mixtures was different in nature from that of sand-gravel mixtures; i.e., clay-gravel failure lines were straight (reflecting stress history normalizability of fine-grained soils) whereas failure lines were curved in the case of sand-gravel mixtures (reflecting the influence of stress increase in reducing the dilatancy of the coarse material).

Statham (1974) performed a series of tests on binary mixtures of spherical glass of various sizes as well as mixtures of spherical glass with rounded river sand. Particular attention was given to the packing status of the grains and the effect of mixture ratio on the initial and residual friction angle of the mixtures. It was found that the residual friction angle reached a peak when the total porosity of the mixture reached a minimum. Statham (1974) also showed that the minimum porosity of the binary mixtures coincided with the state at which small particles just fill the voids of the coarser skeleton.

Lupini et al. (1981) studied the behaviour of the mixtures of clay-clay, sand-clay, silt-sand, and silt-clay with a particular emphasize on the packing orientation of the particles and the failure mechanisms involved. Three types of shear behaviour modes were recognized: (i) turbulent shear behaviour, which is common in sands and other particulate materials. In this mode of shearing, particles roll over each other during failure, accompanies by local dilatancy; (ii) sliding shear behaviour, which can be identified by clear sliding planes within specimens; and (iii) transitional shear behaviour, which is a combination of modes (i) and (ii) above. When different components of the mixtures act in two different modes (i.e., turbulent and sliding), the behaviour of the mixture falls into the transitional shear mode at a threshold mixture ratio. In that situation, small changes in the gradation and/or mixture ratio of the material can significantly change the shear response of the mixture. Lupini et al (1981) also recognized that particle size difference and non-uniformity of different mixture components can affect the threshold mixture ratio and or transitional behaviour of the material.

Thevanayagam (1999), Thevanayagam and Mohan (2000) and Thevanayagam and Liang (2001) performed a series of undrained triaxial tests on mixtures of silts and sand with different mixture ratios. Thevanayagam (1999) introduced two variations of the global void ratio (e): (i) inter-granular void ratio (e_s) (similar to Lupini et al. 1981); and (ii) inter-fine void ratio (e_f) as follows:

Inter-fine void ratio = $e_f = e / f_c$ Eq (1)

Inter-granular void ratio = $e_c = (e + f_c) / (1 - f_c)$ Eq (2)

where f_c is the fines content by dry weight of the coarser particles. Thevanayagam (1999) showed that these indices are much more representative of the contact density between particles than the global void ratio (e) commonly used in geo-materials. Thevanayagam and Mohan (2000) discussed the behaviours of different mixture ratios (Figure 1.1) and concluded that the large-strain undrained shear strength is dependent on the inter-granular void ratio (e_c). Up to a certain inter-granular void ratio, the behaviour of the mixture is very similar to that of clean sand with the same void ratio, whiles the shear strength of more silty sand decreases and is dependent on the initial confining pressure. The threshold void ratio at which transition occurs depends on the fines content or the mixture ratio. Above a certain fines content, the behaviour of the mixture becomes more similar to that of the host fine fraction without inclusion of sand particles.

Vallejo (2001) studied the behaviour of mixtures of rounded glass beads of 5 mm and 0.4 mm in diameter. Vallejo used particle packing theory to show the effect of mixture ratio on the total porosity of the mixture and suggested that the addition of fines decreases the porosity of the mixture until it reaches a minimum at a fines content of about 26%. Such obvious trends have previously been observed by Furnas (1928) with consideration given to the packing arrangements of the spheres. Vallejo (2001) also showed that the transition between the coarse-dominated behaviour and fine-dominated behaviour occurred between coarse grained concentrations of 40% to 70% (by weight), and therefore he concluded that the transition was not as abrupt as suggested by the theory. This might be

due to the fact that the glass beads were all of the same material and might have had similar modes of shear as suggested by Lupini et al. (1981).

Tan et al. (1994) studied mixtures of marine clay and sand of different mixture ratios and concluded that the addition of sand to the clay slurry will increase the shear strength of the mixture as a result of the proximity of sand particles. The void ratio at which sand particles start to influence the behaviour of the mixture was found to be around 5. They also studied the variation in the void ratio of the mixtures with various fines contents and found that for the material used, the minimum void ratio occurs at about 20% fines, although the transition between sand-like behaviour and fine-like behaviour was found to be gradual around this value. It was also observed that the gradation of the parent sand (uniformity and/or well-graded-ness) did not have significant influence on the undrained monotonic response of the mixture. Pitman et al. (1993) performed a series of tests on mixtures of Ottawa sand with plastic and non-plastic fines, and concluded that the addition of fines decreases the strain softening behaviour of the mixture. They also concluded that plasticity of the fines had no effect on the overall behaviour of the mixture, and the mixture ratio is a more important factor.

Lade and Yamamuro (1997) and Yamamuro and Lade (1998) performed a series of undrained triaxial tests on mixtures of four different types of clean sand and non-plastic silt. Their results seemed to contradict the studies published by other researchers since it was found that the addition of fines decreased the static liquefaction resistance of the

mixtures even when the fine content was relatively small. Lade and Yamamuro (1997) further investigated the results and concluded that although the total density of the mixture decreased with the addition of fines, the skeleton (coarser fraction) void ratio increased as a result of the special specimen preparation technique employed. It was further noticed that the presence of loose fine particles between coarser grains at low fines content can create meta-stable structures which tend to be collapsible and increase the liquefaction tendency of the material.

Lade et al. (1998) used existing packing theories to explain the void ratio changes in mixtures of Nevada sand and silt with various mixture ratios. They showed that the minimum and maximum void ratios decrease with the addition of fines until all the voids of the coarser fraction are filled with fine particles; then they increase gradually with further increase in fines content. Lade et al. (1998) attributed the anomaly in the findings with the previous research by Lade and Yamamuro (1997) to the meta-stable structure formation between particles.

Naeini and Baziar (2004) noted similar findings to Lade and Yamamuro (1997) based on undrained triaxial tests performed on mixtures of sand and silt. They found that the addition of up to 35% silts decreased the peak and residual (large strain) strengths of the mixtures. Naeini and Baziar (2004) referred to the inter-granular void ratio similar to the concept introduced by Thevanayagam (1999), but did not use the concept to explain the observed behaviour of the silty sands. Their contradictory conclusion with respect to the

observations made by other researchers (Holtz and Ellis 1961, Tan et al. 1994, Pitman et al. 1994) might be due to the increase in the inter-granular void ratio of the mixture, as explained by Lade and Yamamuro (1997).

In an overall sense, it is clear from the aforementioned studies that the addition of fines to binary mixtures generally leads to a declining total void ratio at first, followed by an increasing total void ratio when the fines content passes a certain threshold. The threshold depends on the gradation of the fine and coarse fractions. Most researchers have correlated the total void ratio of the material with the shear strength, but some have observed that the void ratio of the coarser skeleton correlates better with shear strength when the fines content is small. As long as coarser particles are in contact with each other, the addition of fines seems to improve (or not reduce) the static shear strength characteristics of the mixtures.

1.2.2.3 Cyclic shear response

Kuerbis et al. (1988) performed a series of monotonic and cyclic tests on mixtures of tailings sand with a non-plastic silt and found that the addition of silt to the sand matrix would decrease the global void ratio almost linearly up to 20% silt content (similar to previous research discussed in Section 1.2.2.2), for a constant sand skeleton void ratio. They noticed that the addition of up to 20% silt causes little change to the monotonic and cyclic behaviour of the mixture which can only be explained by void ratio of the sand

skeleton. They suggested that the cyclic resistance and monotonic behaviour of sandy silts can be obtained by examining the clean sand matrix if the silt content is below a certain threshold.

Singh (1994) performed a series of tests on sand, silt, and their mixtures to find that the addition of silt will decrease the cyclic resistance if specimens have comparable global void ratios. Chien et al. (2002) also studied mixtures of sand and silt used at reclaimed sites and derived similar conclusions as Singh (1994) stating that "for a constant relative density, the liquefaction strength decreases as the fines content increases."

Amini and Qi (2000) and Amini and Sama (1999) performed a series of undrained cyclic triaxial tests on homogeneous and layered sandy silts and gravel-sand-silt mixtures and reached contrasting conclusions with respect to observations made by Singh (1994) and Chien et al. (2002) indicating that the addition of fines increases the cyclic resistance of the specimen in both cases. They did not make any reference to the inter-granular void ratio, but used fines content as a variable to categorize various specimens. Koester (1994) studied the behaviour of sand, silt, and clay mixtures based upon undrained monotonic and cyclic triaxial tests. They concluded that cyclic strength cannot be characterized on the basis of gradation alone. Koester (1994) also observed that cyclic resistance increases with the addition of fines up to a certain threshold, after which it starts to decrease.

Evans and Zhou (1995) performed a series of monotonic and cyclic tests on mixtures of sand and gravel and, similar to Singh (1994), concluded that liquefaction resistance decreases with increasing sand content. Evans and Zhou (1995) also suggested a relationship for estimating the liquefaction resistance of mixture based on mixture ratio and the properties of each of the constituents of the mixture.

In addition to the generally limited information available on the cyclic response of mixtures of geomaterials, many conclusions are contradictory. Lade and Yamamuro (1997) tried to explain some of the observed contradictions, by relating them to the microscopic particle structure. They suggested that the presence of fine particles between coarser particles creates a meta-stable structure that is highly collapsible during monotonic and cyclic loading – i.e., although the addition of fines increases the global density (i.e., decreases the global void ratio) it creates a load bearing skeleton that is more prone to liquefaction.

As indicated earlier, because of the contrasting particle size distributions, mixtures of waste rock and tailings can be considered binary mixtures, and it is expected that they would inherit some behavioural aspects of their constituents. Therefore, discussion of some key observations with respect to the mechanical response of gravels and silts in isolation was considered prudent in understanding the experimental data from the testing of tailings/waste-rock mixtures in this study.

1.3. Mechanical Response of Gravels

According to Unified Soil Classification System (ASTM D-2487) a soil is "coarse-grained" if more than 50% of the soil is retained on #200 sieve (0.075 mm); if more than 50% of the soil particles in the coarse-grained fraction are larger than 2.36 mm in size, the soil would then classify as "gravelly". Jamiolkowski et al. (2005) refined this further and defined gravelly soil to be coarse-grained soil with a fines content of less than 10%, and classified it as follows:

- Clean gravels, GC ≥ 90%;
- Sandy gravels, GC ≥ 50%; and
- Gravelly sands GC < 50%.

Undisturbed sampling of gravelly soils is extremely difficult and very expensive although there have been some advancements in the methods of sampling by means of in situ freezing (Jamiolkowski et al 2005, Yoshimi 1994, Yoshimi and Goto 1996). It was originally thought that gravelly soils are not prone to liquefaction because of their ability to dissipate excess pore pressure very quickly without influencing the soil strength; however, review of literature suggests that there are more than twenty reported cases of liquefaction in gravelly soils (Andrus 1994, Andrus et al 1992, Ishihara 1996). As a result, various research studies have been undertaken to study the cyclic and undrained monotonic behaviour of gravelly soils.

Kukosho (2004) performed a series of monotonic and cyclic tests on different reconstituted gravelly soils and investigated the effect of gradation on the behaviour of gravel. He concluded that relative density is one of the key factors affecting the liquefaction resistance of gravelly soils during cyclic loading, regardless of uniformity and/or gradation; however, he noted that, if the particles are weak and crushable, liquefaction resistance will decrease with increasing uniformity of gradation. He also concluded that the behaviour of gravels tends to be similar to other granular geo-materials in general (i.e. sands), with the exception that after large strains (i.e. 20-25%) in both monotonic and post-cyclic monotonic shear tests, the uniformity coefficient and gradation of the soil play a crucial role in defining the undrained shear strength of gravels. He showed that more uniformly-graded gravelly soils tend to show larger undrained shear strengths at large strains (> 20%) in case their particles are not crushable, and vice versa.

Tatsuoka and Shibuya (1991) and Tatsuoka et al. (1997) showed that the stress-strain response of gravelly soils is strain-rate dependent, with faster strain rates inducing higher stiffness. Jamiolkowski et al (2005) stated that critical state theory can be applied to gravelly materials and the shear strength envelope of gravelly soils is in fact curvilinear, which means the mobilized peak friction angle is not constant for a given gravelly soil at a specified initial state, but rather depends on the normal effective stress at failure.

Evans and Seed (1987) and Evans et al. (1992) studied the cyclic performance of Watsonville gravel under undrained cyclic triaxial loading with an emphasis on the effect of membrane penetration. After using a sand sluicing procedure to eliminate the membrane penetration effect it is clear from their results that all of the gradations used showed a cyclic mobility type of failure. Despite that, even at low relative densities no strain softening was observed during cyclic loading. They also stated that none of the specimens developed a 100% pore pressure ratio as a result of cyclic loading.

The presence of oversized particles in gravelly soils imposes practical difficulties on laboratory testing. Therefore, in situ testing has also gained credibility for assessing the strength and cyclic resistance of these geo-materials (Jamiolkowski et al. 2005, Hatanaka and Uchida 1996). Since obtaining standard penetration test (SPT) blow counts for gravelly soils can be very difficult, Daniel et al. (2003) have demonstrated the value of correlations between large penetration tests (LPT) and equivalent SPT blow counts. They have suggested that ordinary sand correlations can be used to estimate the strength parameters of gravelly soils using equivalent SPT blow counts (Daniel et al. 2003, Harder 1997). The equivalent SPT blow counts can be obtained by applying appropriate corrections for the effects of grain size on blow counts. There have also been some correlations established between the shear wave velocity and strength parameters of the gravelly soils (Ohta and Goto 1978, Andrus et al. 1992).

1.4. Behaviour of Fine-Grained Mine Tailings

The available published literature on the monotonic and cyclic shear behaviour of tailings materials is limited. Chern (1985) performed a series of tests on tailings sand to study the effect of particle shape on the mechanical behaviour of sands in general. He concluded that the consolidation effective stress is a determining factor in the behaviour of tailings sand whereas relative density is more important for Ottawa sand. He showed that under higher confining stresses tailings sand might be contractive and show a strain softening response.

Vick (1983) developed a compilation of liquefaction resistance ratios for different types of tailings based on monotonic and cyclic shear tests on undisturbed and reconstituted specimens. Wijewickreme et al. (2005) studied several research results presented by various authors and compiled a series of curves showing the cyclic resistance of different tailings types (Figure 1.2). They performed a series of constant volume cyclic direct simple shear (DSS) tests on specimens prepared from three different types of fine grained tailings. They concluded that under constant volume cyclic DSS loading, fine-grained tailings typically exhibit the cyclic mobility type of behaviour similar to the response of dense reconstituted sands.

Singh (1994) performed a series of triaxial tests on undisturbed and reconstituted glacial silt and found that the generation of pore pressure and axial strain during cyclic loading is very gradual in the case of undisturbed specimens. He postulated that inter-particle

bonds that formed over years can contribute to this phenomenon. He concluded that silts may not reach a 100% pore pressure ratio, in contrast to the behaviour observed for some sands.

1.5. Post-cyclic Monotonic Shear Response

There are three fundamental questions with regard to seismic performance of soils (Wijewickreme et al. 2005): (i) will liquefaction be triggered in significant zones of the soil foundation at the design earthquake; (ii) if so, could a bearing failure or flow slide occur; and (iii) if not, are the displacements tolerable?

Knowledge of the post-cyclic shear response of soils is crucial in assessing the post-liquefaction response of earth structures when addressing question (ii) above. The current state of practice in this regard involves conventional limit equilibrium analysis for assessing post-liquefaction stability, and if no flow slide condition is predicted, then analysis should be carried out to predict the permanent displacement induced by ground shaking.

Field experience during past earthquakes indicates that residual strengths (S_r) can be much lower than values obtained from undrained tests on undisturbed samples (Idriss and Boulanger 2008). This is postulated to be mainly due to the upward flow of water resulting from excess pore water pressure generation. Such flow can sometimes be

retarded by the presence of low permeability "barrier layers" causing water to collect in zones beneath the barrier. This may cause some zones or layers to dilate to a higher void ratio (void redistribution), and hence a lower critical state strength. In the limiting scenario, a water film may form at the interface beneath the barrier (Naesgaard et. al. 2005, Kokusho 2003).

Idriss and Boulanger (2007), Seed and Harder (1990), Stark and Mesri (1992), and Olson and Stark (2002) have developed correlations between residual strength S_r or residual strength ratio S_r/σ'_{vc} (where σ'_{vc} is initial vertical overburden effective stress), and SPT blow counts computed from the back-analyses of field case histories. Such back-analysis for estimation of S_r has been considered more suitable, since laboratory testing is not able to simulate the void redistribution, or water film effects, that take place after liquefaction, particularly in layered deposits with contrasting permeability (Kokusho 2003). Idriss and Boulanger (2007) proposed two residual strength ratio S_r/σ'_{vc} relationships based on penetration testing resistance of a given soil: (i) a relationship attributing a lower strength for conditions where the void redistribution is likely to occur during earthquake loading; and (ii) another relationship attributing a higher strength for situations where void redistribution is considered likely. It is of relevance to note that, for relatively higher values of penetration resistance, these relationships have been recommended without significant support from back-analysis; as such, caution should be exercised when using these relationships to estimate for cases involving higher density levels.

In spite of this, the evaluation of laboratory data from post-cyclic monotonic loading tests conducted on soil specimens after subjecting them to a pre-defined level of cyclic straining provides important information for understanding soil response in a fundamental manner, as well as for supporting and confirming field-based approaches.

1.6. Overview of the Proposed Research

Existing studies show that the mixing waste rock and tailings has various advantages over the traditional methods involving separate disposal of the two waste streams. Nevertheless, current research on this subject is very limited, requiring more extensive studies on the mechanical behaviour of such mixtures. Furthermore, laboratory testing of such mixtures requires addressing some practical considerations including oversized particles, segregation, membrane puncture, and membrane penetration.

With this background, a research program was undertaken with the major objective of studying the mechanical behaviour of paste rock under monotonic and cyclic shear loading conditions, and this thesis presents the outcome of this work. This document consists of the following scope and thesis organization,

1. Chapter One (current chapter): The concept of uniformly mixing tailings and waste rock to overcome difficulties arising from conventional methods of dumping mine waste is introduced. Previous studies conducted on gap-graded

soils in general and tailings/waste-rock mixtures in particular are presented and reviewed, along with an overview of common approaches used for mixing waste rock and tailings. Based on this review, the need to undertake a laboratory research program to understand the mechanical properties of paste rock is justified.

2. Chapter Two: A new large-scale triaxial apparatus which was capable of real time compensation for membrane penetration effects (to simulate truly undrained conditions within the specimens) was developed to accommodate the monotonic and cyclic triaxial testing of paste rock. Chapter two presents the details regarding the triaxial system and data acquisition system. Furthermore, a new specimen reconstitution method was developed to overcome the specific difficulties associated with the preparation of waste rock/tailings mixtures. The method can be used to reconstitute specimens from highly gap-graded materials in general. The specifics of the method are also presented in this chapter. A version of chapter two has been published in ASTM Geotechnical Testing Journal (Khalili and Wijewickreme, 2008);
3. Chapter Three: Understanding the consolidation properties of paste rock forms one of the important components of this research; therefore, one-dimensional and hydrostatic consolidation properties of paste rock was investigated and the results are discussed in Chapter three;
4. Chapter Four: Study of the monotonic shear response of paste rock plays a key role in understanding the material behaviour from a geotechnical point of view.

Therefore, monotonic shear response of paste rock was investigated and the results are presented in Chapter four;

5. Chapter Five: Understanding of the cyclic response of paste rock is also of great importance in the design of engineered dumps and/or barriers located in seismically active areas. Cyclic shear response of paste rock was therefore investigated and results are presented in Chapter five;
6. Chapter Six: A summary of key findings and conclusions is presented in Chapter six along with limitations regarding the use of the results. Potential future studies to further advance the knowledge and support the development of paste rock technology are also identified.
7. Appendix A: To provide an opportunity for accessing the data obtained as part of this research, results of all cyclic triaxial tests are presented in Appendix A.

This document has been prepared in accordance with UBC formatting principles for manuscript-based theses. As a result, some information such as material description and/or experimental details needed to be repeated in various chapters.

Table 1.1. Methods of mine waste disposal (Adopted from Wickland 2006).

- Homogeneous mixtures – waste rock and tailings are blended to form a homogeneous mass (placement method unknown)
- 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 depression
- Separate disposal – waste rock in dumps, tailings in impoundments

Increasing degree of Mixing

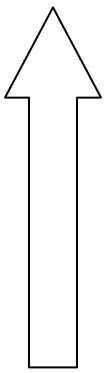


Figure 1 has been removed due to copyright restrictions. The figure removed, contained schematic presentation of four different cases of binary mixtures. a) just filled state b) large particles floating among smaller particles c) meta-stable orientation d) layered orientation of particles.

Figure 1.1. Various scenarios of mixture components (after Thevanayagam 1999).

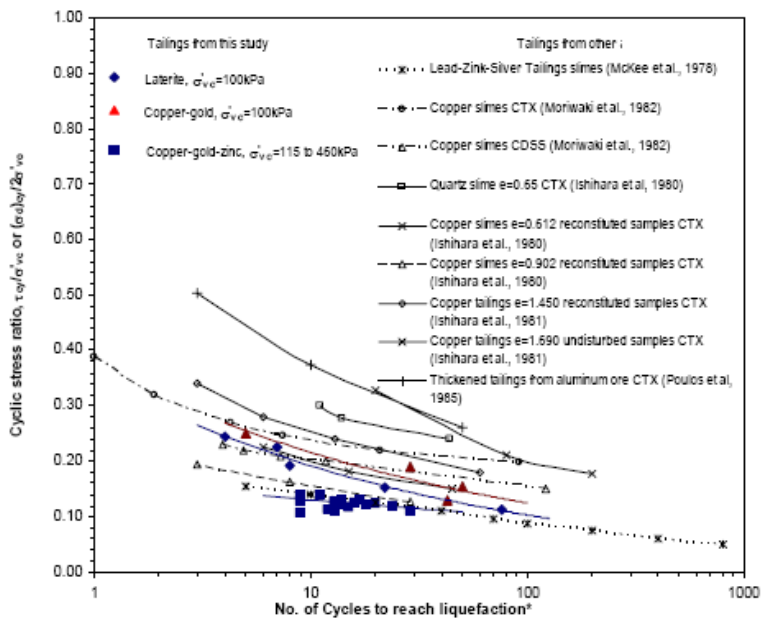


Figure 1.2. Typical cyclic stress ratios for different tailings (after Wijewickreme et al. 2005). © 2008 NRC Canada or its licensors. Reproduced by permission.

1.7. References

Amini, F. and Qi, G.Z. 2000. Liquefaction testing of stratified silty sands. Journal of Geotechnical and Geo-environmental Engineering, **126:** 208-217.

Amini, F. and Sama, K.M. 1999. Behavior of stratified sand-silt-gravel composites under seismic liquefaction conditions. Soil Dynamics and Earthquake Engineering, **18:** 445-455.

Andrus, R.D., Stokoe, K.H., Bay, J.A., and Youd, T.L. 1992. In situ vs of gravelly soils which liquefied. *In* Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, Balkama, pp. 1447-1452.

Andrus, R.D. 1994. In situ characterization of gravelly soils that liquefied in the 1983 borah peak earthquake. Ph.D., The University of Texas at Austin, Texas, USA.

Boulanger, R.W., ASCE, M., Idriss, I.M., and ASCE, F. 2007. Evaluation of cyclic softening in silts and clays. Journal of Geotechnical and Geo-environmental Engineering, **133:** 641.

Brawner, C.O. 1978. Concepts and experience for subsurface disposal of tailings. *In* Proceedings of the second international tailings symposium. Denver, Colorado, USA. pp. 153-173.

Chern, J.C. 1985. Undrained response of saturated sands with emphasis on liquefaction and cyclic mobility. Ph.D., University of British Columbia, Vancouver, Canada.

Chien, L., Oh, Y., and Chang, C. 2002. Effects of fines content on liquefaction strength and dynamic settlement of reclaimed soil. Canadian Geotechnical Journal, **39:** 254-265.

Daniel, C.R., Howie, J.A., and Sy, A. 2003. A method for correlating large penetration test (LPT) to standard penetration test (SPT) blow counts. Canadian Geotechnical Journal, **40:** 66-77.

Eger, P., Lapakko, K., and Chamberlain, P.G. 1984. Mixed disposal of waste rock and tailings to reduce trace metal release from waste rock. *In* Proceedings of the Symposium on Surface Mining, Hydrology, Sedimentology, and Reclamation, Lexington, Ky, USA, 1984. , pp. 2-7.

Evans, M.D. and Zhou, S. 1995. Liquefaction behavior of sand-gravel composites. Journal of Geotechnical Engineering, **121:** 287-298.

Evans, M.D. and Seed, H.B. 1987. Undrained cyclic triaxial testing of gravels - the effect of membrane compliance. UCB/EERC-87/08, University of California at Berkeley, Berkeley, California.

Evans, M.D., Seed, H.B., and Seed, R.B. 1992. Membrane compliance and liquefaction of sluiced gravel specimens. Journal of Geotechnical Engineering, **118:** 856-872.

Fear, C.E. and Robertson, P.K. 1995. Estimating the undrained strength of sand: A theoretical framework. Canadian Geotechnical Journal, **32:** 859-870.

Fines, P., Wilson, G.W., Landreault, D., Langetine, L., and Hulett, L. 2003. Co-mixing of tailings, waste rock and slag to produce barrier cover systems. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, 12-18 July 2003. , pp. 14-17.

Finn, W.D.L. 1998. Seismic safety of embankment dams: Developments in research and practice 1988–1998. *In* Proceedngs of the Specialty Conference on Geotechnical Earthquake Engineering and Soil Dynamics III, Seattle, American Society of Civil Engineers, pp. 812-853.

Fukue, M., Okusa, S., and Nakamura, T. 1986. Consolidation of sand-clay mixtures. *In* Consolidation of Soils: Testing and Evaluation. ASTM, Philadelphia, PA, USA, Fort Lauderdale, FL, USA, pp. 627-641.

Furnas, C.C. 1928. Relations between specific volume voids and size composition. *In* systems of broken solids of mixed sizes. US Bureau of Mines.

Harder, L.F. 1997. Application of the Becker penetration test for evaluating the liquefaction potential of gravelly soils. *In* NCEER workshop on evaluation of liquefaction resistance of soils, Buffalo, NY, 5-9 Jan 1996.

Hatanaka, M. and Uchida, A. 1996. Empirical correlation between penetraion resistance and internal friction of sandy soil. Soils & Foundations, **36:** 1-9.

Holtz, W.G. and Ellis, W. 1961. Triaxial shear characteristics of clayey gravel soils. *In* Proceedings of the fifth international conference on soil mechanics and foundation engineering, Paris, 17-22 July 1961. Dunod, Paris, pp. 143-149.

Idriss, I.M., and Boulanger R.W., 2007. Residual shear strength of liquefied soil. 27th USSD Annual Meeting and Conference, Modernization and Optimization of Existing Dams and Reservoirs, March 5-9, 2007, Philadephia, USA, United States Society on Dams, Denver, Co.

Idriss, I.M., and Boulanger R.W., 2008. Soil liquefaction during earthquakes. EERI Monograph 12, Earthquake Engineering Research Institute, Oakland, California.

Ishihara, K. 1996. Soil behaviour in earthquake geotechnics. Clarendon Press.

Jamiolkowski, M., Kongsukprasert, L., and Lo Presti, D. 2005. Characterization of gravelly geomaterials. *In* Proceedings of The Fifth International Geotechnical Conference, Cairo, Egypt, 2005. Cairo University, Cairo, Egypt, pp. 1-27.

Johnston, G., Sperring, P., and Phelan, K. 1995. Co-disposal of tailings and mine waste. *In* Randol Gold Forum. Perth, Australia. pp. 443-447.

Khalili, A. and Wijewickreme, D. 2008. Method for reconstitution of highly gap-graded specimens for laboratory element testing, ASTM Geotechnical Testing Journal, **31:** 424-432.

Koester, J.P. 1994. The influence of fines type and content on cyclic strength. *In* Ground failures under seismic conditions, Geotechnical Special Publication 44. Atlanta, Georgia. pp. 17-33.

Kokusho, T. 2003. Current state of research on flow failure considering void redistribution in liquefied deposits. Soil Dynamics and Earthquake Engineering, **23:** 585-603.

Kokusho, T., Hara, T., and Hiraoka, R. 2004. Undrained shear strength of granular soils with different particle gradations. Journal of Geotechnical and Geo-environmental Engineering, **130:** 621.

Kokusho, T. 2000. Mechanism for water film generation and lateral flow in liquefied sand layers. Soils and Foundations, **40:** 99-111.

Kuerbis, R., Negussey, D., and Vaid, Y.P. 1988. Effect of gradation and fines content on the undrained response of sand. *In* Proceedings of Hydraulic Fill Structures. Aug 15-

18 1988. Published by ASCE, New York, NY, USA, Fort Collins, CO, USA, pp. 330-345.

Kuerbis, R. and Vaid, Y.P. 1988. Sand sample preparation; the slurry deposition method. Soils and Foundations, **28:** 107-118.

Kulasingam, R., Malvick, E.J., Boulanger, R.W., and Kutter, B.L. 2004. Strength loss and localization at silt interlayers in slopes of liquefied sand. Journal of Geotechnical and Geo-environmental Engineering, **130:** 1192-1202.

Kumar, G.V. and Muir Wood, D. 1997. Mechanical behaviour of mixtures of kaolin and coarse sand. Mechanics of granular and porous materials, 57–68.

Ladd, C.C. and Foott, R. 1974. New design procedure for stability of soft clays. American Society of Civil Engineers, Journal of the Geotechnical Engineering Division, **100:** 763-786.

Lade, P.V., Liggio, C.D., and Yamamuro, J.A. 1998. Effects of non-plastic fines on minimum and maximum void ratios of sand. Geotechnical Testing Journal, **21:** 336–347.

Lade, P.V. and Yamamuro, J.A. 1997. Effects of non-plastic fines on static liquefaction of sands. Canadian Geotechnical Journal, **34:** 918-928.

Lappakko, K.A., Antonson, D.A., and Wagner, J.R. 2000. Mixing of rotary kiln fines with fine grained acid-producing rock. *In* Proceedings of the fifth international conference on acid rock drainage, Denver, CO, 2000. , pp. 901-910.

Lundgren, T. 2001. The dynamics of oxygen transport in soil covered mining waste deposits in Sweden. Journal of geotechnical exploration, **74:** 163-173.

Lupini, J.F., Skinner, A.E., and Vaughan, P.R. 1981. The drained residual strength of cohesive soils. Geotechnique, **31:** 181-213.

Malvick, E.J., ASCE, M., Kutter, B.L., Boulanger, R.W., and Kulasingam, R. 2006. Shear localization due to liquefaction-induced void redistribution in a layered infinite slope. Journal of Geotechnical and Geo-environmental Engineering, **132:** 1293.

Mollins, L.H., Stewart, D.I., and Cousens, T.W. 1996. Predicting the properties of bentonite-sand mixtures. Clay Minerals, **31:** 243-252.

Naeini, S.A. and Baziar, M.H. 2004. Effect of fines content on steady-state strength of mixed and layered samples of a sand. Soil Dynamics and Earthquake Engineering, **24:** 181-187.

Naesgaard, E., Byrne, P.M., Seid-Karbasi, M., and Park, S.S. 2005. Modeling flow liquefaction, its mitigation, and comparison with centrifuge tests. *In* Proceedings of

the ICSMGE TC4 Satellite Conference on Recent Developments in Earthquake Engineering, pp. 95-102.

Ohta, Y. and Goto, N. 1978. Empirical shear wave velocity equations in terms of characteristic soil indexes. Earthquake Engineering and Structural Dynamics, **6:** 167-187.

Olson, S.M. 2001. Liquefaction analysis of level and sloping ground using field case histories and penetration resistance. Ph.D., University of Illinois at Urbana Champaign.

Olson, G.J., Clark, T.R., Mudder, T.I., and Logsdon, M. 2003. A novel approach for control and prevention of acid rock drainage. *In* Proceedings of 6th international conference on acid rock drainage, Ciarns, Australia, 2003. , pp. 789-800.

Olson, S.M. and Stark, T.D. 2002. Liquefied strength ratio from liquefaction flow failure case histories. Canadian Geotechnical Journal, **39:** 629-647.

Pitman, T.D., Robertson, P.K., and Sego, D.C. 1993. Influence of fines on the collapse of loose sands. Canadian Geotechnical Conference, 371-377.

Poulos, S.J., Robinsky, E.I., and Keller, T.O. 1985. Liquefaction resistance of thickened tailings. Journal of Geotechnical Engineering, **111:** 1380-1394.

Robertson, P.K., Wride, C.E., List, B.R., Atukorala, U., Biggar, K.W., Byrne, P.M., Campanella, R.G., Cathro, D.C., Chan, D.H., Czajewski, K., Finn, W.D.L., Gu, W.H., Hammamji, Y., Hofmann, B.A., Howie, J.A., Hughes, J., Imrie, A.S., Konrad, J.-., Kupper, A., Law, T., Lord, E.R.F., Monahan, P.A., Morgenstern, N.R., Phillips, R., Piche, R., Plewes, H.D., Scott, D., Sego, D.C., Sobkowicz, J., Stewart, R.A., Watts, B.D., Woeller, D.J., Youd, T.L., and Zavodni, Z. 2000. The Canadian liquefaction experiment: An overview. Canadian Geotechnical Journal, **37:** 499-504.

Rowe, P.W. 1962. The stress-dilatancy relation for static equilibrium of an assembly of particles in contact. *In* Proceedings of the Royal Society of London. Series A, Mathematical and Physical Sciences JSTOR, pp. 500-527.

Schofield, A.N. and Wroth, P. 1968. Critical state soil mechanics. McGraw-Hill New York.

Seed, H.B. 1987. Design problems in soil liquefaction. Journal of Geotechnical Engineering, **113:** 827-845.

Seed, R.B. and Harder, L.F. 1990. SPT-based analysis of cyclic pore pressure generation and undrained residual strength. Proceedings of the H. Bolton Seed Memorial Symposium, **2:** 351–376.

Singh, S. 1994. Liquefaction characteristics of silts. Journal of Geotechnical and Geological Engineering, **14**: 1-19.

Stark, T.D. and Mesri, G. 1992. Undrained shear strength of liquefied sands for stability analysis. Journal of Geotechnical Engineering, **118:** 1727-1747.

Statham, I. 1974. The relationship of porosity and angle of repose to mixture proportions in assemblages of different sized materials. Sedimentology, **21:** 149-162.

Stewart, D.I., Studds, P.G., and Cousens, T.W. 2003. The factors controlling the engineering properties of bentonite-enhanced sand. Applied Clay Science, **23:** 97-110.

Studds, P.G., Stewart, D.I., and Cousens, T.W. 1998. The effects of salt solutions on the properties of bentonite-sand mixtures. Clay Minerals, **33:** 651-660.

Tan, T.-., Goh, T.-., Karunaratne, G.P., and Lee, S.-. 1994. Shear strength of very soft clay-sand mixtures. Geotechnical Testing Journal, **17:** 27-34.

Tatsuoka, F., Jardine, R.J., Lo Presti, D., Di Benedetto, H., and Kodaka, T. 1997. Characterising the pre-failure deformation properties of geomaterials, theme lecture for the plenary session no. 1. *In* Proceedings of the 14th International Conference on Soil Mechanics and Geotechnical Engineering, Hamburg, 1997. , pp. 2129-2164.

Tatsuoka, F. and Shibuya, S. 1991. Deformation characteristics of rocks and soils from field and laboratory test. *In* Proceedings of the 9th Asian regional conference on SMFE, Bangkok, 1991. , pp. 101-170.

Thevanayagam, S. and Liang, Y. 2001. Shear wave velocity relations for silty and gravelly soils. *In* Proceedings of the 4th international conference on recent advances in geotechnical earthquake engineering and soil dynamics, San Diego, California, 2001. , pp. Paper No: 1.36.

Thevanayagam, S. and Mohan, S. 2000. Intergranular state variables and stress-strain behaviour of silty sands. Geotechnique, **50:** 1-23.

Thevanayagam, S. 1999. Intergrain contacts and shear modulus of non plastic granular mixes. *In* Proceedings of 13th ASCE engineering mechanic division conference, John Hopkins University, 1999. John Hopkins University.

Vaid, Y.P. and Thomas, J. 1995. Liquefaction and post liquefaction behavior of sand. Journal of Geotechnical Engineering - ASCE, **121:** 163-173.

Vaid, Y.P. and Chern, J.C. 1985. Cyclic and monotonic undrained response of saturated sands. *In* Advances in the Art of Testing Soils Under Cyclic Conditions: Proceedings of a session held in conjunction with the ASCE Convention. ASCE, New York, NY, USA, Detroit, MI, pp. 120-147.

Vallejo, L.E. 2001. Interpretation of the limits in shear strength in binary granular mixtures. Canadian Geotechnical Journal, **38:** 1097-1104.

Vick, S.G. 1983. Planning, design, and analysis of tailings dams. Wiley.

Wagener, F., Strydom, K., Craig, H., and Blight, G. 1997. The tailings dam flow failure at merriespruit, south africa, causes and consequences. *In* Proceedings of the Conference on Tailings and Mine Waste '97, Colorado State University, Fort Collins, Colorado, USA, 13-17 January 1997. A.A. Balkema, pp. 13-17.

Wagg, B.T. and Konrad, J.M. 1990. Index properties of clay silt mixtures. *In* Proceedings of the 43rd Canadian Geotechnical Conference. Part 2, Canada, Oct 10-12, 1990. Canadian Geotechnical Soc, Montreal, Can, Chateau, Que, Can, pp. 705-710.

Whitman, R.V. 1985. On liquefaction. *In* Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, pp. 1923-1926.

Wickland, B.E. 2006. Volume change and permeability of mixtures of waste rock and fine tailings. Ph.D., University of British Columbia, Canada.

Wickland, B.E., Wilson, G.W., Wijewickreme, D., and Klein, B. 2006. Design and evaluation of mixtures of mine waste rock and tailings. Canadian Geotechnical Journal, **43:** 928-945.

Wijewickreme, D., Sanin, M.V., and Greenaway, G.R. 2005. Cyclic shear response of fine-grained mine tailings. Canadian Geotechnical Journal, **42:** 1408-1421.

Williams, D.J., Wilson, G.W., and Panidis, C. 2003. Waste rock and tailings mixtures as a possible seal for potentially acid forming waste rock. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, 12-18 July 2003. , pp. 14-17.

Williams, D.J. 1997. Effectiveness of co-disposing coal washery wastes., Fort Collins, Colorado, USA, 13-17 January 1997. A.A. Balkema, pp. 13-17.

Williams, D.J., Gowan, M., and Keefer, P. 1995. Practical co-disposal deposition. *In* Proceedings of the 7th Australian Coal Preparation Conference, Mudgee, New South Wales, 1995. pp. 9-15.

Wilson, G.W., Plewes, H.D., Williams, D., and Robertson, J. 2003. Concepts for co-mixing of tailings and waste rock. *In* Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, 12-18 July 2003. , pp. 14–17.

Wilson, G.W., Willimas, D.J., and Rykaart, E.M. 2003. The integrity of cover systems - an update. *In* Proceedings of 6th international conference on acid rock drainage. Cairns, Queensland, Australia. pp. 445-452.

Yamamuro, J.A. and Lade, P.V. 1998. Steady-state concepts and static liquefaction of silty sands. Journal of Geotechnical and Geo-environmental Engineering, **124:** 868-877.

Yoshimi, Y. and Goto, S. 1996. Liquefaction resistance of silty sand based on in situ frozen samples. International Journal of Rock Mechanics and Mining Sciences and Geomechanics Abstracts, **33:** 357A-357A.

Yoshimi, Y. 1994. Evaluation of in-situ liquefaction resistance of sands based on high-quality undisturbed samples. *In* Proceedings of the Symposium on Developments in Geotechnical Engineering January 12-16 1994. A.A. Balkema, Bangkok, Thailand, pp. 117.

Chapter 2.

Experimental Aspects [i]

Paste rock essentially is a mixture of silt and large size gravel. Therefore, testing of such mixtures in the laboratory presents significant challenges to the researcher. In addition to conventional laboratory procedures required for acquiring high quality data, several complications such as membrane puncture due to sharp particles, and/or segregation and saturation during specimen preparation had to be addressed prior to undertaking this research work. With these considerations in mind, a new triaxial testing system was developed to accommodate the testing of 75 mm diameter specimens with the ability of real-time membrane penetration corrections.

[i] A version of this chapter has been accepted for publication. Khalili, A., and Wijewickreme, D. 2008. New slurry displacement method for reconstitution of highly gap-graded specimens for laboratory element testing. ASTM Geotechnical Testing Journal, **31:** 424-432.

This chapter presents the experimental aspects of this research. Initially, details related to the triaxial testing system with respect to mechanical components, data acquisition, and stress path control system are presented. Difficulties associated with laboratory testing of paste rock and the approaches that were used to overcome those difficulties are documented. A novel specimen preparation method that was developed for reconstitution of highly gap-graded material in general and waste rock/tailings mixtures in particular for element testing is described and the ability of this new method to produce uniform, saturated and repeatable specimens of highly gap-graded mixtures is demonstrated.

2.1. Triaxial Apparatus

A triaxial cell manufactured by GDS Instruments, UK, which is capable of accommodating 75-mm (3”) and 150-mm (6”) diameter specimens, was used for performing the tests. The triaxial chamber is capable of withstanding cell pressures of up to 2000 kPa. The triaxial cell is about 0.6 m in height and 0.4 m in diameter, and a special loading frame had to be designed and fabricated to deliver the anticipated loads. Figure 2.1 shows the triaxial cell with the frame used for this study. A schematic of the overall stress path system is presented in Figure 2.2, and specific details are given in the same figure.

2.1.1 Loading system

All tests were conducted using stress-controlled loading. The cell pressure and back pressure were regulated using two electro pneumatic regulators (EPR). The axial load was applied using a rolling diaphragm (Bellofram) double acting piston with regulated air pressure used to pressurize the piston on both sides. The pressure of the upper chamber was controlled by a separate EPR unit. A schematic diagram of the triaxial system is shown in Figure 2.2. In triaxial compression tests, the pressure in the bottom chamber was set to a small nominal amount (approximately 50 kPa), and the pressure in the top chamber was increased steadily to meet the loading requirement. For triaxial extension and cyclic loading tests, a relatively high air pressure was maintained on both top and bottom chambers and the pressure on the top chamber was varied as required using the EPR unit. All three EPR units were driven with the use of two National Instruments (Austin, Texas, USA) DAQPad 6015 devices using an analog feed. This essentially allowed simulating any desired triaxial stress path on a soil specimen.

2.1.2 Pore water injection system

Membrane compliance corrections to replicate “truly” undrained conditions during undrained triaxial testing were performed using three pore water injection pistons each having a diameter of 25 mm. Each piston was operated by means of a dedicated stepper motor and was controlled by triggering digital pulses through a National Instruments

(Austin, Texas, USA) DAQPad 6015 data acquisition and control device attached to a computer.

2.1.3 Data acquisition system

The following five electronic data acquisition units (i.e. transducers) were used along with a GDS Instruments (Hampshire, England) 8-channel, 32-bit data acquisition system to measure and record specific data from specimens during triaxial tests (also see Figure 2.2):

1- External load cell for measuring the vertical stress on specimens;
2- Cell pressure transducer for measuring cell pressure;
3- Pore water pressure transducer for measuring pore-water or back pressure;
4- Differential pressure transducer (DPT) for measuring volume changes of specimens;
5- External axial LVDT for measuring axial strains.

2.1.4 Measurement resolution and stability of transducers

The cell pressure and pore water pressure transducers used in the triaxial apparatus were capable of measuring pressures with a resolution of 0.25 kPa. The resolution in the vertical stress, derived from vertical load cell measurement, was 1.25 kPa. The volume changes of the specimens during consolidation were measured using pipettes coupled to

the differential pressure transducer (DPT), and the DPT was able to detect volumetric strains in the order of 2.4 x 10^{-4}. The vertical displacements were measured using a linear variable displacement transducer (LVDT) that allowed detection of axial strains in the order of 3.4 x 10^{-4}.

Stability of the electronic devices was monitored for an extended period of time and the results are presented in Figures 2.3 through 2.7. External load cell, cell pressure and pore pressure transducers and external LVDT all show good stability without any significant drift during the monitoring time. The differential pressure transducer shows 0.15 cm^3 of drift in 75 hours which translates to 0.0205% of volumetric strain in 75 hours or 0.00128% of volumetric strain in 4.7 hours (approximate duration of a triaxial test); therefore it was judged that the errors due to drift can be considered negligible.

2.1.5 Software

Lab View Version 6.0 (National Instruments, Austin, Texas, U.S.A.), was used to compose software for data acquisition and feedback control of stress path triaxial device. Figure 2.8 shows the schematic diagram of the algorithm used for data acquisition and stress path control. Based on the real time data collected from all transducers, comparisons were made between the actual and target stresses; pressures were then adjusted using the load-controlling EPRs to achieve the desired target stresses within the specimen. Real-time corrections were also made in order to compensate the membrane

compliance by injecting or withdrawing water from the specimen using the aforementioned injection system.

2.1.6 Difficulties associated with preparation of specimens

Because of the highly gap-graded nature of the material tested, segregation and membrane puncture were two controlling issues which prevented the use of commonly used specimen preparation techniques. Therefore, a new technique had to be developed for the reconstitution of saturated soil specimens for laboratory testing. The details related to this technique and information related to test materials, which has already been published as a technical paper (Khalili and Wijewickreme 2008), is presented in Section 2.2.

In addition to the new technique described herein, significant effort was also expended in selecting and developing appropriate membrane configuration for encapsulating the test specimens. Because of the high angularity of the coarser fraction of the paste rock, membrane puncture during specimen preparation and testing became a major concern, and the development of procedures to overcome this difficulty involved many time-consuming trial and error approaches. In addition to the lack of previous published work, the available experience from industry laboratories who have attempted testing of paste rock also suggested major difficulties associated with membrane puncture.

With these considerations in mind, a range of trial and error options as noted below were attempted to identify a suitable membrane configuration for enclosing paste rock specimens:

1- Using 0.3-mm thick membrane;
2- Using 0.6-mm thick membrane;
3- Using 0.6-mm thick membrane over a 0.3-mm membrane;
4- Using two 0.3-mm thick membranes (with no lubricant between membranes);
5- Using two 0.3-mm thick membranes with talcum powder coating between the membranes; and
6- Using two 0.3-mm thick membranes with a thin layer of silicon grease between the membranes.

It was found that the use of single-membrane configurations (i.e. Options 1 and 2 above), even with a membrane thicknesses of 0.6 mm (i.e. Option 2), was often subjected to puncture. The Option 3 also proved to be inefficient since manipulation and stretching of the 0.6 mm membrane over the thinner membrane for reducing the wrinkles resulted in frequent puncture of membrane. The Options 4 and 5 (i.e., without lubricant or with talcum powder between membranes, respectively), again, required considerable manipulation and stretching of the outer membrane which, in turn, increased the risk of membrane puncturing. The use of two 0.3-mm thick membranes, with application of a thin layer of silicon grease after placement of the first 0.3-mm thick membrane (i.e.,

Option 6 above), provided the most effective configuration for enclosing the specimen with least opportunity for membrane puncture; as such, this approach was adopted for the production testing.

It is of relevance to note that, even with the selected optimum Option 6 above, the success rate of reconstituting paste rock specimens (and especially rock-only specimens) without puncture was still in the order of 35 to 45% (i.e., a total of 173 triaxial test specimen preparation attempts were required in order to yield the seventy (70) successful triaxial tests presented in this thesis).

2.1.7 Validation of triaxial apparatus and loading system

Verification of the mechanical performance of the triaxial apparatus and loading system was an important consideration in generating high quality experimental data. To achieve this objective, two conventional consolidated drained (CD) triaxial tests (i.e. compression and extension) were performed on relatively loose specimens of Ottawa sand reconstituted using the method of water pluviation for comparison with previously published information. This approach was judged reasonable since high quality shear testing data for Ottawa sand are available from previous research at UBC (using other triaxial devices) for comparison. The specimens for both CD tests were hydrostatically consolidated to an effective confining stress of about 300 kPa prior shearing. Table 2.1 presents the parameters pertaining to these two tests. The observed behaviour of Ottawa

sand during compressive and extension loading are presented in Figure 2.9. The mobilized friction angles are about 30 degrees (Figure 2.9c) in both shearing modes. The observed behaviour and the mobilized friction angles are in accord with those reported by Vaid and Chern (1985), Wijewickreme (1986), and Ayoubian and Robertson (1998) as may be noted from Table 2.2. This observed agreement with previously published behaviour confirmed satisfactory mechanical performance of the new triaxial shear device and its suitability for testing paste rock for the present research work.

2.2. Selection of the Waste Rock to Tailings Mixture Ratio for the Present Study of Paste Rock

Since the proposed research work involved examining the effect of a significant number of load and drainage variables (e.g., confining stress level, drained versus undrained, monotonic compression versus extension shear loading, cyclic loading with and without shear stress bias, cyclic loading with and without shear stress reversal, etc.) on the mechanical response of paste rock, it was judged preferable to undertake the study using paste rock prepared at a single selected waste rock to tailings mixture ratio.

Wickland (2006), using the same waste rock and tailings material considered for the present study, performed a comprehensive study of waste rock and tailings design mixtures and concluded that a mixture ratio of 4.8:1 (waste rock to tailings by weight) provides a state at which tailings particles “just fill” the void spaces between the rock

particle matrix. He further confirmed that such mixture ratio provides the maximum density for the mixed material.

Since the work by Wickland (2006) and this study are two phases of an overall research program on the same paste rock material, it was considered prudent to use paste rock material reconstituted at a waste rock to tailings (by weight) mixture ratio of 4.8:1 representing the "just filled" state for the present investigation. Further information related to the waste rock and tailings used in this study, and paste rock developed after mixing is presented in Section 2.3.3 (also see Figure 2.10 for the grain size distributions).

2.3. New Slurry Displacement Method for Reconstitution of Highly Gap-Graded Material

Laboratory element testing has a major role to play in advancing our understanding of the mechanical response of geomaterials. Undisturbed samples obtained from the field are considered most suitable for studying the behaviour of natural soils because of the need to have test specimens that closely represent the in situ soil particle fabric, stress history, ageing, etc. The use of reconstituted soil specimens for laboratory element testing, on the other hand, has gained wide acceptance due to a number of reasons, including: (i) the difficulties in obtaining high-quality undisturbed field samples; (ii) the need to test essentially "identical" homogeneous specimens (without variability commonly found in field samples) under differing loading states/conditions for fundamental soil property

characterization work; and (iii) the need to characterize man-made engineering materials. The content of the following sections stems from the need with respect to item (iii) above with specific relevance to the development of new technology and material science for the mining industry.

The commonly available specimen reconstitution methods (Kuerbis and Vaid 1988) do not provide a suitable way to prepare specimens of highly gap-graded geomaterials such as waste rock and tailings mixtures in a saturated condition. With the recognition of this need, a new technique was developed for reconstitution of paste rock in particular, and it is considered applicable for any heavily gap-graded soil in general. The following sections present the details of the method and demonstrate its suitability for preparation of uniform, saturated and repeatable specimens of gap-graded soil. For the reasons mentioned in Section 2.2., a mixture ratio of 4.8:1 (rock to tailings) was used for preparation of specimens. Basis for selection of such mixture ratio is explained in details in section 2.2 and chapter 1.

2.3.1 Review of commonly used specimen reconstitution techniques

Homogeneity of soil particle fabric and density (void ratio or porosity), and level of saturation are critical considerations in the preparation of soil specimens for laboratory element testing. Furthermore, it is important to have a procedure that can be easily repeated in order to obtain essentially identical specimens.

Several commonly used reconstitution techniques for preparing soil specimens are presented in Table 2.3. The brief discussion below provides the necessary background for the work presented herein. All of these current methods have been developed with the objective of simulating the particle structure (fabric) and saturation conditions found in naturally-deposited or man-made soil masses.

One of the most commonly used techniques for preparation of reconstituted sand specimens is water pluviation. This technique essentially involves raining saturated sand from a flask into a specimen mould filled with water. The deposition process takes place completely in a water medium under gravity. The drop height in this technique has no significant effect on the as-deposited density of the specimen, because sand particles usually reach the terminal velocity over a relatively short distance, thereby causing the velocity at the time of deposition to be almost the same for all the particles (Vaid and Negussey 1986). For example, sand particles with a mean diameter (D_{50}) of 0.4 mm were shown to reach the terminal velocity within merely 2 mm of displacement (Vaid and Negussey 1986). As a result, sand specimens prepared using this technique have been found to be relatively uniform and in a generally loose state. If required, denser samples are typically obtained by tamping or vibrating the specimen mould after or during water pluviation.

In the method of air pluviation, sand particles are poured into the specimen mould in air from a drop height which is typically maintained constant during deposition. The drop

height used in this technique can be adjusted to yield different relative densities (Vaid and Negussey 1986; Wijewickreme et al. 2005). It has been shown that the relative density of the specimen is dependent on the total kinetic energy of the particles which is a function of their velocity at the time of impact with the surface of the specimen and also on the particle flow rates. The higher mass rates at a given height of pouring are shown to lead to relatively lower densities.

The nature of anisotropy and soil particle fabric obtained by both water and air pluviation have been judged to duplicate those resulting from the natural deposition process observed in alluvial environments (Oda 1972; Vaid and Negussey 1986). Water pluviation is used more often than air pluviation, because: (i) there is no need to control the particle drop height; and (ii) the method automatically provides initially saturated specimens. However, neither of these pluviation methods is suitable for soils that are not uniformly graded (e.g., silty sands, sandy silts, gap-graded soils in general), since there is ample opportunity for segregation of particles during the deposition process leading to significantly non-uniform and layered soil specimens (Vaid and Neguessey 1986).

The method of moist-tamping consists of placing moist soil in layers in the specimen mould, and tamping it to achieve a desired relative density. Tamping is commonly performed with a specified force and frequency of tamping before the next layer is placed. Relative densities of each layer can be controlled by placing a specific mass of particles in each layer, and trying to reach a certain height for that layer during

compaction. In the process of tamping, not only the current top layer is being compacted but also all the layers that underlie the top layer as well. Therefore, in this method, the lower layers are typically placed "under-compacted", so that some of the compaction energy transmitted during the tamping of the upper layers would have the opportunity to bring the density of lower layers to the target density at the end of the compaction process (Ladd 1978). Soil specimens prepared using moist tamping have been suggested to possess a "honey comb" and "meta stable" particle structure with much larger void ratios in comparison to those arising from pluviation methods (Casagrande 1979, Vaid and Sivathayalan 1998). Vaid et al. (1999) have demonstrated that the moist tamping method leads to specimens having non-uniform density. The other disadvantage with this method is the need to saturate the specimen after compaction, and the technique does not mimic the natural deposition processes. The method is considered to simulate the soil particle fabric in rolled construction fills. There is not much opportunity for segregation when the moist-tamping method is used, even when the material is not uniformly graded.

In order to avoid segregation of particles during specimen preparation, Kuerbis and Vaid (1988) developed a slurry deposition method for reconstitution of silt and sand mixtures. In this process, a slurry comprising silt and sand mixed to a desired ratio is vigorously shaken in a capped cylindrical tube. Immediately after shaking, the tube is quickly transferred inside a specimen mould, and then it is withdrawn from the mould leaving behind the mixed slurry in the cavity of the mould. This method has shown to be

effective in preparing homogeneous, uniform, and repeatable silt/sand mixtures without segregation.

The method of slurry consolidation is used for reconstituting fine-grained soils (i.e., clays and relatively high plastic silts). Herein, larger block samples are prepared by consolidation of the material in slurry form, and then specimens for testing are obtained by trimming cut-portions from the block (Sheeran and Krizek, 1971).

The suitability of the above techniques for the preparation of specimens of heavily gap-graded paste rock material considered in this study is examined in Table 2.4. As may be noted, all of the current techniques described above are unsuitable for preparing specimens of paste rock. Although the method of moist-tamping may produce a specimen with uniform density, saturation of a "moist" gap-graded specimen prepared using this method would still be extremely difficult, if not impractical, due to the presence of fine particles. To confirm the validity of such a statement, two specimens were formed using the moist tamping method, and B-Values were measured in an undrained state as an indication of saturation. B-Values of 62% and 57% were obtained which represent poor saturation within specimens as a result of air entrapment during the moist tamping procedure.

2.3.2 New slurry displacement method

To overcome the limitations of the aforementioned specimen reconstitution techniques, a new "slurry displacement" method was developed for preparing saturated, uniform, and repeatable specimens of highly gap-graded paste rock. The suitability of this approach for preparing specimens of paste rock mixture is demonstrated below.

2.3.3 Materials used

The raw material used for this study includes blasted waste rock and Carbon in Pulp (CIP) tailings from the Porgera Gold Mine, Enga Province, Papua New Guinea. The waste rock has an altered sedimentary origin, and, as expected from a mining process, the particles are very angular in nature, with sharp edges. Waste rock particles were scalped after dry-sieving through a Tyler Standard sieve with 9.423 mm openings (as per ASTM Standard D422). The CIP tailings were treated with calcium hypochlorite to destroy cyanide and yield a chemically inert material for the purpose of this study. The CIP tailings material consisted of primarily silt and clay size particles. By weight, more than 90% of the particles were less than 75 μm in size. The particle size distributions of waste rock and tailings are presented in Figure 2.10.

2.3.4 Method of specimen preparation

2.2.4.1 Preparation of slurry medium and mixture

De-aired water was added to CIP tailings with the objective of preparing a tailings slurry medium having a water content of about 85% ± 2% (i.e. solid content of 54% ± 0.5%). The tailings and de-aired water were mixed vigorously using a hand-held mixer to obtain an essentially homogenous slurry medium; care was taken to avoid splashing and entrapment of air bubbles during this mixing process. Slurry was prepared in this manner, so that there was sufficient quantity for mixing with waste rock, and also for filling a 75-mm diameter specimen, as described in the following sections.

Measured quantities of waste rock and tailings slurry medium prepared as per above were mixed manually using a bent tablespoon for the preparation of paste rock for testing (Note: the ratio of waste rock to tailings was kept at 4.8:1 by weight in order to meet the optimum mix ratio for "just filling" the void space in waste rock). The quantities were determined so that there is a sufficient volume (approx. 1100 ml) of uniformly blended mixture for reconstitution of a 75-mm diameter, ~150-mm high cylindrical specimen for triaxial testing. The mixing was conducted gently with rock material introduced in small quantities so as to minimize entrapment of air in the mix during the mixing process. Some pictures taken during the preparation of the slurry medium and pre-mixed paste rock are presented in Figure 2.11.

2.2.4.2 Preparation of specimens

As the first step, the cylindrical split-mould for preparing a 75-mm specimen is initially set up over the triaxial base platen (containing a saturated well-placed porous stone), with a 0.3-mm thick rubber membrane stretched over the inside cavity of the mould (Figure 2.12a). The porous stone permeability was considered to be much more than the material tested in this study. The membrane is stretched to form a cylindrical cavity using a vacuum applied to the mould (since this is a commonly used approach, additional details are not provided herein). The next step is to place the already pre-mixed paste rock in the mould; one of the key concerns during this step is the opportunity for entrapment of air during placement. It was recognized that, although placement of the pre-mixed paste rock into a mould filled with de-aired water would minimize entrapment of air, it would not be suitable since the fines in the mixture will get "washed away" by the water during the deposition process.

A new alternate approach was developed, wherein instead of water, the cavity of the mould is filled with the same slurry medium that was used for the preparation of the paste rock. Once the cavity is filled to a height of about 50 mm with the slurry medium (Figure 2.12b), the pre-mixed paste rock, is gently placed in the cavity (using a bent tablespoon), in layers, through this slurry medium (see Figures 2.12c and 2.13a). In this process, the slurry medium gets displaced upwards as the paste rock (i.e., mixture of waste rock and tailings medium) gets deposited in the cavity. The bent tablespoon

facilitates the process of placement while avoiding segregation. Since the water content of the slurry medium in the cavity is identical to that of the tailings used for preparation of the pre-mixed paste rock, any given pore space within the rock particle matrix of the as-deposited specimen will end up containing tailings material that has the same water content (regardless of whether the tailings in a given pore space originated from the pre-mixed paste rock or from the slurry medium in the cavity). Moreover, since the pre-mixed paste rock is directly deposited through the saturated slurry medium, this approach minimizes the possibility of entrapment of air.

The preparation of a given specimen with approximately ~150-mm height, required placement of paste rock in about 7 layers, each layer essentially comprising about two tablespoonfuls of the mix. Once placed, each layer was given about 25 to 30 strokes of gentle tamping using a 12.6 mm diameter aluminum rod (Figures 2.12d and, 2.13b); the intent of gentle tamping herein was not to densify the coarser skeleton but to approximately level the placed material.

Once the paste rock had been placed to the approximate desired specimen height, the excess tailings slurry present at the top was gently removed using a low pressure vacuum system without causing any disturbance to the specimen below. The top of the specimen was then carefully leveled, and the top cap of the specimen was then placed. The rubber membrane was unrolled from the mould, placed around the top cap, and then sealed and secured using an o-ring. At this stage, about 25 kPa vacuum was applied to the pore

water drainage line; in effect, this caused application of a relatively low (~25 kPa) effective confining pressure to the specimen thus allowing the split-mould to be dismantled, second membrane to be placed over the specimen and the triaxial cell assembled while keeping the specimen intact.

After assembling and filling the triaxial cell chamber with water, the next step was to determine the Skempton pore pressure parameter B (B-Value) to check the level of saturation. In the B-value determination phase, the cell pressure was applied in stages while keeping the specimen undrained and monitoring the transducers that are dedicated to measure cell water pressure and pore water pressure (Note: the B-value phase is commenced with an initial cell pressure that is essentially at zero, and a negative pore water pressure of about -25 kPa which was arising from the application of vacuum as mentioned earlier). The incremental increase in pore water pressure (Δu) arising due to a given increase in applied cell pressure ($\Delta\sigma_3$), allowed determination of the B-value. Upon confirming that the level of saturation is acceptable (see the sub-section 2.2.6.1 titled "Saturation" under the section on "Effectiveness of the Slurry Displacement method"), the specimens were left overnight under undrained conditions, for commencement of the consolidation phase on the following day. In the consolidation phase, the specimens were hydrostatically consolidated to the required stress level against a back pressure of more than 100 kPa, thus in preparation for monotonic and/or cyclic shear testing, as required.

One difficulty encountered during specimen preparation was the frequent puncturing of the specimen membrane due to the sharp edges of the angular coarser fraction (i.e., crushed waste rock). As explained earlier in section 2.1.6, this was overcome by applying silicone grease on the outer surface of the membrane and then placement of a second 0.3-mm thick membrane to encapsulate the already prepared specimen.

2.2.5 Control of waste rock to tailings mix ratio

The waste rock to tailings mix ratio can be controlled effectively by changing the water content of the slurry medium during preparation of gap-graded test specimens. The variation of this mix ratio with different water contents of the slurry medium is presented in Figure 2.14, and it can be noted that rock to tailings ratio increases with increasing water content of the tailings. In spite of some observable scatter, there is a reasonably good correlation between the specimen mix ratio and the initial water content of the slurry medium; this suggests that a desired specific mix ratio can be achieved by selection of the initial water content of the slurry medium, and a tailings water content of around 84% is needed to achieve a desired rock to tailings ratio of 4.8:1 for the testing herein.

The final mixture ratio of the specimen is also dependent on the amount of energy being used during tamping when preparing the specimen. Tamping makes the coarser skeleton (i.e. crushed waste rock component) denser, and increases the rock to tailings ratio as a result. Controlling the energy when tamping being performed manually is more difficult

than regulating the water content of the slurry. Therefore, in this work, tamping was not used as a means to densify the coarser fraction; rather it was only used to level the material in the mould.

2.2.6 Effectiveness of the slurry displacement method

A series of laboratory tests were conducted to assess the effectiveness of the slurry displacement method for reconstituting saturated, uniform, and repeatable specimens of heavily gap-graded soils. The details related to these tests and the results are presented below.

2.2.6.1 Saturation

The Skempton B-values determined during testing of over 30 specimens of paste rock prepared using the new slurry displacement method, are presented in the histogram in Figure 2.15. Out of 34 specimens tested, 29 specimens (~85% of total specimens) had a B-value of over 0.95 and 14 (~41% of total specimens) specimens had a B-value over 0.98, at which the pore water pressures in the specimens had reached a value of about 100 kPa. The high B-values yielded suggest that the slurry deposition method is capable of producing triaxial specimens with very good level of saturation, without any need for additional complex saturation methods such as flushing with water or carbon dioxide (Baldi et al. 1988) after specimen preparation.

2.2.6.2 Specimen homogeneity

The uniformity of sand specimens is often assessed using the density of dissected sections obtained from gel-impregnated samples (Emery et al. 1973; Vaid and Negussey 1986; Vaid et al. 1999). Since this approach is not applicable to paste rock mix with fine-grained soils, a simple, but accurate technique was developed to "dissect" specimens and determine the uniformity of the paste rock specimens (i.e., homogeneity) prepared using the slurry displacement method.

After placement of paste rock to the desired height and placement of the top cap, the height of the specimen was established by measuring the elevation of the top cap. The top cap was then removed, and the specimen was "dissected" into four sections as per below. Initially, about a 2-cm portion of the specimen material was removed from the top of the specimen using a spoon. Weight of the removed material was measured to the nearest 0.01 g. In order to obtain the "as-prepared" density of the material that was in this upper zone, it was required to measure the volume of the cavity made by the soil removal. The volume of cavity was filled with vegetable oil. The vegetable oil was released from a container with a known volume/mass of oil; as such, the volume/mass of the vegetable oil used to fill the cavity could be obtained to the nearest 1 cm^3 from the difference between the initial and final volume/mass of the oil in the container. The choice of vegetable oil to fill the cavity was for two reasons: (i) with a density lighter than water, oil would not penetrate into the wet soil mass below; (ii) oil would not

dissolve in water, as such it could be removed with relative ease. Knowing the mass and the volume, it was possible to calculate the density of the paste rock that was within the upper zone removed from the sample.

At this stage, the vegetable oil placed in the mould was removed using a vacuum, and the paste rock within the next 2.5-cm of the specimen material was removed using the spoon, and the same procedure (i.e. filling with oil and measurement of volume) was repeated. This approach was repeated so that the density values of the paste rock within four layers of the specimen were determined. The whole procedure above was repeated for another specimen prepared using the same method.

The variation of bulk density along the specimen height for two paste rock specimens prepared using the slurry deposition method are given in Figures 2.16(a) and 2.16(b), respectively. As may be noted from the two figures, in both the specimens, the observed density of the bottom layers is slightly higher than the density of the upper layers. The results indicate that the slurry displacement technique allows the preparation of triaxial specimens that are relatively uniform, with densities of a given ~35-mm thick zone deviating not more than $\pm$3.7% from the corresponding average density (Note: the lines representing ±5% variation in mean density are shown to assist the comparison). Uniformity checks, using gel-impregnation techniques, conducted by Vaid and Negussey (1986) have shown that sand specimens can be prepared using water-pluviation to achieve relative densities within $\pm$3% from the average relative density. On the other

hand, sand specimens prepared using moist-tamping seem to have given rise to wide non-uniformities with up to ±10% deviations from the average (Vaid et al. 1999). Based on this, the uniformity of density in specimens prepared using the new slurry displacement technique was judged acceptable from the point of view of laboratory element testing.

Close proximity of the mean densities (i.e., 2.18 g/cm^3 and 2.21 g/cm^3) observed for the two specimens shown in Figures 2.16(a) and 2.16(b), respectively, also demonstrates the ability of the new technique to replicate specimens of the same density.

In addition to the determination of density, gradation analysis was carried out on material removed from four quarter-zones of a specimen prepared using the new slurry displacement method, and the gradations obtained are plotted in Figure 2.17. Essentially identical gradations for the material obtained from the four zones confirms the effectiveness of the method in preparing uniform specimens. The variation of ratio of the coarse to fine fraction (i.e. ratio of mass of rock to mass of tailings component) derived from these grain size testing for the same four zones are presented in Figure 2.18. The average coarse to fine fraction ratio registered for the aforementioned specimen was 4.74:1 which is close to the desired 4.8:1 ratio. It can also be noted that the maximum deviation of the ratio of coarse to fine fraction of a layer is within ±2.6% from the mean value, further confirming the suitability of the method (the lines representing ±5% deviation from the average coarse to fine fraction presented in the same figure to assist the comparison).

2.2.6.3 Repeatability of specimens

Another key requirement of a good specimen reconstitution technique is the ability of the method to replicate test specimens. In order to investigate this, four specimens prepared using the slurry displacement method in an essentially identical manner were tested under: (i) undrained monotonic compression shear (2 specimens); and (ii) undrained cyclic shear (2 specimens). All specimens were initially consolidated hydrostatically to an effective stress of 200 kPa. Since the tests were conducted in a computer-controlled stress path triaxial device, it was possible to correct for the volume changes due to membrane compliance and thereby maintaining the specimen in a "true" constant volume condition (see section 2.1.2). More information regarding the membrane compliance correction is detailed in chapter 4. Some key information pertaining to the five test specimens are presented in Table 2.5.

The stress-strain, pore water pressure and stress path response obtained from identical monotonic triaxial compression tests on Specimens No. 1 and 2 (detailed in Table 2.5) are compared in Figure 2.19. As may be observed, test results from the two specimens are in good agreement, which, in turn, supports the ability of the slurry displacement method to replicate specimens. The response from identical cyclic triaxial tests performed on Specimens No. 3 and 4 are presented in Figure 2.20 (only the results for cycle numbers 2, 5 and 9 are presented for visual clarity). As may be noted, the test results from cyclic loading conducted on similar specimens are in good agreement, again

confirming the effectiveness of the slurry displacement method. The slight difference between the cyclic shear test results of specimens 3 and 4 could be attributed to minor variation of their initial consolidation pressures and void ratios.

2.2.7 Summary

With increased emphasis on sustainable mine waste management practices, disposal of mine waste rock and tailings in the form of a mixture (called paste rock) is one of the considerations receiving wide attention. Current understanding of the fundamental mechanical response of paste rock is very limited, and there is a need to undertake laboratory element testing of reconstituted specimens of this highly gap-graded material.

It is shown that the commonly available specimen reconstitution methods are not suitable for preparing uniform/homogeneous specimens of waste rock and tailings mixtures in a saturated condition. In recognition of this need, a new technique was developed for reconstitution of paste rock, which is also generally applicable for most heavily gap-graded soils. The new approach essentially involves preparation of the paste rock by mixing of waste rock and slurry medium made of tailings in predetermined proportions, and then placing the mixture into a specimen-mould filled with the same slurry medium used for preparing paste rock. Since a slurry medium is displaced by the paste rock as the specimen is being constructed, the technique is named “slurry displacement” method.

The method is capable of forming specimens with very good uniformity and degree of saturation. The mix ratio between coarse and finer fractions of soil, and density of the specimens, can be controlled by changing the water content of the slurry. It was also shown that the method has the ability to replicate near identical specimens which is an essential characteristic for a specimen reconstitution technique to be acceptable for laboratory element testing.

Table 2.1. Parameters pertaining to the two tests conducted on Ottawa Sand.

SPECIMEN	OTCTC-300	OTCTE-300
CELL PRESSURE, KPA	465	468
BACK PRESSURE, KPA	165	168
EFFECTIVE STRESS, KPA	300	300
VOID RATIO [A],	0.720	0.723
RELATIVE DENSITY (DR) [B], %	31.3	30.3

[a]: Void ratio after consolidation
[b]: e_{max} and e_{min} of Ottawa sand were assumed to be 0.82 and 0.50 respectively (Vaid and Chern 1985)

Table 2.2. Reported friction angles of Ottawa sand in extension and compression shearing

PUBLICATION	SHEARING MODE	FRICTION ANGLE, DEG
VAID AND CHERN (1985)	COMPRESSION	30
WIJEWICKREME (1986)	COMPRESSION	30
AYOUBIAN AND ROBERTSON (1998)	EXTENSION	30

Table 2.3. Commonly used soil specimen reconstitution techniques.

Method	Typical Soil Type	Brief Description of Specimen preparation procedure	References
Air pluviation	Cohesionless soils (sands)	Gravity deposition of particles in air	Vaid and Negussey. (1986) Emery et al. (1973) Wijewickreme et al. (2005)
Water pluviation	Cohesionless soils (sands)	Gravity deposition of particles in water	Vaid and Negussey (1986), Vaid and Sivathayalan (1998) Miura and Toki (1982)
Moist-tamping	Cohesionless soils (sands), silty sands, silts, clays, silt/clay/sand mixtures	Placement of moist soil in layers with tamping after placement of each layer	Castro (1969), Casagrande (1979)
Deposition as a slurry mixture	Silty sand/sandy silt	Placement of silt-sand mixture prepared by vigorous shaking (prior to placement)	Kuerbis and Vaid (1988)
Slurry consolidation	Clays and relatively high plastic silts	Prepare larger block samples by consolidation of slurry and then obtain specimens by trimming from the block.	Sheeran and Krizek (1971)

Table 2.4. Suitability of commonly used reconstitution techniques for preparation of heavily gap-graded paste rock.

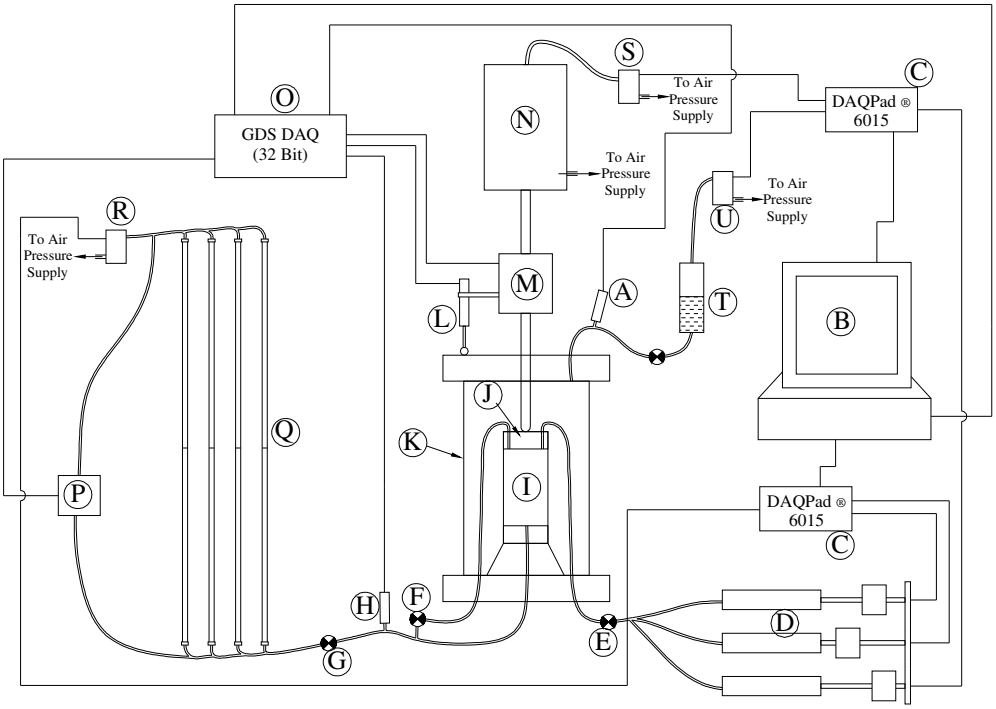
Method	Suitability for preparation of paste rock specimens
Water Pluviation	Not considered suitable since the water-pluviated gap-graded paste rock material will be segregated leading to highly non-uniform specimens.
Air Pluviation	Not considered suitable since the air-pluviated gap-graded paste rock material will be segregated leading to highly non-uniform specimens. Moreover, saturation of air-pluviated specimens from a dry state will be impractical.
Moist Tamping	To achieve a suitable level of compaction, the paste rock material should be in an optimum moisture condition which is typically unsaturated. Due to the presence of a high fine-grained fraction, saturation of such specimens will be extremely difficult (impractical).
Slurry Deposition	With the presence of relatively coarse particles in paste rock, it would not be practical to use a "shaking process" to make a homogeneous slurry that could be "poured" into a mould without segregation.
Consolidation of mixture to form a block sample	This method is not suitable since smaller specimens cannot be trimmed from a consolidated block due to the presence of relatively coarse particles in paste rock.

Table 2.5. Some key parameters related to the four test specimens used in triaxial shear testing.

SPECIMEN	1	2	3	4
TOTAL DENSITY, G/CM^3	2.18	2.19	2.20	2.16
ROCK TO TAILINGS RATIO	4.81	4.91	4.77	4.89
TOTAL VOID RATIO[A]	0.445	0.441	0.432	0.472
TAILINGS COMPONENT VOID RATIO[A]	2.771	2.794	2.668	2.977
ROCK COMPONENT VOID RATIO[A]	0.722	0.712	0.709	0.749

[a]:After consolidation

Figure 2.1. Picture showing the triaxial system used in this research.



A: Cell pressure transducer
B: Computer
C: NI DAQPad 6015 controllers
D: Injection system
E: Injection valve
F: Top drainage valve
G: Master drainage valve
H: Pore pressure transducer
I: Specimen
J: Top cap
K: Triaxial cell
L: LVDT
M: External load cell
N: Double acting loading piston
O: GDS data acquisition unit
P: Differential pore pressure transducer
Q: Parallel pipettes
R: EPR controlling back pressure
S: EPR controlling axial load
T: Water reservoir
U: EPR controlling cell pressure

Figure 2.2. Schematic diagram of the triaxial setup.

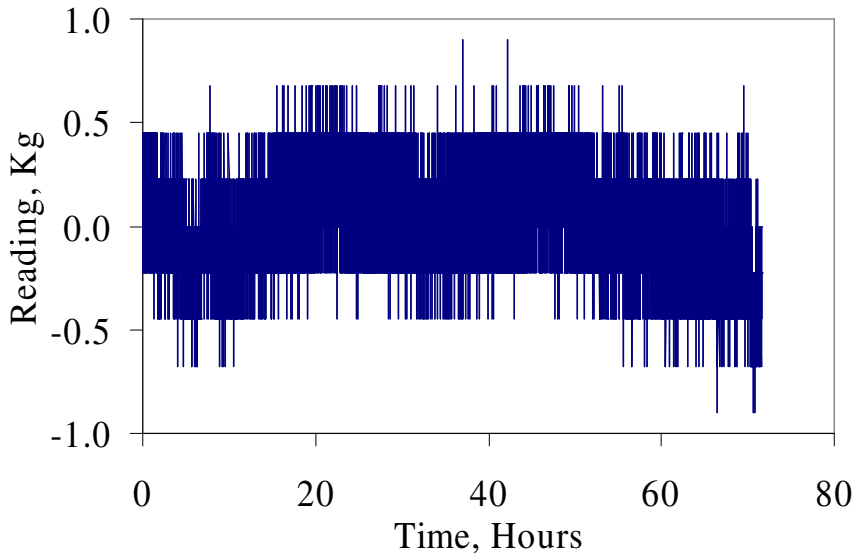


Figure 2.3. Stability of the external load cell with time.

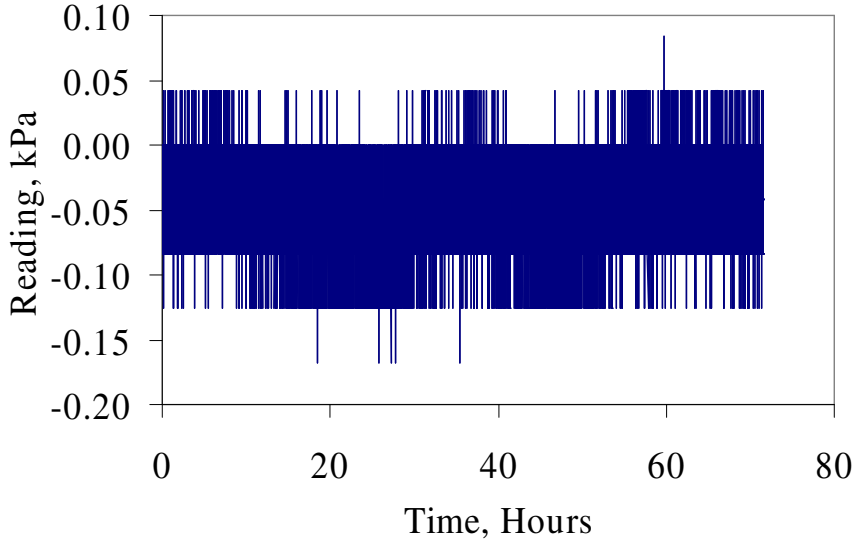


Figure 2.4. Stability of the cell pressure transducer with time.

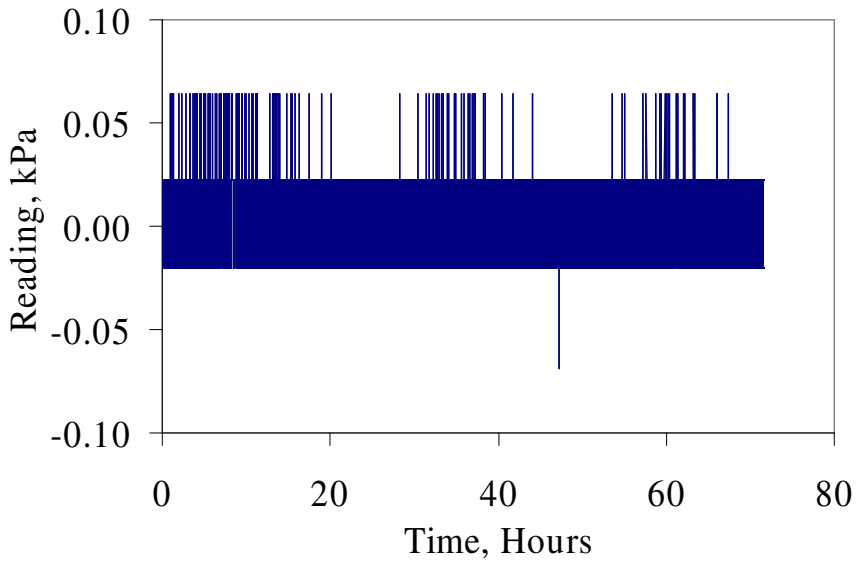


Figure 2.5. Stability of the pore pressure transducer with time.

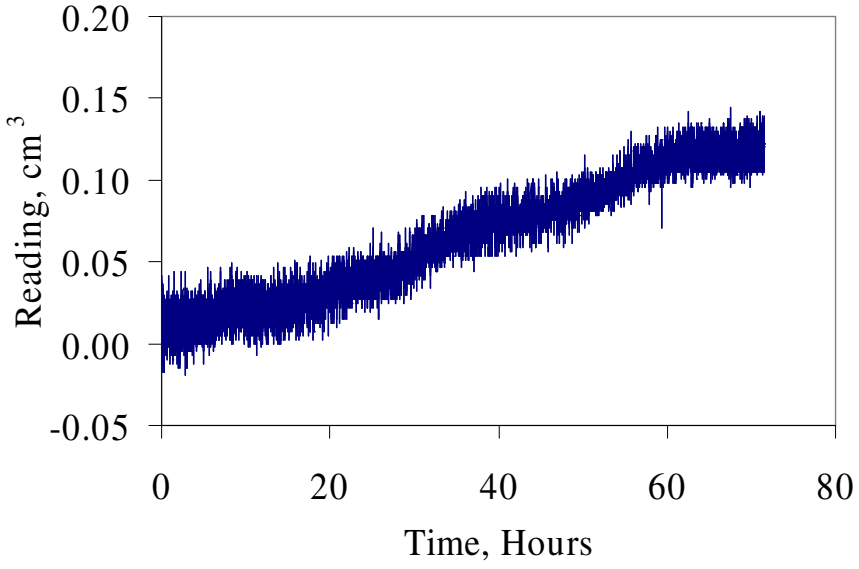


Figure 2.6. Stability of the differential pore pressure transducer with time.

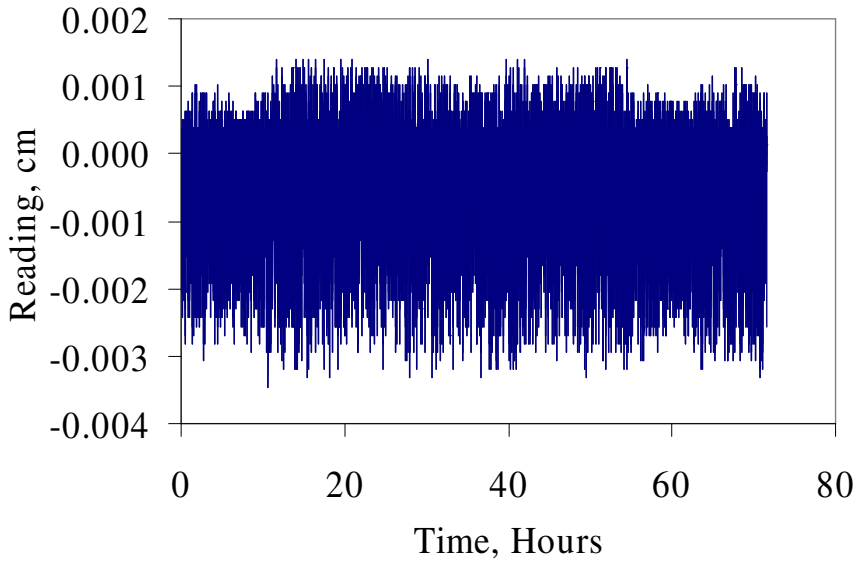


Figure 2.7. Stability of the axial LVDT with time.

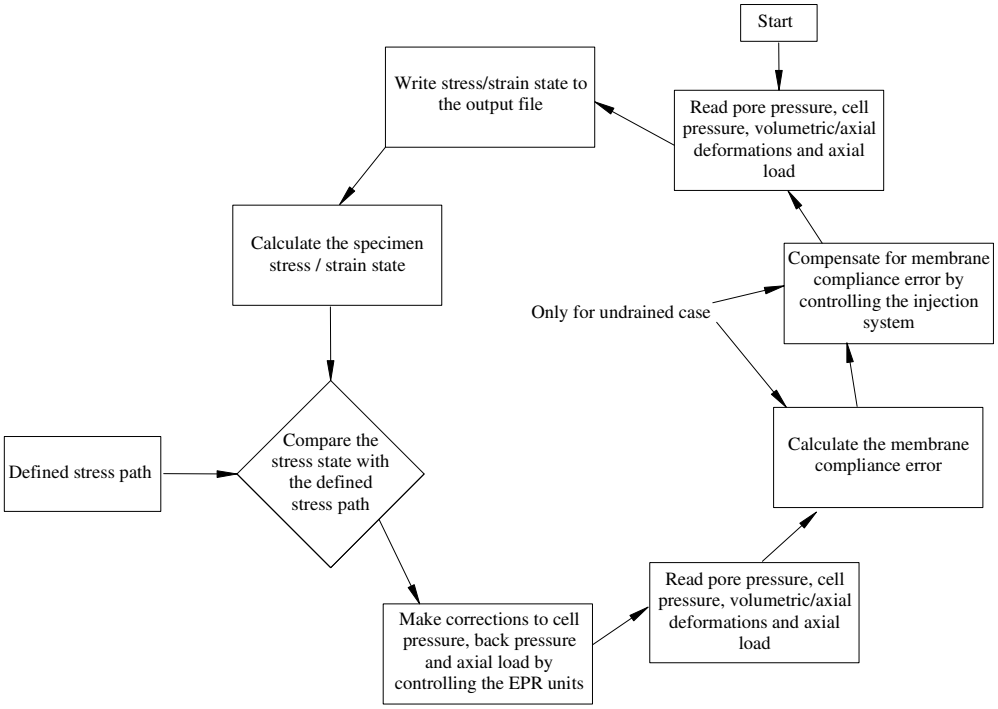


Figure 2.8. Schematic view of the algorithm used for performing stress path triaxial tests.

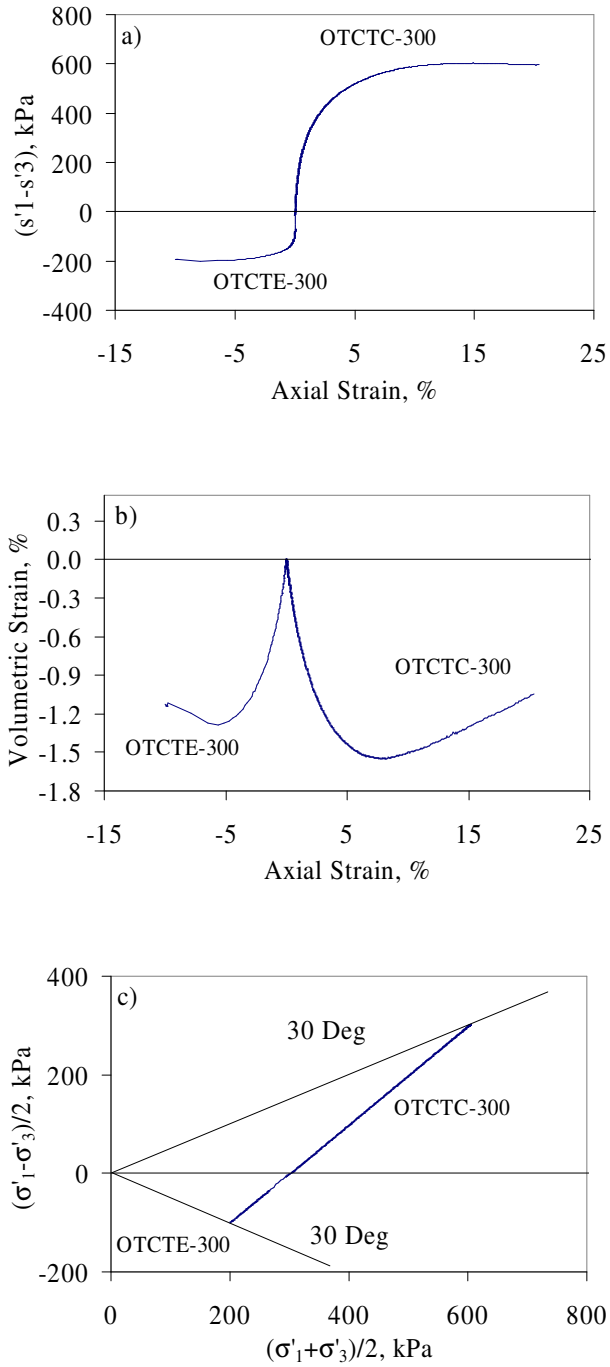


Figure 2.9. Drained behaviour of two identical Ottawa Sand specimens (a) Deviatoric stress vs. axial strain (b) Excess pore pressure vs. axial strain. (c) stress path.

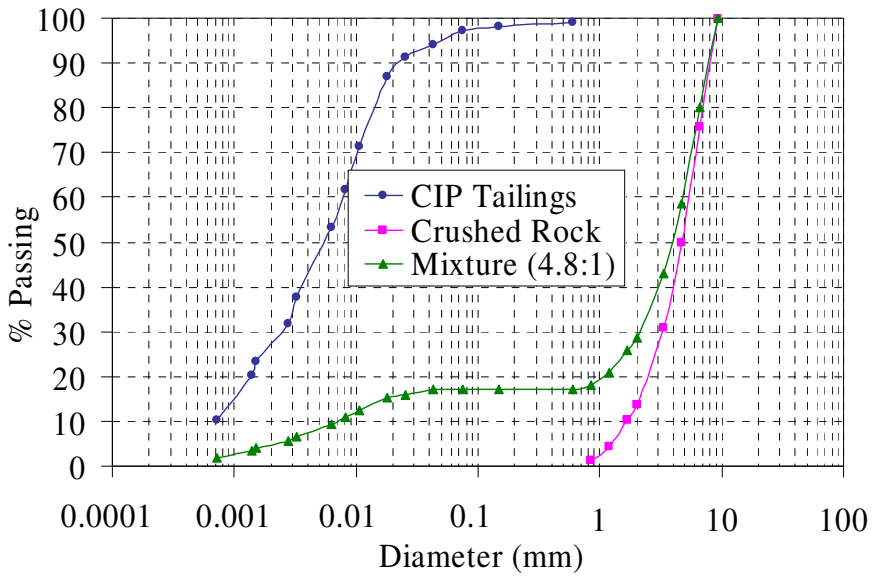


Figure 2.10. Typical particle size gradation curves for tailings, scalped waste rock, waste rock and tailings mixture (paste rock).

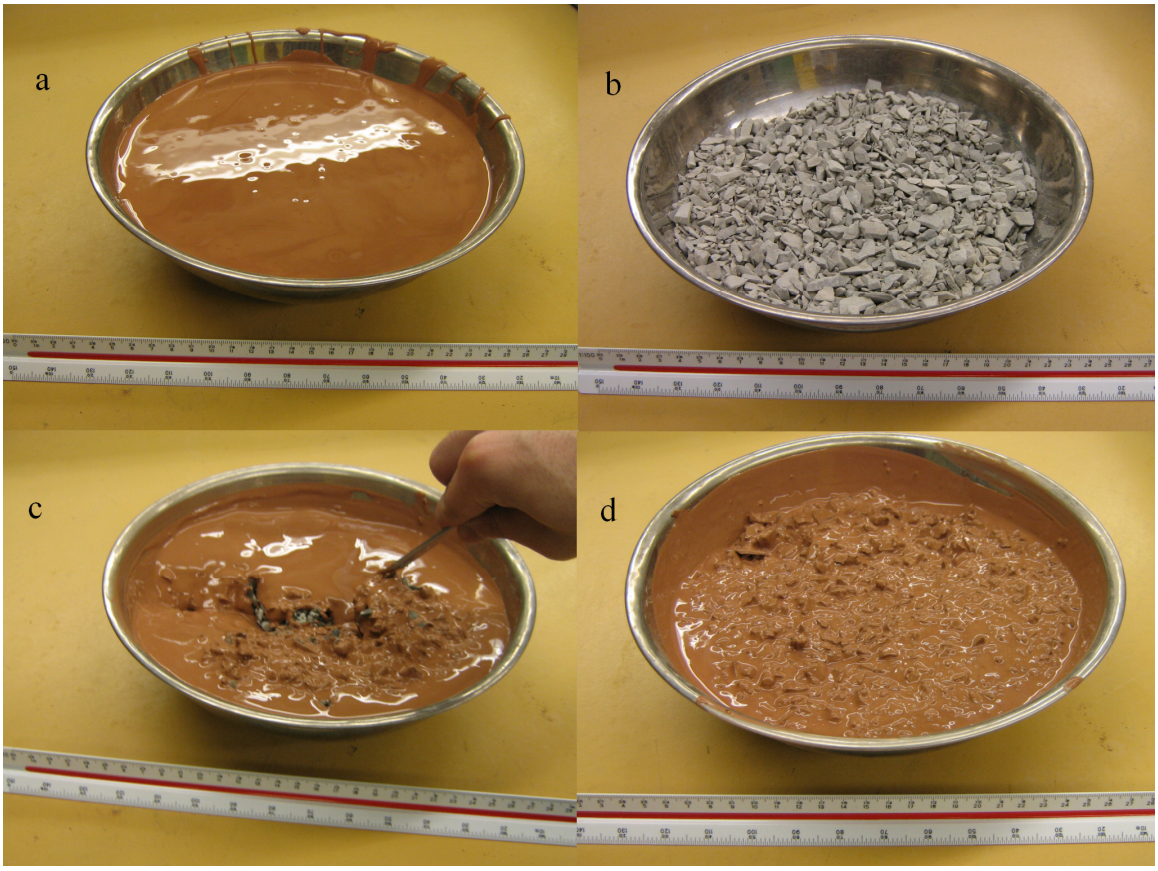


Figure 2.11. Pictures taken during preparation of slurry medium and paste rock – (a) Mixed tailings slurry medium at a specific water content (b) Scalped waste rock particles (< 9.4 mm) (c) Mixing rock and tailings using a bent tablespoon (d) Uniformly mixed rock and tailings (paste rock).

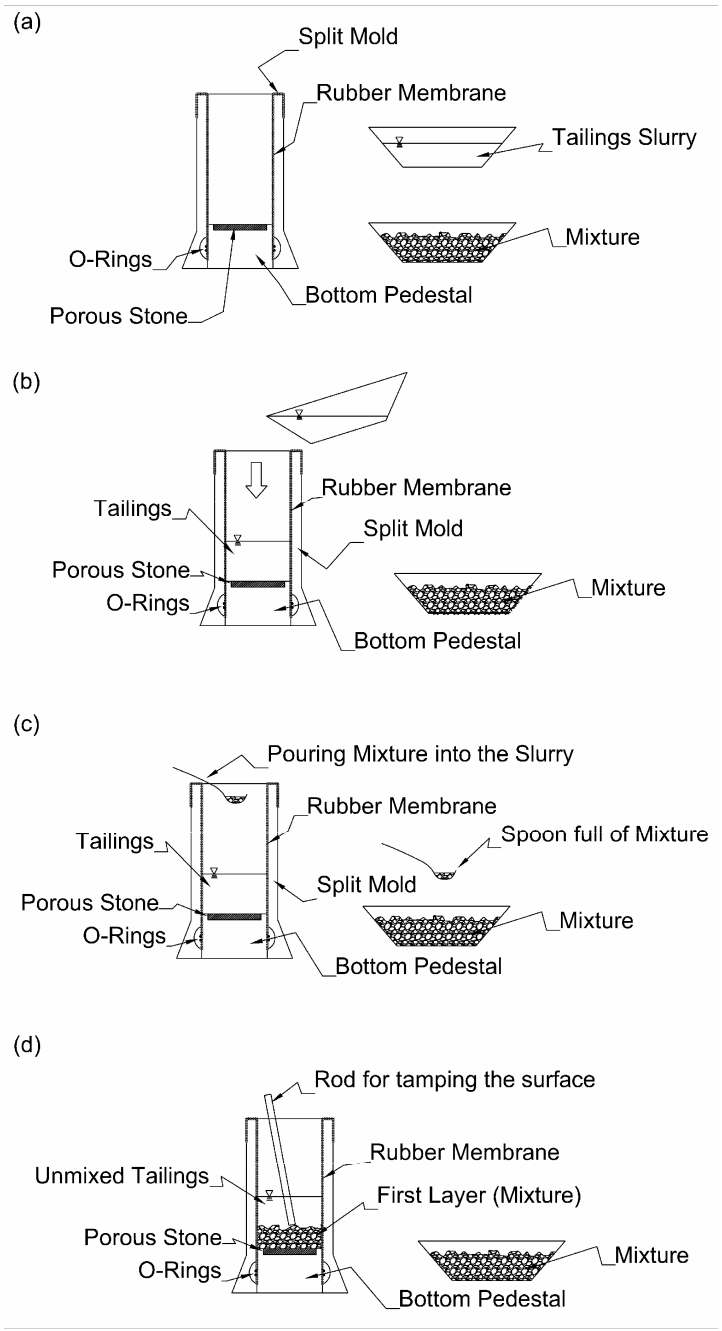


Figure 2.12. (a) Preparation of mould, slurry and mixture (b) pouring tailings slurry into the mould (c) placing the mixture into the slurry bath (d) tamping the mixture to form a flat surface.

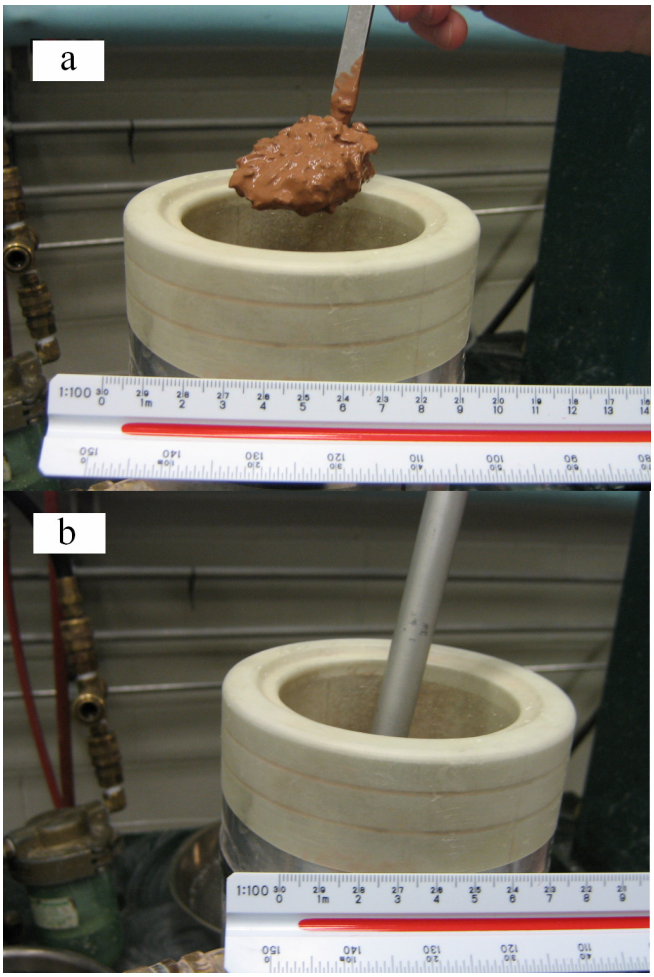


Figure 2.13. (a) Placement of paste rock in slurry medium and (b) use of rod for gentle tamping of the placed paste rock during specimen preparation.

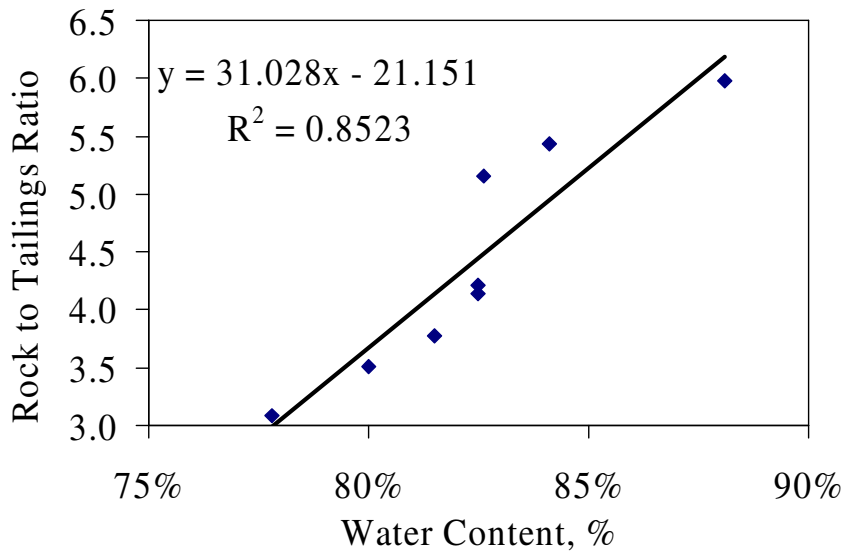


Figure 2.14. Variation of waste rock: tailings mix ratio (by weight) with slurry water content.

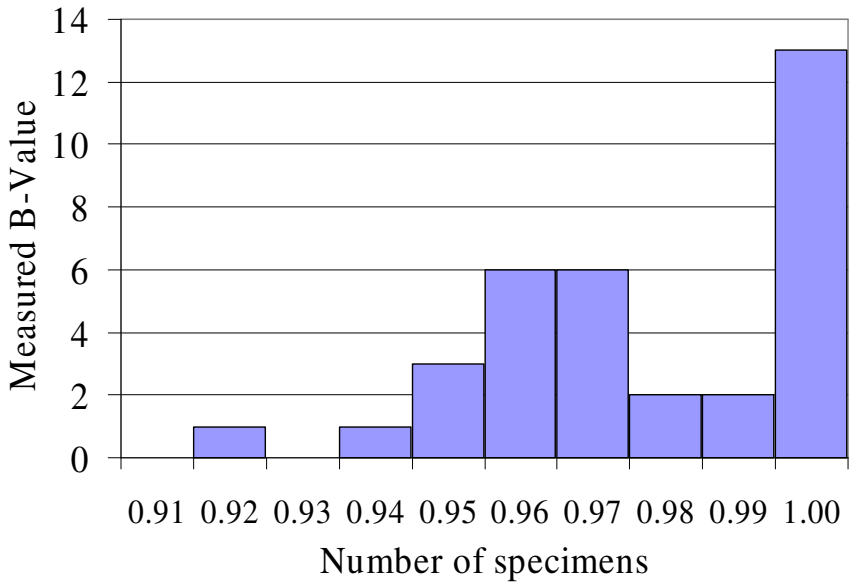


Figure 2.15. Histogram of measured B-values before triaxial shearing of paste rock specimens.

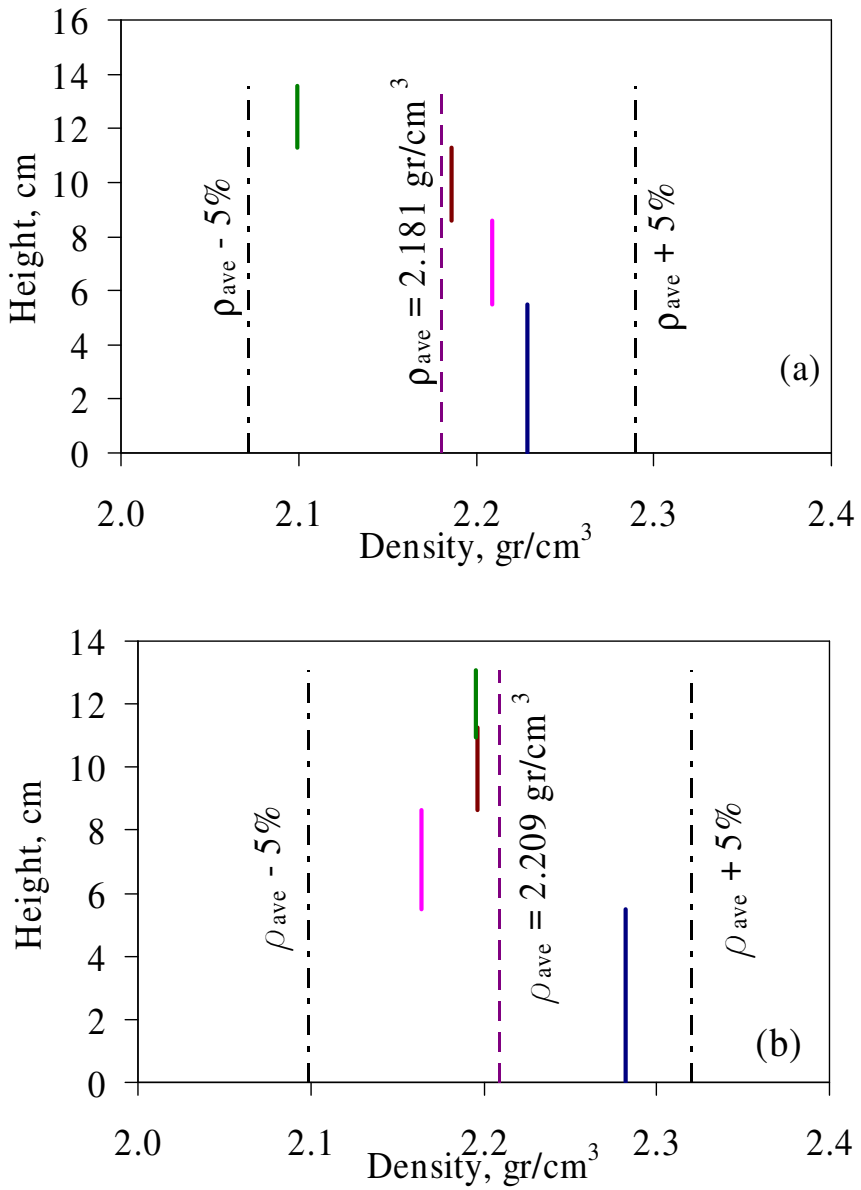


Figure 2.16. Density variations along the height of two specimens reconstituted by slurry displacement method (computed mean density for the specimens and 5% deviations from the mean are shown using dashed lines).

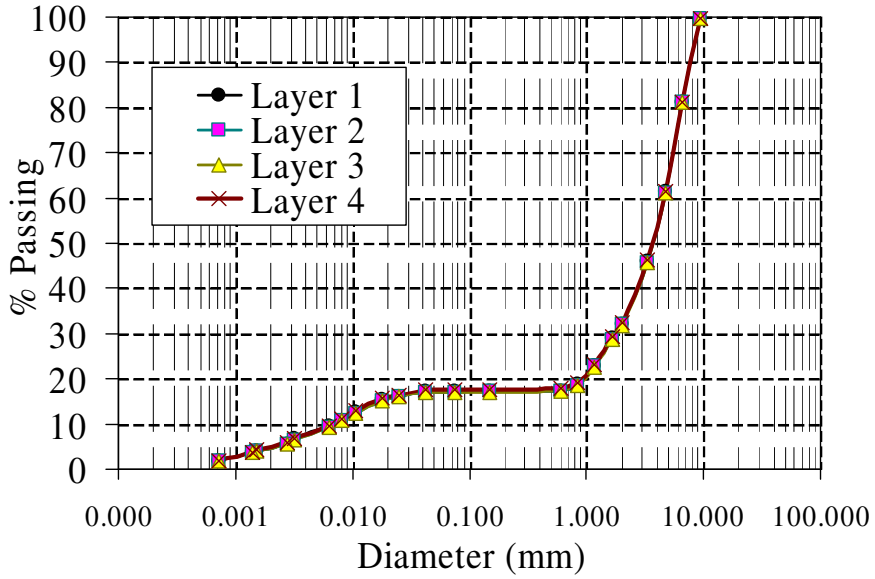


Figure 2.17. Gradation curve of various layers within a specimen.

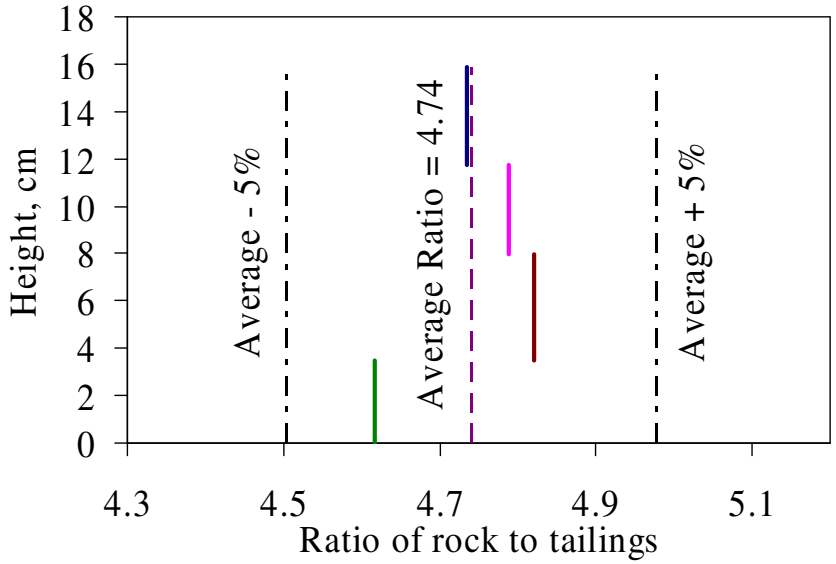


Figure 2.18. Variation of ratio of rock to tailings within a specimen.

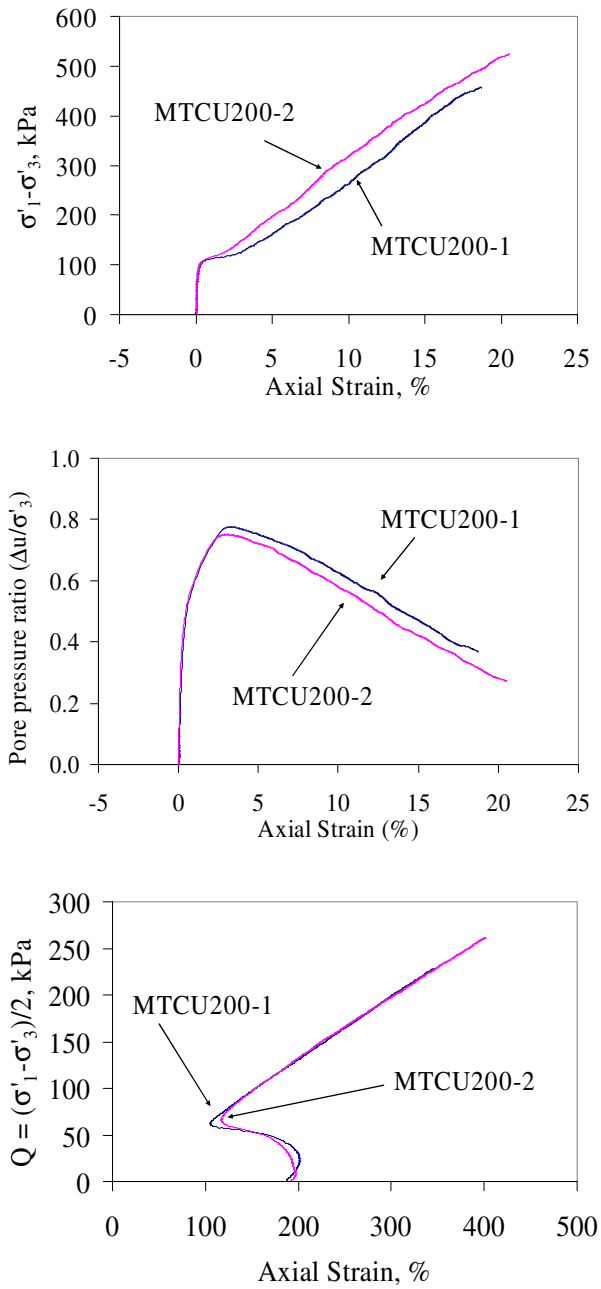


Figure 2.19. Undrained behaviour of two identical specimens (a) Deviatoric stress vs. axial strain (b) Excess pore pressure vs. axial strain. (c) stress path.

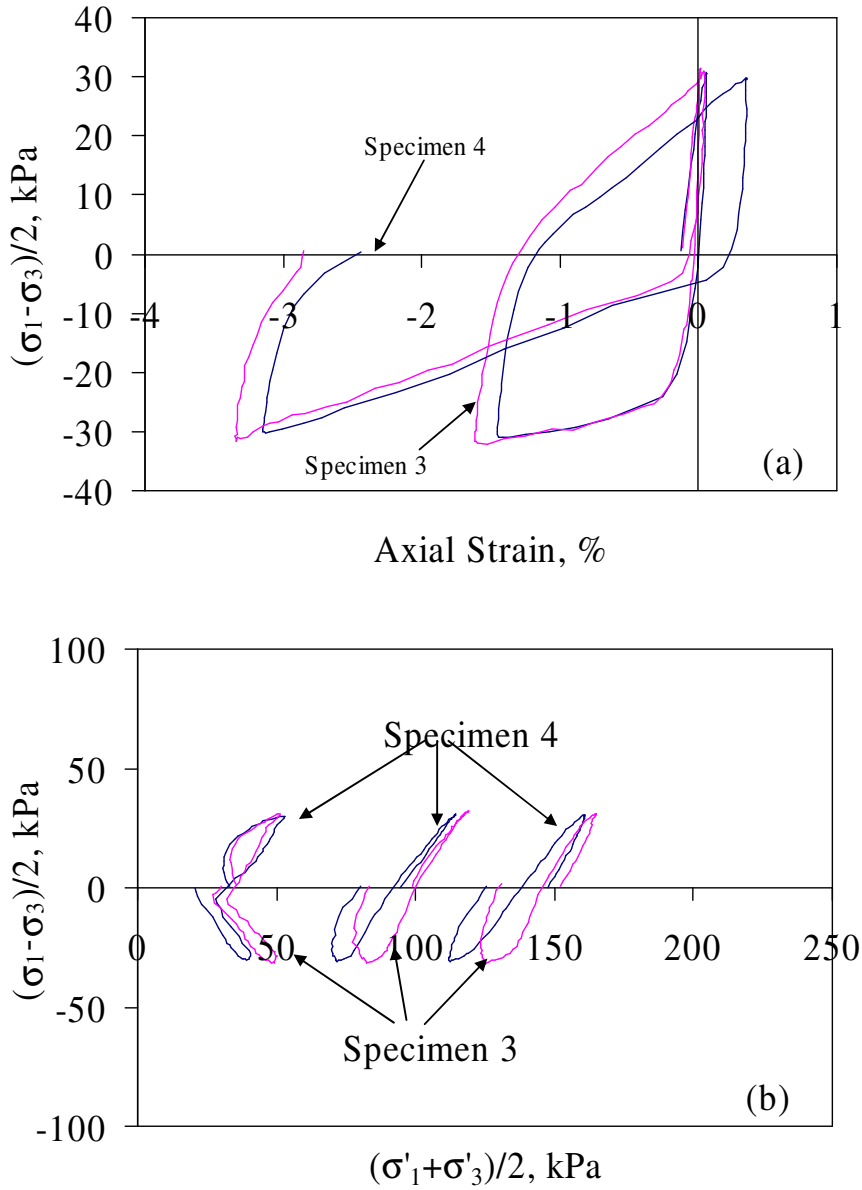


Figure 2.20. Cyclic undrained behaviour of three identical specimens (a) Q vs. axial strain (b) Q vs. Mean effective stress.

2.4. References

ASTM. 2002. Standard test method for particle-size analysis of soils (standard D422-63). *In* Annual Book of ASTM Standards. *Edited by* ASTM International, West Conshohocken, PA.

Ayoubian, A, and Robertson, P.K. 1998. Void Ratio Redistribution in Undrained Triaxial Extension Tests on Ottawa Sand. Canadian Geotechnical Journal, **35:** 351-359.

Baldi, G., Hight, D.W., and Thomas, G.E., 1998. A reevaluation of conventional triaxial test methods. Advanced Triaxial Testing of Soil and Rock, R.T. Donaghe, R.C. Chaney, and M.L. Silver, Eds., ASTM International West. Conshohocken, PA, pp. 219-263.

Casagrande, A. 1979. Liquefaction and cyclic deformation of sands: A critical review. *In* Proceedings of the 5th Pan-American Conference on Soil Mechanics and Foundation Engineering, Buenos Aires, Argentina, 1979. Harvard University, pp. 79-133.

Castro, G. 1969. Liquefaction of sands. Ph.D., Harvard University.

Emery, J.J., Finn, W.D.L., and Lee, K.W. 1973. Uniformity of saturated sand samples. ASTM STP, **523:** 182–194.

Junqueira, F., Rykaart, M., Weeks, B., Wickland, B., and Wilson, G.W. 2007. Emerging technologies in mine waste management. *In* Proceedings of the Professional Development Seminar on Emerging Technologies in Mine Waste Management, Vancouver, BC, April 2007. Association of Professional Engineers and Geoscientists of BC, Canada.

Khalili, A. and Wijewickreme, D. 2008. Method for reconstitution of highly gap-graded specimens for laboratory element testing, ASTM Geotechnical Testing Journal, **31:** 424-432.

Khalili, A., Wijewickreme, D., and Wilson, G.W. 2005. Some observations on the mechanical response of mixtures of mine waste and tailings. *In* Proceedings of 58th Canadian Geotechnical Conference, Saskatoon, 2005. , Canada.

Kuerbis, R. and Vaid, Y.P. 1988. Sand sample preparation; the slurry deposition method. Soils and Foundations, **28:** 107-118.

Ladd, R.S. 1978. Preparing test specimens using undercompaction. Geotechnical Testing Journal, **1:** 16-23.

Miura, S. and Toki, S. 1982. A sample preparation method and its effect on static and cyclic deformation-strength properties of sand. Soils and Foundations, **22:** 61-77.

Oda, M. 1972. Initial fabrics and their relations to mechanical properties of granular material. Soil and foundation, **12:** 17-36.

Sheeran, D.E. and Krizek, R.J. 1971. Preparation of homogeneous soil samples by slurry consolidation. Journal of Materials, **6:** 356-373.

Vaid, Y.P. and Chern, J.C. 1985. Cyclic and monotonic undrained response of saturated sands. *In* Advances in the Art of Testing Soils Under Cyclic Conditions: Proceedings of a session held in conjunction with the ASCE Convention.ASCE, New York, NY, USA, Detroit, MI, Engl, pp. 120-147.

Vaid, Y.P., Sivathayalan, S., and Stedman, D. 1999. Influence of specimen-reconstituting method on the undrained response of sand. Geotechnical Testing Journal, **22:** 187-195.

Vaid, Y.P. and Sivathayalan, S. 1998. Fundamental factors affecting liquefaction susceptibility of sands. *In* Proceedings of the International Workshop on the Physics and Mechanics of Soil Liquefaction, Baltimore, MD, 10-11 Sep 1998. Hopkings University, USA, pp. 105-120.

Vaid, Y.P. and Negussey, D. 1986. Preparation of reconstituted sand specimens. *In* Soil Mechanics Series. University of British Columbia, Vancouver, Canada. pp. 1-26.

Wickland, B.E., Wilson, G.W., Wijewickreme, D., and Klein, B. 2006. Design and evaluation of mixtures of mine waste rock and tailings. Canadian Geotechnical Journal, **43:** 928-945.

Wijewickreme, D. 1986. Constant volume friction angle of granular materials. M.A.Sc., University of British Columbia, Canada.

Wijewickreme, D., Sriskandakumar, S., and Byrne, P. 2005. Cyclic loading response of loose air-pluviated fraser river sand for validation of numerical models simulating centrifuge tests. Canadian Geotechnical Journal, **42:** 550-561.

Chapter 3.

Consolidation Properties of Paste Rock [i]

A laboratory research program was undertaken to study the static/cyclic shear response of the select mixtures of tailings and waste rock under fully saturated conditions. This chapter presents some of the observations made during one-dimensional (1-D) and hydrostatic consolidation of re-constituted specimens of tailings/waste-rock. Data available from consolidation tests conducted on the same material types, but with different coarse particle size, by Wickland et al. (2006) in a counterpart study have also been included for comparison purposes.

[i] A version of this paper has been published. Khalili, A., Wijewickreme, D., and Wilson G.W. 2007. Consolidation response of mixtures of waste rock and tailings with different particle size of coarse fraction. *In* proceedings of 60th Canadian Geotechnical Conference, Ottawa, Canada.

3.1. Experimental Aspects

3.1.1 Material tested

Materials used for this testing program essentially involve crushed sedimentary waste rock and Carbon In Pulp (CIP) tailings derived from a gold extraction process. Crushed waste rock particles are angular with some particles having very sharp edges. In order to obtain a gradation suitable for the triaxial shear testing herein, particles of waste rock larger than 9.4 mm were removed using a standard sieve. CIP tailings originate from a gold extraction process that involves autoclaving to oxidize sulphide minerals. The tailings material mainly consists of silt size particles and has a red-brown colour.

Particle size distribution curves of crushed rock and tailings are represented in Figure 3.1. A mixture ratio of around 4.8:1 waste rock to tailings was used for the construction of test specimens. This ratio is identical to the mixture ratio used by Wickland et al. (2006) in a counterpart study to examine the consolidation characteristics of waste rock and tailings mixtures with waste rock scalped of sizes greater than 50 mm. The scalped particle size (50 mm) is different from the size used for the study of shear response of the paste rock later in this research; nevertheless leading to the same mixture ratio of 4.8:1.

3.1.2 Specimen preparation

Specimens were prepared for one-dimensional and triaxial hydrostatic consolidation testing as outlined below:

1-D consolidation: The waste rock and tailings were blended manually with the use of a 30-mm wide spoon, and the mixed material was deposited into a 148-mm diameter polished aluminium cylinder to produce test specimens having a height of ~200 mm for 1-D consolidation. Prior to placement of the material the cylinder was partially filled with a slurry medium using the same tailings material. This new "slurry displacement" technique was specially developed to reconstitute saturated specimens with minimal opportunity for segregation, and the approach is described in chapter 2. The material was placed in the chamber in lifts of ~25 mm thickness with gentle tamping applied using a small rod after placement of each lift to ensure that a generally flat surface is formed.

In addition to the 1-D consolidation tests performed on mixed material, one 1-D consolidation test was also performed on a specimen entirely made of tailings material, commencing from a slurry state.

Triaxial consolidation: Triaxial specimens for hydrostatic consolidation were prepared using a split-mould capable of developing 76-mm diameter, 160-mm height specimens. The material was placed in the mould lined with a rubber membrane in accordance with

the same "slurry displacement" procedure described above. After constructing to the desired height, placement of top cap, and securing with the use of rubber membranes and o-rings, the reconstituted triaxial specimens were confined using an approximately 25 kPa vacuum applied through top and bottom drainage ports; at this point, the specimen-mould could be removed and the triaxial cell assembled in preparation for testing.

In order to avoid membrane puncture due to the sharp edges of crushed rock particles, the triaxial specimens had to be enclosed using two membranes, having thicknesses of 0.3 mm each. The saturated as-placed density, $\gamma_{(sat)}$, of all specimens ranged between 2.15 and 2.27 g/cm^3. These observations combined with density and gradation data obtained from dissected sacrificial specimens suggested that the preparation technique is effective in producing essentially identical specimens suitable for a systematic experimental study.

3.1.3 Test program

Information and test parameters pertaining to the two types of consolidation tests undertaken in this study are presented in Table 3.1a. Data from twenty-two (22) hydrostatic consolidation tests on specimens having identical gradations were available since all the specimens were prepared as a part of a test program involving the study of shear behaviour of hydrostatically-consolidated waste rock/tailings mixtures.

The tests from Wickland et al. (2006), used herein for comparison, are presented in Table 3.1b. As may be noted, in addition to the difference in specimen diameter, the maximum particle size (D_{max}) of the waste rock used by Wickland (2006) is about 5 times larger than those used in the present study.

During consolidation testing, a continuous record of test data was obtained by a computer interfaced data acquisition system. In 1-D consolidation, the test variables monitored consisted of full time-histories of applied vertical normal load and settlements due to consolidation which was a direct indicator of volumetric strain. In triaxial testing, full time-histories of applied hydrostatic effective stress ($\sigma'_{1c} = \sigma'_{2c} = \sigma'_{3c}$), volume change of the specimen and axial strain was monitored. The triaxial test results were corrected for the stiffness of membrane, as well as the membrane compliance.

3.2. Experimental Results

3.2.1 1-D consolidation response

The changes in the total void ratio (e) of the specimen during 1-D consolidation test CM1 (D_{max} = 9.4 mm) versus vertical consolidation effective stress (log σ'_v) is presented in Figure 3.2. The same test data plotted as changes in total volumetric strain versus consolidation vertical effective stress is presented in Figure 3.3. The results are

compared with those from similar tests (CM2, and CM3) performed by Wickland et al. (2006) on material with D_{max} = 50 mm.

The compression indices obtained from linear portions of CM1, CM2, and CM3 consolidation tests are 0.016, 0.027 and 0.022 respectively. It can be seen that, in spite of the significant difference in particle size between the present study (Dmax = 9.4 mm) and by Wickland et al. (2006, D_{max} of 50 mm), the overall trend of the change in void ratio versus effective stress is very similar. In terms of the total volumetric strain, ε_v, 1-D consolidation tests CM1 and CM2 yield very similar results while showing slight difference with the results of the test CM3. This difference can be attributed to the very first stage of loading under low vertical stress in which measurement of applied loads and volumetric strains is with difficulty.

Figures 3.4 and 3.5 present the variation of the coefficient of consolidation (C_v) and hydraulic conductivity (k) with applied pressure during consolidation tests CM1, CM2, and CM3 back calculated from settlement-time data in each loading stage. As may be observed, discrepancies were noted between the computed values of C_v and k from different tests. It is likely that the difficulties in accurately estimating time for 50% degree of consolidation (t_{50}) would have partly contributed to these observed anomalies. In other words, when applying small load increments during 1-D consolidation tests, differentiating between primary and secondary consolidation was found to be difficult due to steady and continuous variation of specimen volume with time. For example,

Figures. 3.6a and 3.6b show the displacement of the specimen versus time under a small load in test CM1 on both log time and root time scales. Interpreting the initial part of the consolidation curve in root-time scale is relatively difficult due to the gradual change of slope. Figure 3.6a shows various possible interpretations for the linear section of the chart leading to t_{90} values ranging from 6 to 32 minutes (i.e. C_v values of 0.042 to 0.227 cm^2/sec). Also, it can be seen that there is no clear distinction between primary and secondary consolidation in the log-scale plot (Figure 3.6b) therefore making it difficult to use log-time approach for determination of t_{50} and C_v.

The tailings void ratio vs. effective vertical stress relationship developed from 1-D consolidation testing of tailings slurry only (Test No. CT1) is plotted in Figure 3.7, and it is compared to the results from the counterpart Test No. CT2 from Wickland et al. (2006). It should be noted that the e vs. log σ'_v behaviour observed from the two specimens are almost identical after 20 kPa of effective stress in spite of the difference in specimen diameters. Figure 3.9 and 3.10 show the variation of coefficient of consolidation and permeability obtained from 1-D consolidation curves for CT1 and CT2 tests. Again, the results confirm that behaviour of tailings specimens are very similar (except at very small stress levels where the determination of k value can be subject to error due to difficulties in estimating time-factors).

Using the data obtained from consolidation tests and the mixture ratios, the void ratio of tailings fraction of mixture material during Tests No. CM1, CM2, and CM3 could be

computed. These "tailings component" void ratios have also been superimposed in Figure 3.7. In a similar manner, the void ratio in "rock component" of Tests No. CM1, CM2, and CM3 were computed, and compared in Figure 3.8 with the e vs. log σ'_v behaviour from a consolidation test conducted only using waste rock (Test No. CR1) by Wickland (2006).

It can be seen that, for a given externally applied σ'_v level, the "tailings component" void ratio is higher than those observed from the "tailings-only" tests CT1 and CT2. This suggests that the tailings particles have not experienced the full externally applied stress. Clearly, a major portion of the applied load is being carried by coarser skeleton (waste rock skeleton) further confirming the previous observations by Wickland et al. (2006).

The ratio of specimen diameter to maximum particle size (D_s/D_{max}) varied from ~6 in Tests No. CM2 and CM3 tests from a counterpart study (Wickland et al. 2006), to about 15 in Test No. CM1, and about 8 in all hydrostatic consolidation tests. This suggests that the consolidation behaviour is essentially identical between tests conducted with the specimen diameter to maximum particle size ratio (D_s/D_{max}) of 6 and 15. It appears that satisfaction of (D_s/D_{max}) $\geq$ 6 could be used as a general guideline to determine the acceptable maximum particle size in the preparation of gap graded materials for 1-D consolidation testing. This is in line with conclusion of Jamiolkowski (2005) that a minimum ratio of specimen size to maximum grain size of 5 is necessary to eliminate

particle size effects during laboratory element testing, with any ratio equal or greater than 8 being ideal.

3.2.2 Hydrostatic consolidation response

The variation of total void ratio (e) of the waste rock - tailings mixture obtained from a series of hydrostatic consolidation tests conducted on triaxial specimens of the tailings/waste rock mixture is presented in Figure 3.11. The 1-D consolidation response from Test No. CM1, which was conducted with material having the same gradation as the triaxial tests, is also presented in the same figure.

The (e) vs. (log σ'_v) response from triaxial specimens seem to generally follow the same trend observed in 1-D consolidation tests although compressibility index values (slope of the lines) are higher compared to 1-D consolidation tests. Compressibility index values for hydrostatic consolidation tests varied between 0.014 and 0.079 compared to the compressibility index value of 0.016 for 1-D consolidation test CM1. Slightly higher compressibility index values in hydrostatic consolidation tests can be attributed to the 3-D nature of the consolidation process in these tests. This difference can also be explained considering stress deformation principles in continuum mechanics. Compressibility index during hydrostatic consolidation is related to the bulk modulus; whereas compressibility of 1-D consolidation is proportional to the inverse of the

constrained modulus (m_v), and it can be easily shown that m_v would be generally greater than the bulk modulus for a given material.

Coefficients of consolidation derived from 3-D hydrostatic consolidation tests based on final time of primary consolidation (t_{100}) [Bishop and Henkel, 1962] is presented in Figure 3.12. In terms of the order-of-magnitude, the results are comparable to the counterpart 1-D consolidation tests on the same material.

3.3. Summary

Laboratory data from limited 1-D and hydrostatic consolidation tests conducted on specimens re-constituted from a mixture of tailings and blasted rock indicate the following:

- The consolidation response of mixtures of tailings and waste rock, in combination with that of tailings-only and rock-only specimens, suggests that a major portion of the applied load after consolidation is being resisted by the coarser waste rock skeleton. The results are in accord with the previous observations by Wickland et al. (2006) from tests on material mixtures from the same origin but with different coarse particle size.
- The variations in ratio of specimen diameter to maximum particle size ratio (D_s/D_{max}) from 6 to 15 did not significantly affect the observed general trend in

consolidation behaviour of the specimens (Note: D_s is the specimen diameter and D_{max} is the maximum particle size in the specimen). This is in accord with findings of Jamiolkowski et al. (2005) regarding the minimum acceptable specimen diameter to particle size.

- The (e) vs. (log σ'_v) response from triaxial hydrostatic consolidation generally followed the same trend observed in 1-D consolidation tests while yielding higher compressibility indices.
- The coefficient of consolidation (C_v) values obtained from observations during hydrostatic consolidation of triaxial specimens were found to be generally comparable with those derived based on one-dimensional consolidation testing of the same material.

Table 3.1. Consolidation test program - present study.

Test Number	Material	Rock to Tailings Ratio	Specimen Diameter (cm)	Max Particle Size (mm)
CM1	Mix	4.5:1	14.8	9.4
CT1	Tailings	N/A	14.8	~1
Hydrostatic Consolidation Tests (22 Tests)	Mix	4.8:1	7.6	9.4

Table 3.2. Consolidation test program - from counterpart study by Wickland et al. (2006).

Test Number	Material	Rock to Tailings Ratio	Specimen Diameter (cm)	Max Particle Size (mm)
CM2	Mix	4.4:1	30.8	50
CM3	Mix	4.8:1	30.8	50
CT2	Tailings	N/A	30.8	~1
CR1	Rock	N/A	30.8	50

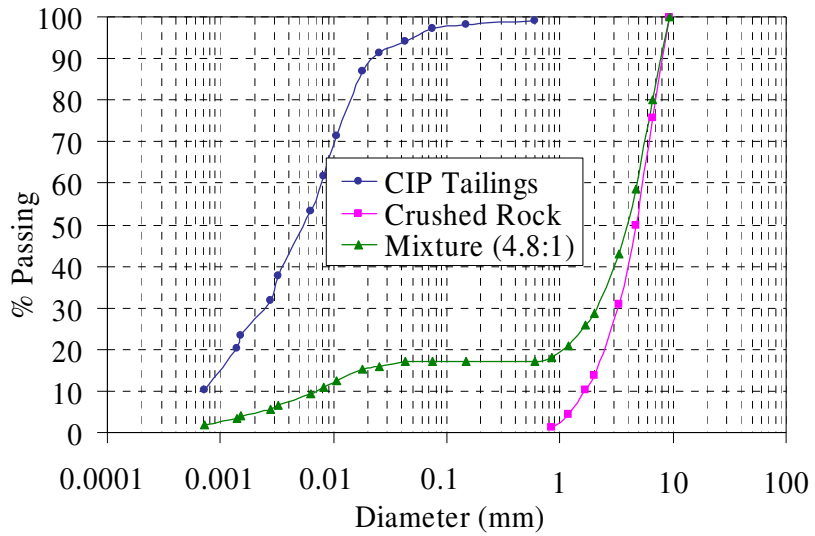


Figure 3.1. Particle size distributions of waste rock and tailings.

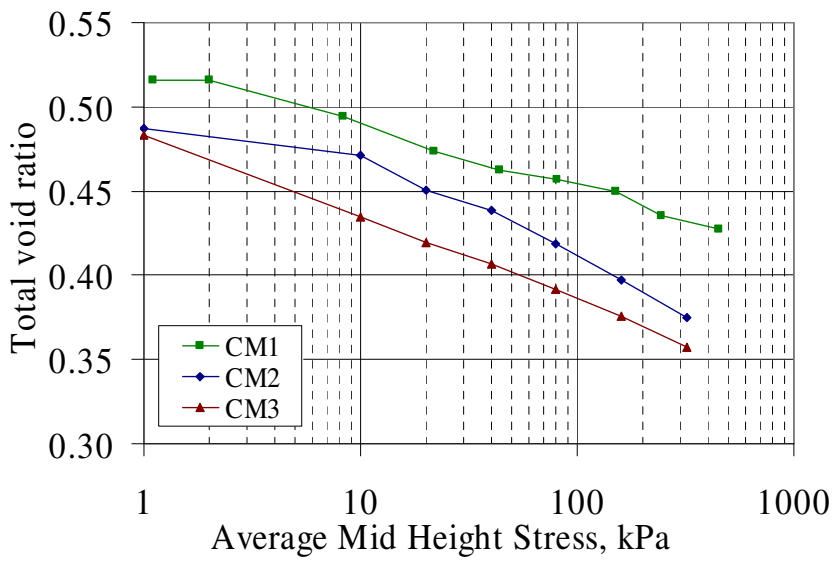


Figure 3.2. Variation of void ratio vs. effective stress of waste rock–tailings mixtures during 1-D consolidation. (Note: Data points shown above for Tests CM2 and CM3 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

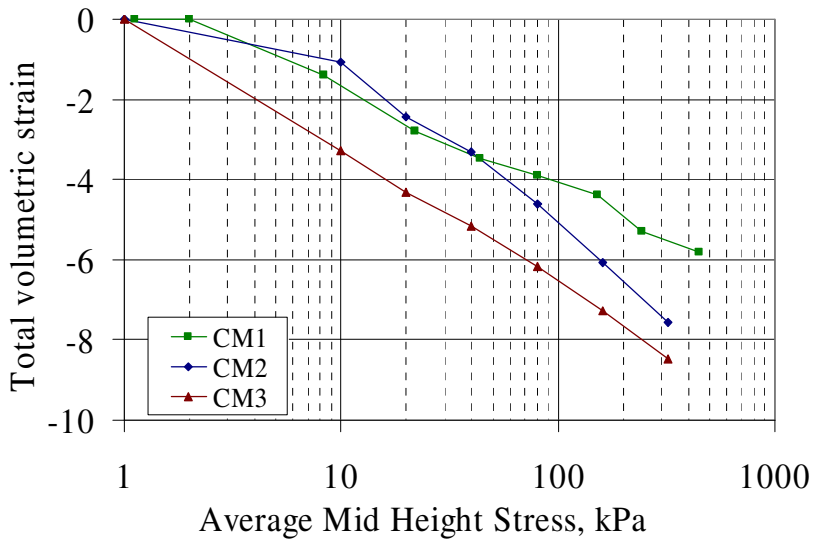


Figure 3.3. Variation of total volumetric strain vs. effective stress of waste rock – tailings mixtures during 1-D consolidation. (Note: Data points shown above for Tests CM2 and CM3 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

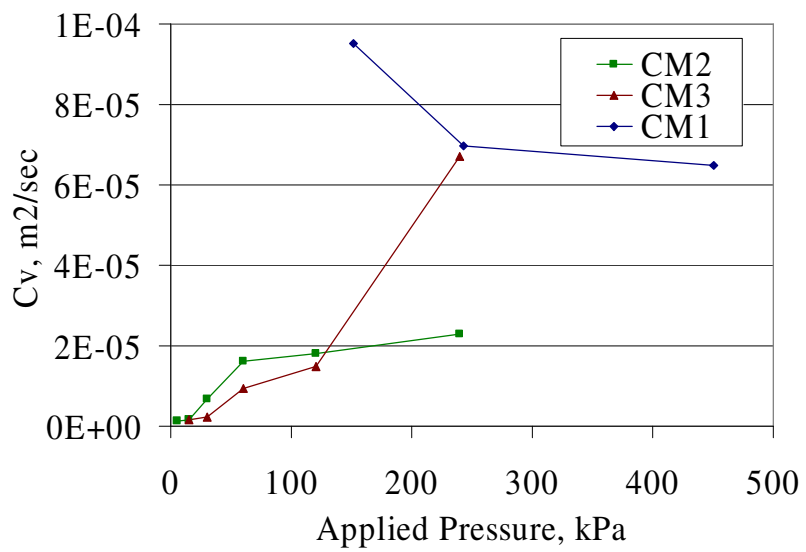


Figure 3.4. Variation of coefficient of consolidation with applied pressure in consolidation tests. (Note: Data points shown above for Tests CM2 and CM3 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

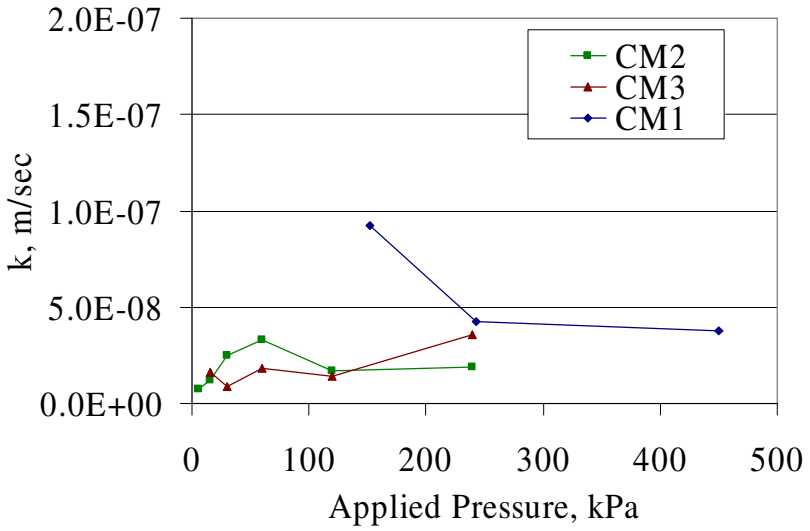


Figure 3.5. Variation of permeability with applied pressure during 1-D consolidation tests. (Note: Data points shown above for Tests CM2 and CM3 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

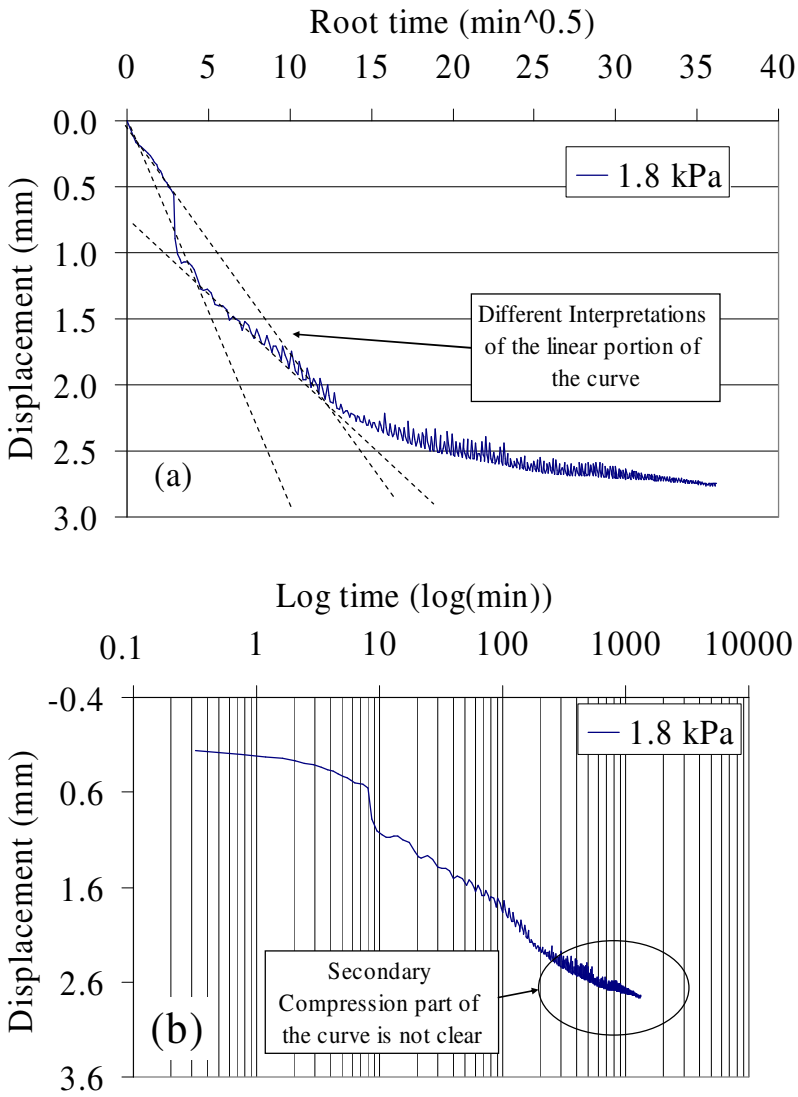


Figure 3.6. Displacement versus time during test CM1 (i.e. 1.8 kPa of axial load) (a) square-root (time) x-scale (b) $\log_{10}$(time) x-scale.

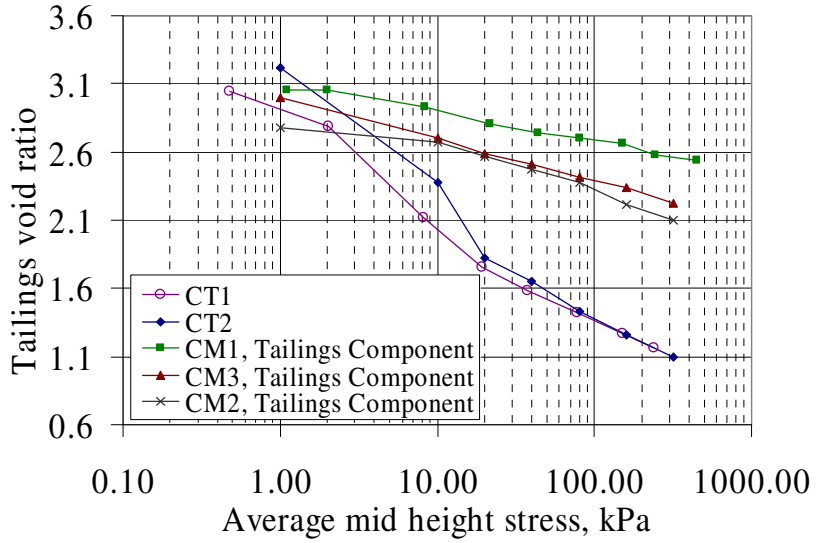


Figure 3.7. Variation of tailings void ratio vs. effective stress during 1-D consolidation testing. (Note: Data points shown above for Tests CM2 and CM3 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

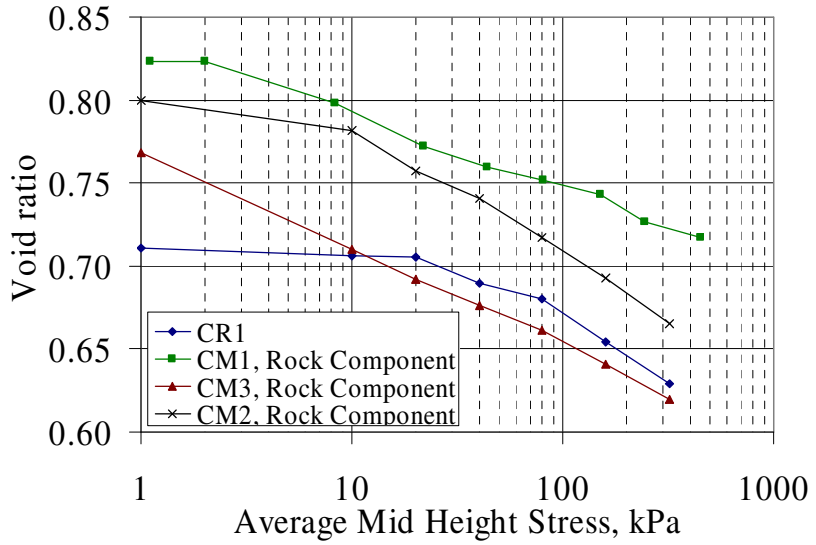


Figure 3.8. Variation of rock void ratio versus effective stress during different 1-D consolidation tests. (Note: Data points shown above for Tests CM2 and CM3 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

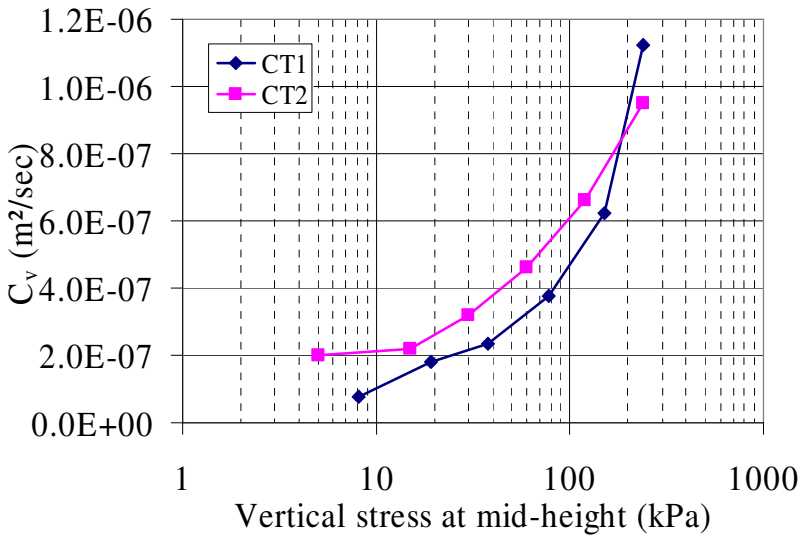


Figure 3.9. Variation of coefficient of consolidation with effective stress for CIP tailings material – Obtained from 1-D consolidation test. (Note: Data points shown above for Test CT2 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

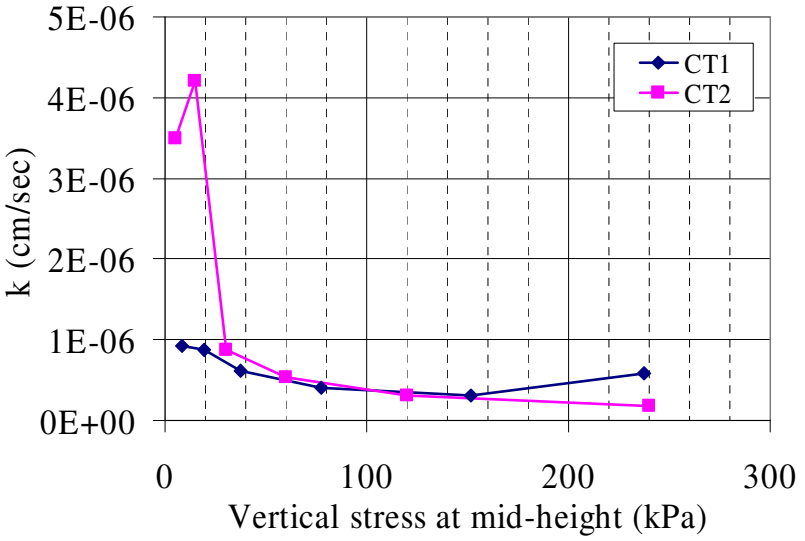


Figure 3.10. Variation of permeability with effective stress for CIP tailings material – Obtained from 1-D consolidation test. (Note: Data points shown above for Test CT2 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

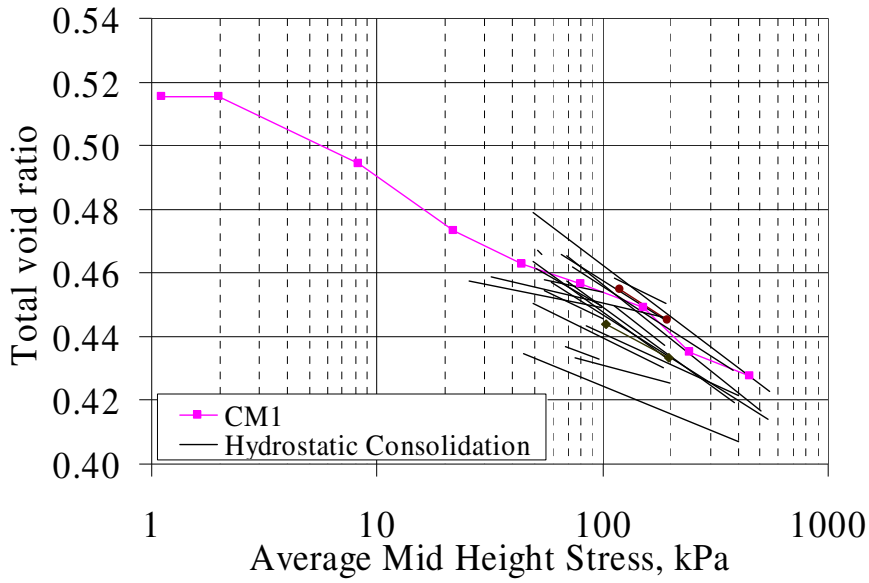


Figure 3.11. Variation of total void ratio of specimens during 1-D and hydrostatic consolidation tests performed on mixtures of tailings and blasted rock.

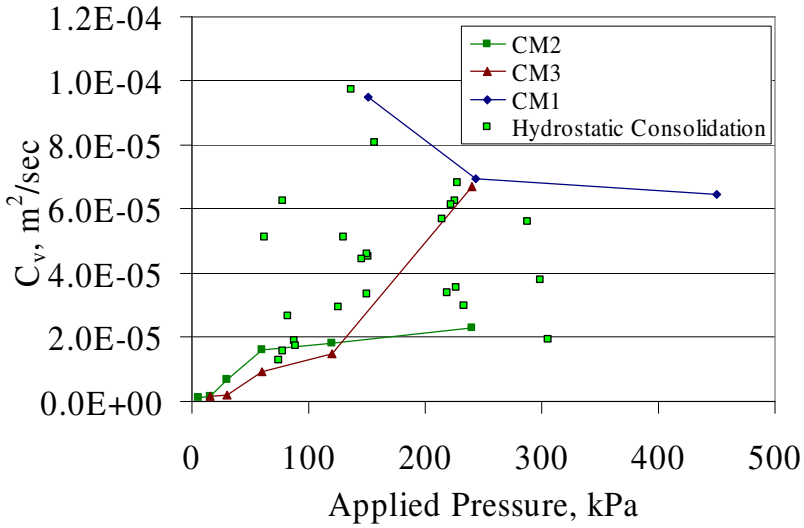


Figure 3.12. Coefficients of consolidation derived from hydrostatic and 1-D consolidation tests. (Note: Data points shown above for Tests CM2 and CM3 have been reproduced from Wickland et al. (2006)). © 2008 NRC Canada or its licensors. Reproduced by permission.

3.4. References

Bishop, A.W. and Henkel, D.J. 1962. The measurement of soil properties in the triaxial test. Edward Arnold Ltd., UK.

Jamiolkowski, M., Kongsukprasert, L., and Lo Presti, D. 2005. Characterization of gravelly geomaterials. *In* Proceedings of The Fifth International Geotechnical Conference, Cairo, Egypt, pp. 1-27.

Khalili, A. and Wijewickreme, D. 2008. New slurry displacement method for reconstitution of highly gap-graded specimens for laboratory element testing. ASTM Geotechnical Testing Journal, **31:** 424-432.

Khalili, A., Wijewickreme, D., and Wilson, G.W. 2005. Some observations on the mechanical response of mixtures of mine waste and tailings. *In* Proceedings of 58th Canadian Geotechnical Conference, Saskatoon, 2005. , Canada.

Kuerbis, R., Negussey, D., and Vaid, Y.P. 1988. Effect of gradation and fines content on the undrained response of sand. *In* Proceedings of Hydraulic Fill StructuresAug 15-18 1988. Publ by ASCE, New York, NY, USA, Fort Collins, CO, USA, pp. 330-345.

Thevanayagam, S. and Mohan, S. 2000. Intergranular state variables and stress-strain behaviour of silty sands. Geotechnique, **50:** 1-23.

Vaid, Y.P. and Chern, J.C. 1985. Cyclic and monotonic undrained response of saturated sands. *In* Advances in the Art of Testing Soils Under Cyclic Conditions: Proceedings

of a session held in conjunction with the ASCE Convention.ASCE, New York, NY, USA, Detroit, MI, Engl, pp. 120-147.

Wickland, B.E. 2006. Volume change and permeability of mixtures of waste rock and fine tailings. Ph.D., University of British Columbia, Canada.

Wickland, B.E., Wilson, G.W., Wijewickreme, D., and Klein, B. 2006. Design and evaluation of mixtures of mine waste rock and tailings. Canadian Geotechnical Journal, **43:** 928-945.

Wilson, G.W. 2001. Co-disposal of tailings and waste rock. Geotechnical News, **19:** 44-49.

Chapter 4.

Monotonic Shear Response of Highly Gap-graded Mixtures of Waste Rock and Tailings [i]

In a general context, tailings/waste-rock blends are essentially mixtures of heavily gap-graded gravel-sand-silt. Understanding the mechanical response is of great importance to support the development of new technology and material science required for such blended co-disposal of mine waste, and, in turn, the design of engineered dumps and/or barriers using this new material. For example, knowledge on shear strength (e.g., friction angle), shear deformation (e.g., shear modulus), compressibility (e.g., coefficient of

[i] A version of this chapter has been submitted for publication. Khalili, A., Wijewickreme, D. and Wilson G.W., Monotonic Shear Response of Highly Gap-graded Mixtures of Waste Rock and Tailings.

volume compressibility), and hydraulic characteristics (e.g., hydraulic conductivity) form key components in this regard.

Although the response of sands (or relatively coarse materials) has been the topic of extensive research during the past 50 years (Bishop and Henkel 1962, Rowe 1962, Lee and Seed 1967, Sivathayalan and Vaid 1998), the behaviour of mixtures of soils having significantly different particle sizes (e.g., sand and gravel, silty sand) has been studied only on a relatively limited scale. These mainly include studies on: (i) the effect of fines content on the stress-deformation characteristics of sands (Holtz and Ellis 1961, Thevanayagam 1998, Lade and Yamamuro 1997, Kuerbis et al. 1988); (ii) hydraulic flow characteristics with respect to internal stability of mixtures of coarse/fine-grained soils (Wagg and Konrad 1990, Fannin and Moffat 2006); (iii) packing density of particulate mixtures particularly on binary (two-size-only) particle mixtures (Furnas 1928; Vallejo 2001).

With particular reference to "paste rock", the research focus has been towards development of methods for mixing of waste rock and tailings (Wilson 2001, Wilson et al. 2003; Fines et al. 2003; Williams et al. 2003), and determination of the optimum mix ratios of waste rock and tailings (Williams et al. 1995; Wickland et al. 2006). Williams et al. (1995) for example concluded that the ideal mix ratio for paste rock is the one which causes the voids of the coarser fraction to be filled with tailings particles while the coarser particles still in contact with each other (i.e., "just filled" state).

Wickland (2006) studied the effect of mixture ratio on consolidation and air-entry properties of homogeneous mixtures of waste rock and tailings, with findings confirming that "just filling" the voids of the waste rock skeleton with tailings would result in a mixture with low compressibility (close to that of the waste rock matrix) while giving rise to the maximum tailings storage and reduced potential for ARD. Mixtures of mine waste rock and tailings were examined for hydraulic conductivity and consolidation response at the laboratory scale, and in a meso-scale column test by Wickland et al. (2006); in this, a conceptual model for mixture particle structure was used to quantitatively and qualitatively explain the observed behaviour. It was found that mixtures had less volume change due to consolidation than tailings alone. Volume change of mixtures due to consolidation was constrained by the presence of a load bearing 'waste rock skeleton.' In general, the laboratory studies confirmed values of hydraulic conductivity and consolidation parameters derived from a meso-scale column study. It was concluded that mixtures of waste rock and tailings can offer the hydraulic conductivity of tailings alone, and the consolidation volume change response of waste rock alone. In spite of this initial work, and except for the information available from sand/silt mixtures which has to be extrapolated, no detailed studies have been undertaken to understand the shear response of "paste rock". Clearly, laboratory element testing forms a key part in advancing the knowledge on this subject.

With this background, a laboratory triaxial element testing program was undertaken to study the mechanical behaviour of paste rock under shear loading conditions. The shear

stiffness and strength, the development of shear-induced volume changes and/or excess pore water pressures in a given soil mass is controlled mainly by parameters such as packing density, confining stress, particle fabric, etc. (Leroueil and Hight 2003). The research program was developed with these considerations in mind, while recognizing that field conditions would involve both "static" (monotonic) and "cyclic" loading conditions; the latter cyclic loading aspect was considered important since earthquake loading is a key consideration in mine waste management.

This chapter presents the findings of the research component examining the monotonic shear loading response of paste rock where a series of drained and undrained monotonic laboratory triaxial shear tests were performed on reconstituted mixtures of waste rock and tailings. Considering the strong participation of the 'waste rock skeleton' in governing the compressibility of mixtures (Wickland et al. 2006) due to consolidation, triaxial tests were also undertaken to study the shear response of reconstituted rock-only specimens. The results from a program of undrained cyclic shear loading testing conducted on paste rock, forming the second part of this research, are presented in the next chapter.

4.1. Experimental Aspects

4.1.1 Material tested

The raw materials used for this testing program essentially involve crushed sedimentary waste rock and Carbon in Pulp (CIP) tailings derived from a gold extraction process at the Porgera Gold Mine, Enga Province, Papua New Guinea.

The waste rock has an altered sedimentary origin and, as expected from a mining process, the particles are very angular in nature, with sharp edges. The specific gravity of rock particles was determined to be 2.7 (ASTM Standard D854-06). CIP tailings originate from a gold extraction process that involves autoclaving to oxidize sulphide minerals. Specific gravity of tailings particles is found to be 2.93 using ASTM Standard D854-06. The tailings material mainly consists of silt size particles and has a red-brown colour with a plastic limit of 21% and plasticity index of 12% (ASTM D4318).

Element testing of soils with coarse-grained particles involves several difficulties arising due to the size of particles. For example, the presence of oversized particles can lead to non-uniform laboratory specimens, in turn, leading to results not representative of the soil skeleton to be tested. Current state of practice on dealing with oversized particles for laboratory testing involves three different solutions (Siddiqi et al. 1987): (a) Replacing all oversized particles with an equal mass of large particles that fall within the range of

acceptable sizes (Donaghe and Townsend 1976); (b) Preparing a gradation parallel to the gradation of original material for testing; (c) Scalping the oversized particles and preparing the test specimen from the remaining of the material. Each of these methods has its own merits and deficiencies in giving rise to specimens that would replicate the actual behaviour of the material in the field. Studies have also shown that if oversized particles float among finer grains without touching, they can be removed without changing the behaviour of the mixture significantly (Siddiqi et al. 1987). In order to obtain a gradation suitable for the triaxial shear testing herein, particles of waste rock larger than 9.4 mm were removed using a Tyler standard sieve with 9.4 mm openings (ASTM Standard D422). This resulted in ratio of specimen diameter to maximum grain size of larger than 8 in all cases. Previous studies have shown that it is necessary to maintain a ratio of specimen size to maximum grain size of at least 5 in order to eliminate the size effect with 8 or more being the ideal ratio (Jamiolkowski 2005). Minimum and maximum void ratios of waste rock skeleton after scalping was determined to be 0.5 and 0.9 respectively (using ASTM Standards D4253 and D4254).

Particle size distribution curves of crushed rock and tailings are represented in Figure 4.1. A mixture ratio of around 4.8:1 waste rock to tailings was used for the preparation of paste rock test specimens. This ratio is identical to the mixture ratio used by Wickland et al. (2006) in a counterpart study to examine the consolidation characteristics of waste rock and tailings mixtures (with waste rock sizes greater than 50 mm removed). Wickland (2006) demonstrated that this mixture ratio yields the maximum density for the

mixture with tailings particles just filling the voids among rock grains for the material used herein. Some pictures taken during the preparation of the slurry medium and paste rock are presented in Figure 4.2. Grain size distribution of the mixed paste rock material used for this study is also shown in Figure 4.1.

4.1.2 Specimen preparation and laboratory testing considerations

The commonly available specimen reconstitution methods are not suitable to prepare uniform/homogeneous specimens of waste rock and tailings mixtures in a saturated condition. In recognition of this need, a new "slurry displacement" method was developed for preparing saturated, uniform, and repeatable specimens of highly gap-graded paste rock as documented in Khalili and Wijewickreme (2008). The approach essentially involves preparation of the paste rock by mixing of waste rock and slurry medium made of tailings in predetermined proportions, and then placing the mixture into a specimen-mould filled with the same slurry medium used for preparing paste rock. Since the slurry medium is displaced by the paste rock as the specimen is being constructed, the technique is named "slurry displacement" method. The method is capable of forming specimens with very good uniformity and degree of saturation (Khalili and Wijewickreme 2008). The mixture ratio between coarse and finer fractions of soil, and density of the specimens, can be controlled by changing the water content of the slurry.

Triaxial test specimens were prepared using a split-mould capable of developing 76-mm diameter, 160-mm height specimens. The material was placed in the mould lined with a rubber membrane in accordance with the above described "slurry displacement" procedure. When preparing rock-only specimens, clear de-aired water was used instead of slurry in the proposed method, and the rock particles were placed gently in the mould to minimize densification during specimen preparation. After placement to the desired height, placement of top cap, and securing with the use of rubber membranes and o-rings, the reconstituted triaxial specimens were confined using an approximately 25 kPa vacuum applied through top and bottom drainage ports; at this point, the specimen-mould was removed and triaxial cell assembled in preparation for testing.

In order to avoid membrane puncture due to the sharp edges of crushed rock particles, the triaxial specimens had to be enclosed using two membranes, having thicknesses of 0.3 mm each. Based on data from dissected sacrificial specimens, Khalili and Wijewickreme (2008) demonstrated that the above technique is effective in producing essentially identical, uniform, and repeatable specimens, which is an essential consideration for a specimen reconstitution technique to be acceptable for laboratory element testing.

The specimens were sheared using stress-controlled loading during triaxial testing. All triaxial specimens were subject to double drainage (top and bottom) during consolidation phase and drained shearing. The strain time-rate requirements were determined based on the commonly used guidelines for equalization of shear-induced pore pressure for

undrained triaxial testing (Bishop and Henkel 1962), with material-specific C_v values derived from 1-D consolidation tests and hydrostatic consolidation tests combined with time-pore water pressure responses observed in B-value determination phases. A rate of axial stress application of 480 kPa/hr was found to result in strain rates that met the above requirements for paste rock material. Hence, this rate was chosen for all triaxial tests on rock-only and paste rock materials.

4.2. Test Program

A series of drained and undrained triaxial monotonic shear tests were performed on reconstituted paste rock as well as reconstituted rock-only specimens at the geotechnical research laboratory at the University of British Columbia (UBC), Vancouver, Canada. A total of 17 undrained and 11 drained monotonic triaxial tests were undertaken, and the test parameters pertaining to each of the tests performed is shown in Tables 4.1, and 4.2. As may be noted, tests performed on reconstituted paste rock forms the key component of the test program.

Wickland et al. (2006) noted that the 'waste rock skeleton' has a major role to play in governing the compressibility of mixtures; as such, another series of tests were undertaken on reconstituted rock-only specimens to compare the response of rock-only material response with that of paste rock. All tests were conducted to explore the monotonic response of specimens hydrostatically consolidated to initial effective

confining stresses (σ'_{3c}) approximately ranging between 100 and 400 kPa, with the exception of a few additional tests performed at σ'_{3c} = 64 and 70 kPa only on paste rock specimens. The as-placed saturated density $\rho_{(sat)}$ of paste rock specimens were in a relatively narrow range between 2.15 and 2.24 g/cm^3 as summarized in Table 4.2. The triaxial specimens made of rock-only material had as-placed saturated density $\rho_{(sat)}$ ranging between 1.913 and 2.024 g/cm^3.

The determination of testing program for the tailings-only material was dependent on a number of considerations including the quantity of tailings material available for the overall research program. Since the assessment of performance of reconstituted paste rock is the key component of the test program, it was recognized that adequate number of tests should be undertaken on paste rock and sufficient tailings material quantities should be allotted for preparation of paste rock specimens. As such, it was decided to give first priority for using the limited available tailings for preparing specimens of paste rock and then manage the remaining material for tailings-only tests. With these constraints in mind, a cylindrical "block" sample of tailings-only material was prepared using the ~0.3 m diameter large-diameter consolidation device at UBC; herein, the cylindrical "block" sample is obtained by consolidating the tailings from a slurry state to a stress level slightly lower than that of the triaxial test using the consolidometer. Once consolidated, the block sample of tailings-only material was gently extruded from the consolidometer, and the specimens for triaxial testing were trimmed from portions obtained from the block sample.

In order to meet the constraints, it was decided to conduct only one monotonic triaxial compression shear test using a tailings-only specimen trimmed from the block sample. (Note: as may be noted from Chapter 5, three tailings-only specimens were also prepared for cyclic shear testing using the same block sample). The monotonic test was performed after hydrostatically consolidating the tailings-only specimen to a pressure of 200 kPa, which conforms to the mid-range of the σ'_{3c} values used in the overall testing program. In comparison to the rock-only (gravel size) or paste rock materials, the mechanical response of tailings-only material and silty soils in general have been subject of numerous studies in the past (Poulos et al. 1985; Wijewickreme et al., 2005); due to this significant available knowledge, it was judged that reduced number of tests on tailings-only material herein would not compromise the overall research program.

In triaxial compression tests, the shearing process was continued so that the specimen experienced axial strain of about 15%; in extension tests, it was impossible to reach such large strains since the specimen reached its peak strength at lower strain levels.

4.3. Correction for Membrane Compliance

All triaxial testing of paste rock and rock-only specimens were corrected to allow for the membrane compliance. A fluid injection system was used to inject pore water into (or withdraw out of) the specimens to compensate for the effect of membrane compliance

during undrained shear tests. Membrane penetration parameters were calculated by use of data obtained from hydrostatic unloading tests as described by Vaid and Negussey (1984). Figure 4.3 shows the membrane compliance factor (ε_m) for paste rock and rock-only triaxial specimens. As may be noted, the value of ε_m for rock specimens is approximately six times larger than the same parameter for mix specimens due to the larger peripheral voids.

Two identical specimens (MTCU400-1 and MTCU400-2) were tested with and without compensation for membrane compliance to evaluate the effect of injection during shear tests. The test parameters pertaining to these two specimens can be found in Tables 4.1 and 4.2. The observed mechanical response during undrained monotonic triaxial shear loading for the two tests is presented in Figure 4.4. The specimen that was tested without membrane compliance correction exhibited strain hardening response; whereas, the specimen with fluid injection displayed some mild strain softening prior to development of dilative behaviour. Such differences in the stress/strain response between the two tests justify the importance of compensating for membrane penetration in conducting truly undrained shear tests.

4.4. Test Results and Discussion

4.4.1 Hydrostatic consolidation response

The volumetric strain (after correction for membrane compliance) versus axial strain response obtained for a rock-only specimen during hydrostatic consolidation to a σ'_{3c} of about 400 kPa is compared with that obtained for a paste rock specimen during an identical consolidation process in Figure 4.5a. The rock-only specimen seems to have exhibited a more anisotropic response during hydrostatic consolidation than the paste rock specimen. It may be possible that the rock-only particles during placement may have acquired a particle configuration where more rock grain contacts are oriented in the vertical direction in comparison to the rock grain contact distribution that may have resulted during the preparation of paste rock specimens. Although no work was undertaken as a part of this research to directly assess the orientation of rock particle contact distributions arising from the two methods, it can possibly be argued that rock-only specimens where particles are placed under water would have had a relatively "free" opportunity to fall under gravity in comparison to paste rock specimens where rock particles would have had to find their final equilibrium positions while depositing in a relatively thick slurry medium – i.e., the rock particles in paste rock would have had better opportunity to be oriented randomly (or to reach a more isotropic arrangement). Additional research work would be required to investigate the validity of this postulate.

The volumetric strain vs. axial strain observed at the end of hydrostatic consolidation for some triaxial specimens made from paste rock and rock-only material is shown in Figure 4.5b. Again, it is important to note that all registered volumetric strains have been calculated after compensating for the membrane compliance. The gradient of the best-fit straight line indicate that the observed volumetric strain in triaxial specimens during hydrostatic loading tends to be very close to three times the axial strain for paste rock; this behavioural characteristic suggest that the as-prepared paste rock specimens have a close-to-isotropic particle fabric. Figure 4.5b also shows the gradient of the same best-fit straight line for rock-only specimens which shows more anisotropy compared to paste rock material.

4.4.2 Drained shear response

Typical stress-strain response, as well as volumetric strain response and stress paths of drained triaxial tests performed on paste rock specimens consolidated to different effective confining pressure (σ'_{3c}) values is shown in Figure 4.6. The test designation number corresponding to each response curve is also identified in the figure, and the information pertaining to the tests is presented in Tables 4.1 and 4.2. The numeric characters at the end of a given test designation number indicate the σ'_{3c} value, in kPa, corresponding to that particular test.

Under drained triaxial compression loading, all paste rock specimens displayed an initially contractive behaviour followed by a dilative response. The specimens clearly behaved in an increasingly contractive manner with increasing σ'_{3c}. It can also be noted from Figure 4.6 that the axial strain at which the maximum deviator stress occurs increases with increasing σ'_{3c}, indicating that the material would become less brittle with increasing confining stress. As may be noted from the effective stress paths, an essentially identical mobilized friction angle at peak deviator stress of about 42.8° was observed for all paste rock specimens. During triaxial extension, paste rock specimens yielded a generally brittle response with mild strain-softening (Figure 4.6). A mobilized friction angle at peak deviator stress of about 41.3° was observed for this extensional loading mode, and it is reasonably close to the friction angle of 42.8° observed above for paste rock in triaxial compression. It is of relevance to note that the part of the response to the left of the arrows marked in Figure 4.6 should be interpreted with caution since the specimens experienced "necking" approximately around these strain levels, in turn, these non-uniformities leading to potentially significant errors in the computed stress/strains. The above observed drained stress-strain characteristics of paste rock are similar to those previously observed for sands, or coarse-grained materials in general, as reported by Lee and Seed (1967) and Muir Wood (1991).

The stress-strain, volumetric strain response and stress paths of drained triaxial tests performed on rock-only specimens consolidated to effective confining pressures (σ'_{3c}) of 200 kPa and 400 kPa are presented in Figure 4.7. As may be noted from Table 4.2, the

skeleton void ratios of rock-only specimens (after consolidation to the desired σ'_{3c}) are reasonably comparable to those for the paste rock specimens. The mobilized friction angle at peak deviator stress for rock-only specimens (i.e., 42.1°) in compression is essentially identical to that (i.e., 42.8°) observed for paste rock in the same loading mode. The mobilized friction angle in rock-only specimens during extension was noted to be less than that of compression side (i.e., 36.9° in extension vs. 42.1° in compression as illustrated in Figure 4.7); this is somewhat different from the similar friction angles in compression and extension observed for paste rock. Also, mild strain-softening response that was observed during triaxial extension of paste rock (Figure 4.6) was not evident in the rock-only specimens (Figure 4.7). (Some rationalization of the observed anisotropic behaviour of rock-only material is attempted in Section 4.4.4). In spite of these differences, it is reasonable to state that, in terms of general trends, the drained response of rock-only specimens is fairly similar to that of paste rock; this supports the Wickland et al.'s (2006) observations that, in terms of compressibility, paste rock behaves more like waste rock alone.

4.4.3 Undrained shear response

The mechanical response of paste rock specimens initially consolidated to varying effective confining pressures (σ'_{3c}) are presented in Figure 4.8 (again, the numeric characters at the end of a given test designation number indicates the σ'_{3c} value, in kPa, corresponding to that particular test). As may be noted, the specimens deformed initially

in a contractive manner followed by a dilative response with the undrained stress paths from all tests following a consistent pattern. The point of phase transformation (i.e., with excess pore water pressure response Δu changing from contractive to dilative) was reached when the excess pore water pressure ratio ($u_r = \Delta u/\sigma'_{3c}$) was about 70-80%. The friction angle at the point of phase transformation (ϕ_{PT}) seems to be unique for a given loading mode. The friction angles (ϕ_{PT}) at phase transformation of paste rock material in compression and extension are determined to be 35.5° and 32.7°, respectively, and friction angles at maximum obliquity of shear stress (ϕ_{peak}) in compression and extension are 40.5° and 39.8°, respectively, as illustrated in Figure 4.8c.

It is clear that, during triaxial compression loading, the paste rock material displayed increased tendency for shear stiffness reduction and more contractiveness at higher σ'_{3c} levels. In contrast to the response of the specimens at lower σ'_{3c} levels, the paste rock specimen tested with σ'_{3c} = 400 kPa (i.e., Test No. MTCU400) shows a clear yielding point after which shear resistance stays nearly constant from about 1 to 2.5% axial strain, after which the specimen gained strength. This is conceptually similar to the "limited liquefaction" type strain development mechanism identified by Vaid and Chern (1985) based on their work on sand behaviour - although their definition involved clear post-peak reduction of shear resistance (i.e., strain-softening) with significant deformations prior to gaining strength. The observed initially contractive and then dilative tendencies during shear loading, and the increasing contractiveness with increasing σ'_{3c}, are in accord with the observations on counterpart drained monotonic tests conducted on paste

rock as described in the previous section. During triaxial extension tests, the shear resistance of paste rock reached a plateau phase after reaching a peak value although there was no visible strain-softening. During this plateau the axial strain was observed to increase without significant change in pore pressure (Figure 4.8). Amount of pore pressure build up during undrained extension was generally lower than that during undrained compression.

It is also important to note that, although a limited-liquefaction-type strain development mechanism was noted at higher effective confining stress levels, there was no strain-softening associated with significant loss of strength either in compression or extension modes of loading. Furthermore, the variations in the homogeneity of the material and loading paths experienced in the field are different from those applied in laboratory. It should be noted that the results presented herein are derived from element testing of paste rock and any extrapolations to field conditions should be undertaken with due attention paid to the variations between laboratory and field conditions. Limitations in the use of results from element test for the prediction of field behaviour are discussed in Section 6.5.

The mechanical response during undrained triaxial shearing of rock-only specimens consolidated to effective confining pressures (σ'_{3c}) between 100 kPa and 400 kPa are presented in Figure 4.9. The friction angles (ϕ_{PT}) at phase transformation of rock-only material in compression and extension are about 36.2°, which is slightly higher than those

observed from tests on paste rock. The friction angles at maximum obliquity of shear stress (ϕ_{peak}) in compression and extension are 41.7° and 36.2°, respectively, as illustrated in Figure 4.9c. The observed higher value of undrained ϕ_{peak} value during triaxial compression in comparison to that during triaxial extension for rock-only material is in accord with that observed for the same material during drained loading. Unlike for paste rock, there is no clear distinction between ϕ_{PT} and ϕ_{peak} of rock-only specimens during triaxial extension (i.e., $\phi_{PT} \sim \phi_{peak} \sim 36.2°$).

The pore pressure generation patterns of paste rock and rock-only specimens during triaxial tests are comparable in trend with some differences which may be pointed out with a close examination of pore pressure ratio charts (Figures 4.8b and 4.9b). The axial strains at which phase transformation (maximum pore pressure ratio) occurs during triaxial compression tests on paste rock specimens are about 3% (Figure 4.8b) with no apparent dependency to the consolidation effective stress. On the contrary, the counterpart axial strains at phase transformation for rock-only specimens are around 1% for the low confining pressure test (RTCU100) and increases with increase of consolidation effective stress to a value of 4% (RTCU400). The clear stress dependency of phase transformation point for rock-only specimens is further illustrated in Figure 4.10. It also worth noting that negative pore pressures generated during dilative response of rock-only specimens (after phase transformation point) are greater in absolute value than the counterpart values from paste rock specimens during both compression and extension triaxial tests. As can be noted, changes in the permeability of the specimens

due to presence of fines among coarser grains is caused some differences in pore pressure development patterns in triaxial tests on paste rock and rock-only specimens.

The mechanical response observed during undrained compression loading of one specimen made of tailings-only material (Test No. TTCU1, σ'_{3c} = 200 kPa) was also available to provide comparisons with the behaviour of paste rock material (see Figure 4.11). The tailings-only specimen deformed essentially in a contractive manner all the way to strains in the order of 15% without any strain-softening. A peak mobilized friction angle (ϕ_{peak}) of 30.6 degrees was noted when the specimen sheared to an axial strain of about 15%. This behaviour is similar to those typically noted for normally consolidated fine-grained soils (Muir Wood 1993; Sanin and Wijewickreme 2006).

The shear response of paste rock, rock-only, and tailings-only specimens (consolidated to σ'_{3c} = 200 kPa) in undrained triaxial compression can be superimposed as in Figure 4.12 to facilitate a direct comparison. It can be noted that, except for the slight weakness in terms of strength and deformation characteristics, the behaviour of the paste rock is still closer to the behaviour of rock-only material than that of tailings-only material. This is in accord with the observations by Wickland (2006) and Khalili et al. (2007), based on 1-D consolidations tests, that the rock skeleton mostly controls the shear resistance in "just filled" paste rock.

4.4.4 Possible reasons for the observed anisotropy in friction angle for rock-only material

The observed difference between the mobilized friction angles at failure between compression and extension loading modes for the behaviour of rock-only material (see Figures 4.7 and 4.9) is not typical, and not in accord with those previously observed for sands (e.g., Vaid and Chern 1985). It is important to note that the essentially similar friction angles in both extension and compression loadings observed for Ottawa sand using the same device in this study suggests that the above noted discrepancy for rock-only material is very likely a result of the anisotropy in arrangement of the rock-only particles.

As noted in Section 4.4.1, it may be possible that the rock-only particles during placement may have acquired a particle configuration where more rock grain contacts are oriented in the vertical direction in comparison to the rock grain contact distribution that may have resulted during the preparation of paste rock specimens. The observed lower friction angle in extension (in comparison to compression) loading can be argued to be in accord with this thinking on the fabric anisotropy. As discussed earlier, more research is warranted on this subject to reach firm conclusions; particle flow modelling using discrete element techniques would be one way to study and verify this postulate as a part of future studies on this subject.

4.5. Summary

A detailed laboratory research program was undertaken to investigate the monotonic shear response of mixtures of waste rock and tailings (i.e. paste rock). Because of the low permeability inherited from the fine tailings leading to potential reduction of ARD, and high shear strength of coarse waste rock leading to increased mechanical stability, this idea of combining waste rock and tailings to form paste rock for disposal is now receiving significant attention from the point of view of sustainable mine waste management practices.

Triaxial element tests were performed on hydrostatically consolidated specimens of paste rock, rock-only and tailings-only materials. Paste rock specimens were prepared to represent a state where the tailings particles "just fill" the void spaces in the rock particle matrix, a condition that would provide the optimum density of the mixed material.

Under drained triaxial compression loading, all paste rock specimens displayed an initially contractive behaviour followed by dilative response. The specimens behaved in an increasingly contractive manner with increasing σ'_{3c}. The observed behavioural pattern during undrained compression loading of this material was generally in harmony with the findings from the drained monotonic tests. In undrained loading at higher effective confining stress levels, limited-liquefaction-type strain development mechanism was noted in paste rock; however, no strain-softening associated with significant loss of

strength was noted both in compression as well as extensional modes of undrained loading. The data presented here showing the monotonic behaviour of tailings-only material is too sparse to reach a definite conclusion. Nevertheless, the comparison of the behaviour of two materials suggests that paste rock prepared to meet "just filled" conditions is likely to behave in a more dilative way than tailings-only material and therefore is less likely to experience flow deformation under monotonic (static) loading conditions.

Except for the slight weakness in terms of strength and deformation characteristics, the behaviour of the paste rock is still closer to the behaviour of rock-only material than that of tailings-only material. This suggests that the rock skeleton mostly controls the shear resistance in "just filled" paste rock, and it is in accord with the observations based on 1-D consolidation tests conducted on the same material.

Table 4.1. Key parameters related to various tests.

Test No.	Specimen Type	Drainage / Loading	Cell Pressure	Back Pressure	Consolidation Effective Stress	Injection
(1)	(2)	(3)	(4)	(5)	(6)	(7)
MTCD65	Mix	DC	169.7	105.4	64.3	
MTCD100	Mix	DC	245.1	145.3	99.8	
MTCD200	Mix	DC	350.4	165.7	184.7	
MTCD400	Mix	DC	565.7	165.0	400.7	
MTCD500	Mix	DC	681.1	174.6	506.5	
MTED200	Mix	DE	410.7	217.1	193.6	
MTED400	Mix	DE	608.2	208.6	399.6	
MTCU65	Mix	UC	219.3	149.7	69.6	Yes
MTCU100	Mix	UC	280.0	184.0	96.0	Yes
MTCU200-1	Mix	UC	335.5	149.7	185.8	Yes
MTCU200-2	Mix	UC	579.1	388.3	190.8	Yes
MTCU200-3	Mix	UC	438.2	249.6	188.6	No
MTCU400-1	Mix	UC	568.6	185.5	383.1	Yes
MTCU400-2	Mix	UC	531.7	148.5	383.2	No
MTEU100	Mix	UE	298.9	202.2	96.7	Yes
MTEU200	Mix	UE	358.2	171.1	187.1	Yes
MTEU370	Mix	UE	588.4	223.0	365.4	
RTCD200	Rock	DC	407.6	204.7	202.9	
RTCD400	Rock	DC	558.1	161.1	397.0	
RTED200	Rock	DE	392.5	191.5	201.0	
RTED400	Rock	DE	590.9	189.3	401.6	
RTCU100	Rock	UC	189.5	88.9	100.6	Yes
RTCU200	Rock	UC	316.2	118.0	198.2	Yes
RTCU400	Rock	UC	515.7	135.5	380.2	Yes
RTEU100	Rock	UE	240.7	140.8	99.9	Yes
RTEU200	Rock	UE	423.4	223.2	200.2	Yes
RTEU400	Rock	UE	603.2	212.7	390.5	Yes
TTCU1	Tailings	UC	301.3	101.9	199.4	

Description of Columns: (1) Test No., (2) Specimen Type, (3) Drainage and Loading Condition: DC: Triaxial Drained Compression, DE: Triaxial Drained Extension, UC: Triaxial Undrained Compression, UE: Triaxial Undrained Extension, (4) Cell Pressure (kPa), (5) Back Pressure (kPa), (6) Nominal Consolidation Pressure (kPa), (7) Injection for membrane compliance correction.

Table 4.2. Key parameters related to various tests.

Test No. (1)	Total Density (2)	Rock to tailings ratio (3)	e^f_{total} (4)	e^f_{rock} (5)	$e^f_{tailings}$ (6)	$(R_d)^f_{rock}$ (7)
MTCD65	2.165	4.85:1	0.491	0.774	3.076	35.3
MTCD100	2.221	4.83:1	0.423	0.695	2.641	54.8
MTCD200	2.178	4.85:1	0.449	0.724	2.814	47.6
MTCD400	2.239	4.32:1	0.374	0.667	2.127	61.8
MTCD500	2.195	4.62:1	0.403	0.684	2.424	57.6
MTED200	2.194	4.95:1	0.444	0.712	2.825	50.5
MTED400	2.174	4.80:1	0.439	0.716	2.728	49.7
MTCU65	2.177	4.96:1	0.477	0.751	3.039	41.0
MTCU100	2.154	4.94:1	0.501	0.780	3.187	33.7
MTCU200-1	2.166	4.81:1	0.467	0.748	2.907	41.6
MTCU200-2	2.174	4.91:1	0.462	0.737	2.927	44.5
MTCU200-3	2.181	4.68:1	0.459	0.746	2.789	42.3
MTCU400-1	2.157	4.89:1	0.458	0.733	2.89	45.3
MTCU400-2	2.192	4.64:1	0.417	0.698	2.520	54.0
MTEU100	2.187	4.84:1	0.461	0.739	2.880	44.0
MTEU200	2.214	4.85:1	0.415	0.683	2.599	57.7
MTEU370	2.190	4.99:1	0.421	0.684	2.700	57.6
RTCD200	1.990		0.705	0.705		52.3
RTCD400	1.995		0.676	0.676		59.6
RTED200	1.996		0.704	0.704		52.5
RTED400	1.962		0.734	0.734		45.1
RTCU100	1.947		0.779	0.779		34.0
RTCU200	1.913		0.776	0.776		34.8
RTCU400	1.917		0.756	0.756		39.8
RTEU100	1.945		0.791	0.791		31.1
RTEU200	1.951		0.797	0.797		29.6
RTEU400	1.995		0.682	0.682		58.0
TTCU1	1.836		1.124		1.124	

Description of Columns: (1) Test No. as in Table 4.1, (2) Total Density (gr/cm^3), (3) Rock to tailings ratio, (4) Final total void ratio after consolidation, (5) Final rock void ratio after consolidation, (6) Final void ratio of tailings component after consolidation, (7) Final relative density of rock skeleton after consolidation.

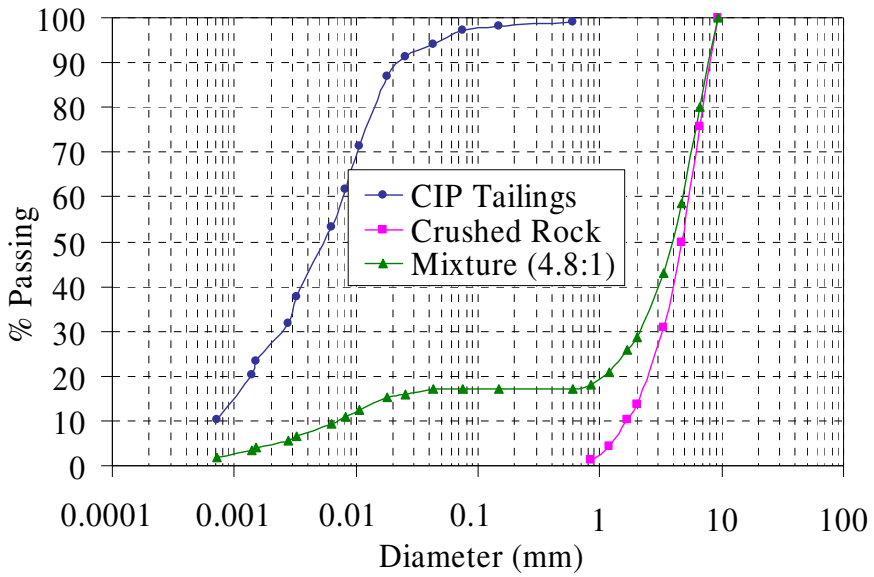


Figure 4.1. Particle size distributions of paste rock, waste rock-only and tailings-only materials.

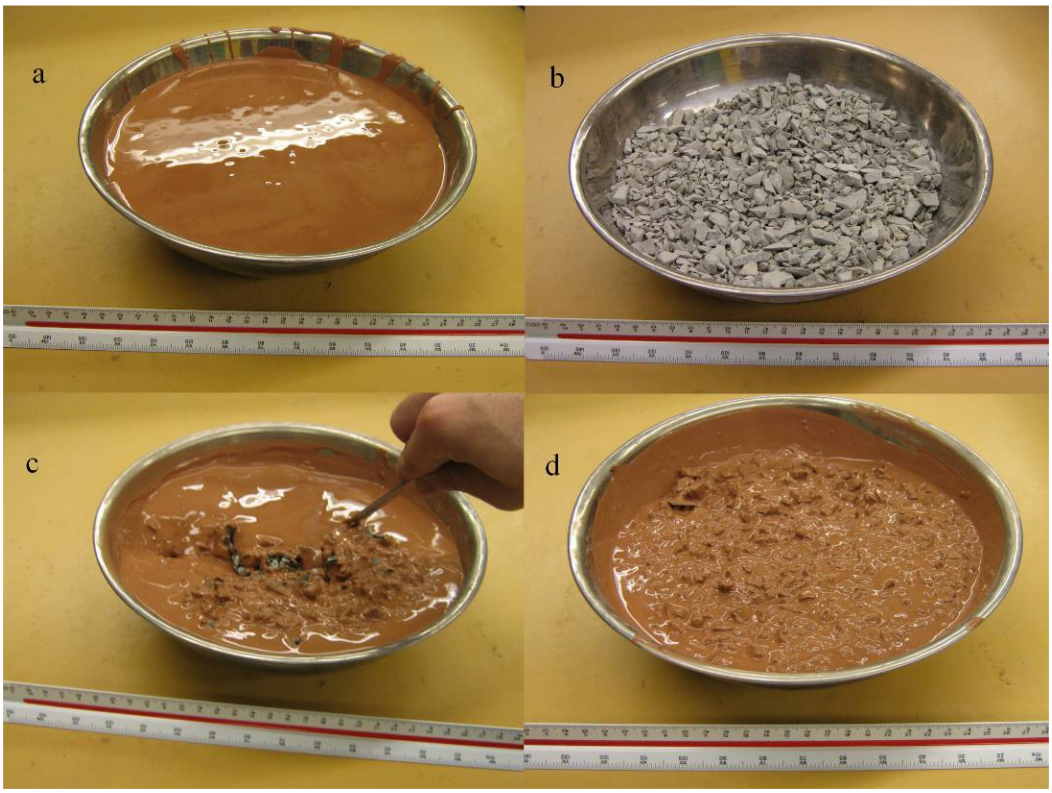


Figure 4.2. Pictures taken during preparation of slurry medium and paste rock: (a) Tailings slurry medium at a specific water content; (b) Scalped waste rock particles (< 9.4 mm); (c) Mixing rock and tailings using a bent tablespoon; and (d) Uniformly mixed rock and tailings (paste rock).

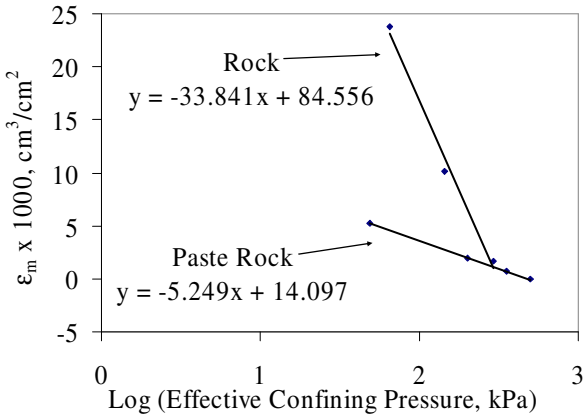


Figure 4.3. Membrane compliance factor(ε_m) for paste rock and rock-only specimens obtained from hydrostatic unloading tests.

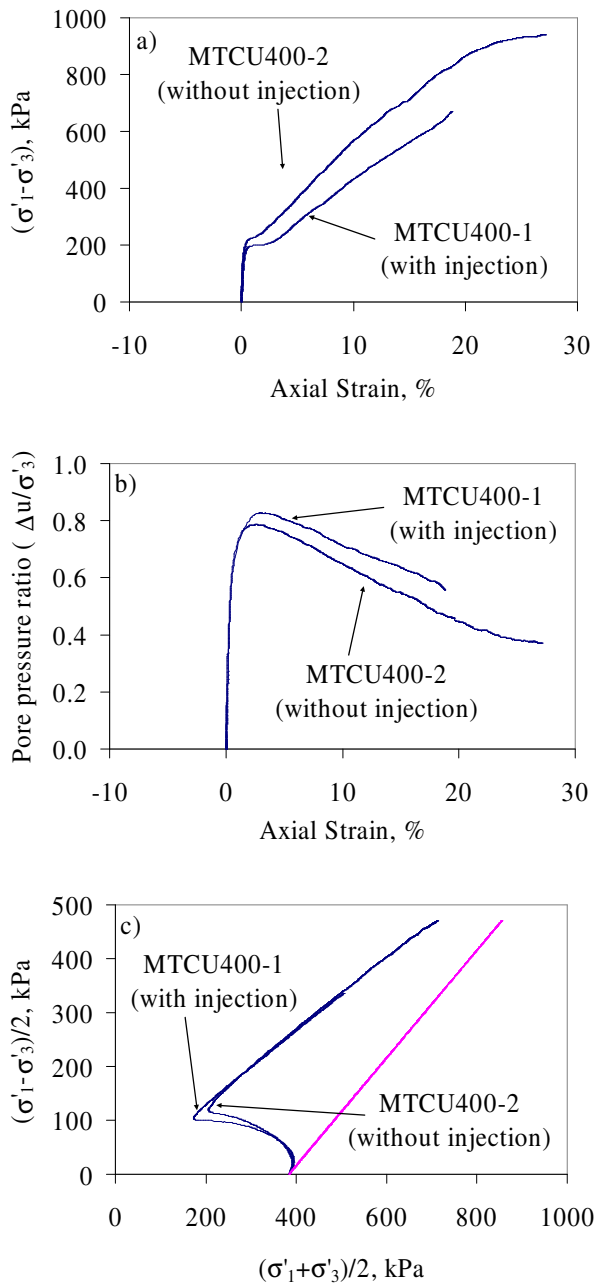


Figure 4.4. Undrained behaviour of two sets of identical specimens with and without fluid injection to correct for membrane compliance: (a) Deviatoric Stress vs. Axial strain; (b) excess pore water pressure response vs. axial strain; (c) effective stress paths.

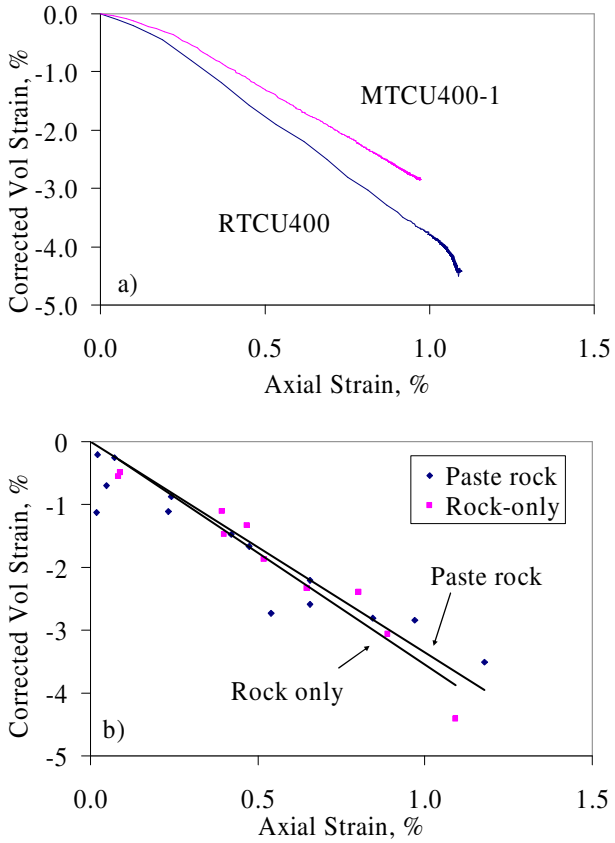


Figure 4.5. Volumetric strain vs. axial strain based on observations made at the end of initial hydrostatic consolidation phase undertaken prior to different shear tests on paste rock and rock-only specimens.

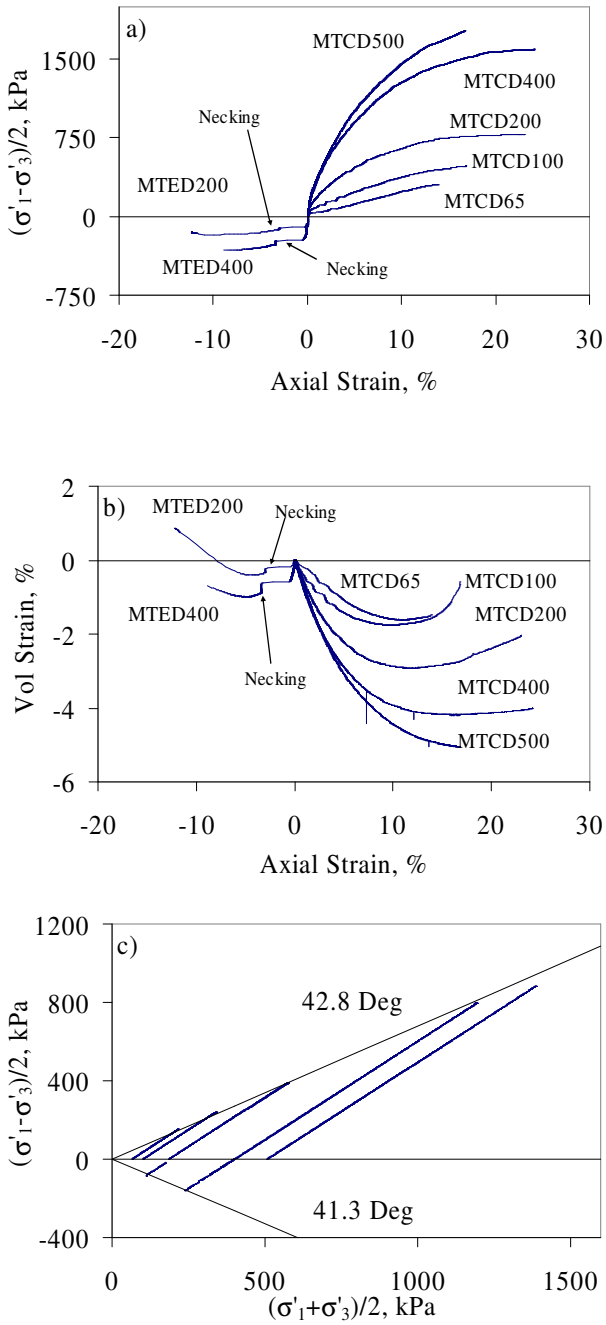


Figure 4.6. Drained monotonic shear response of mixtures of rock and tailings during triaxial compression and triaxial extension tests: (a) deviator stress vs. axial strain; (b) volumetric strain vs. axial strain; and (c) effective stress path.

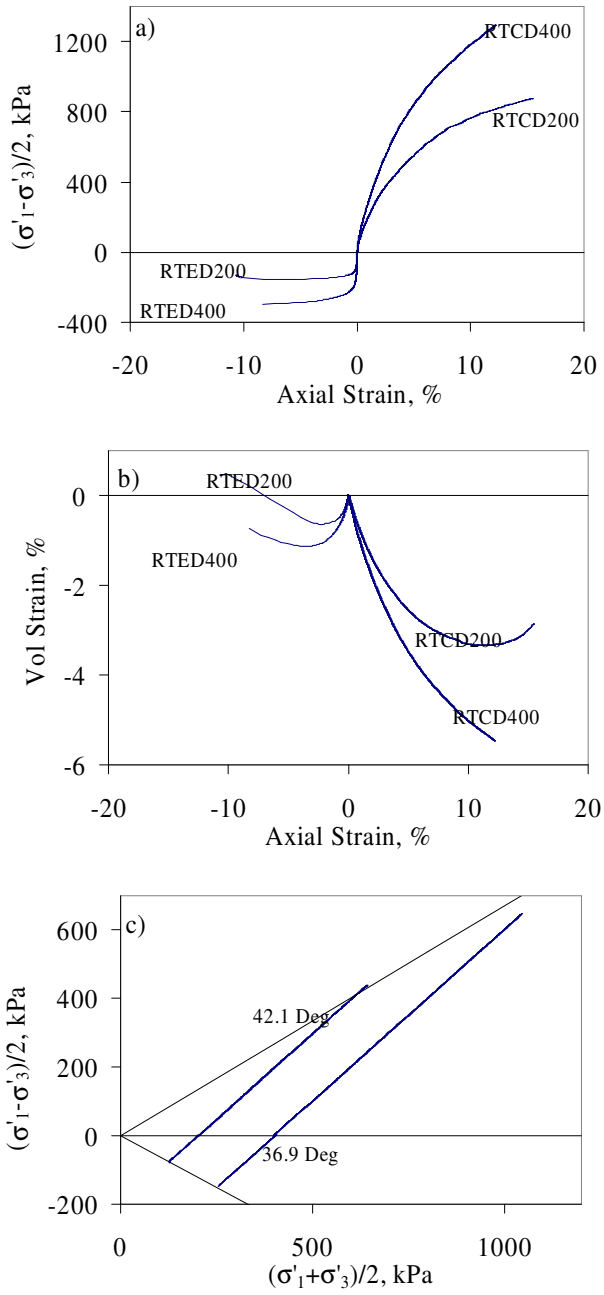


Figure 4.7. Drained monotonic shear response of rock-only material during triaxial compression and triaxial extension tests: (a) deviator stress vs. axial strain; (b) volumetric strain vs. axial strain; and (c) effective stress path.

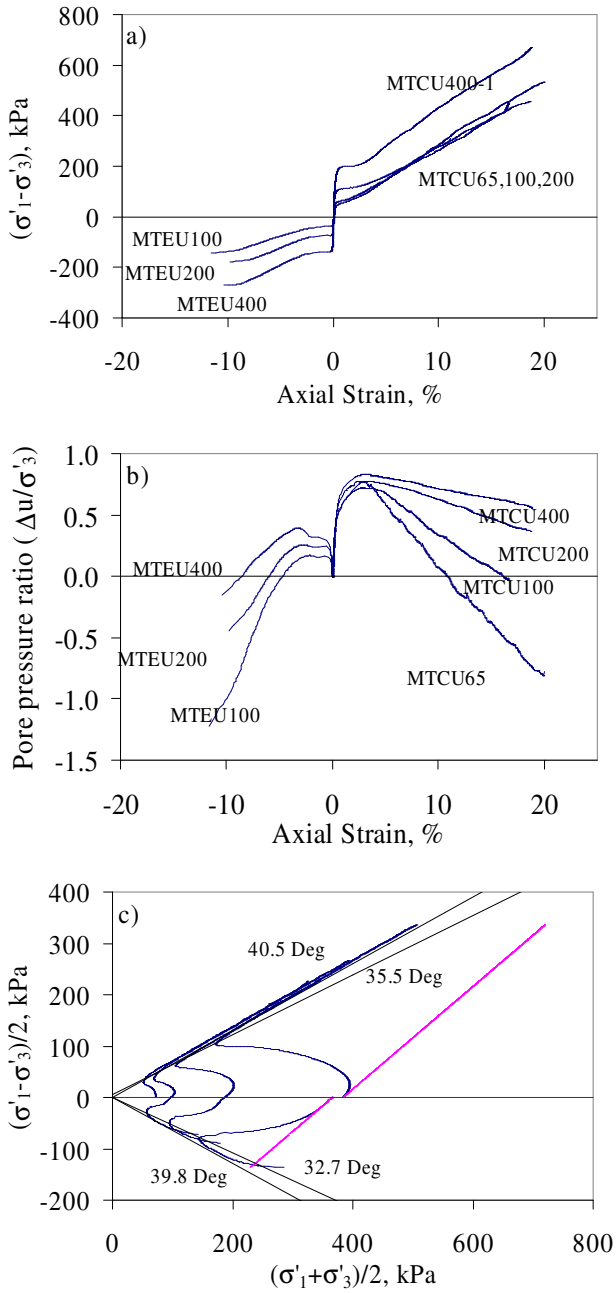


Figure 4.8. Undrained monotonic shear response of paste rock during triaxial compression and triaxial extension tests: (a) deviator stress vs. axial strain; (b) excess pore water pressure ratio vs. axial strain; and (c) effective stress path.

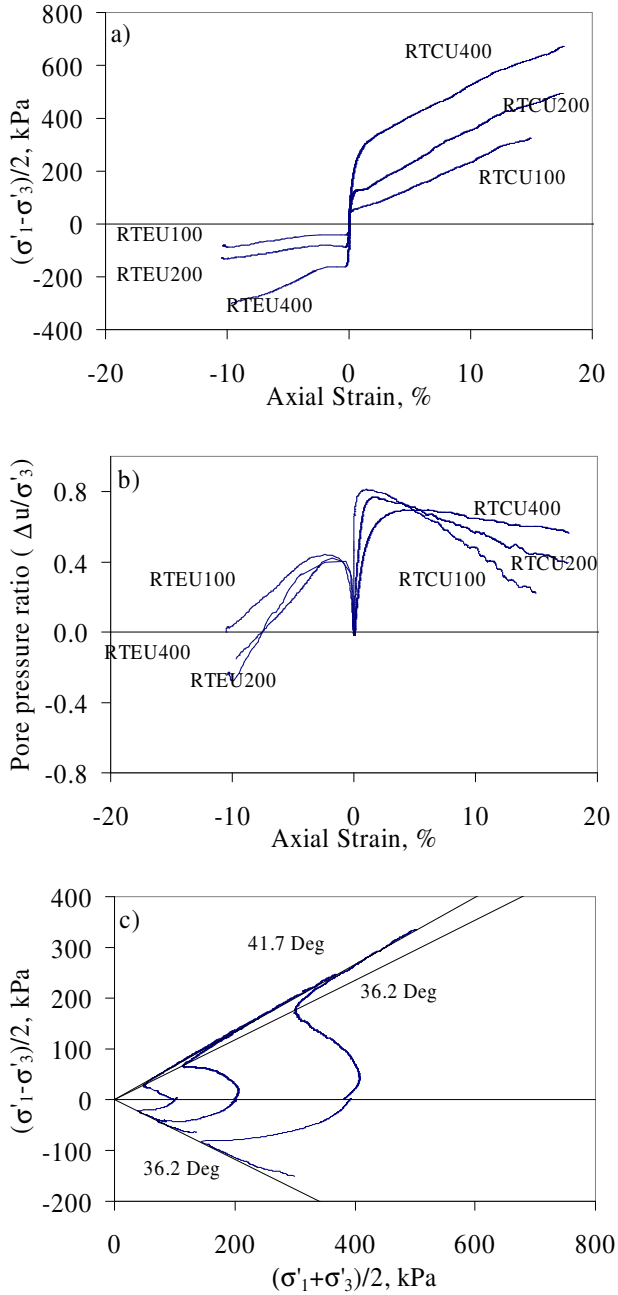


Figure 4.9. Undrained monotonic shear response of rock-only material during triaxial compression and triaxial extension tests: (a) deviator stress vs. axial strain; (b) excess pore water pressure ratio vs. axial strain; and (c) effective stress path.

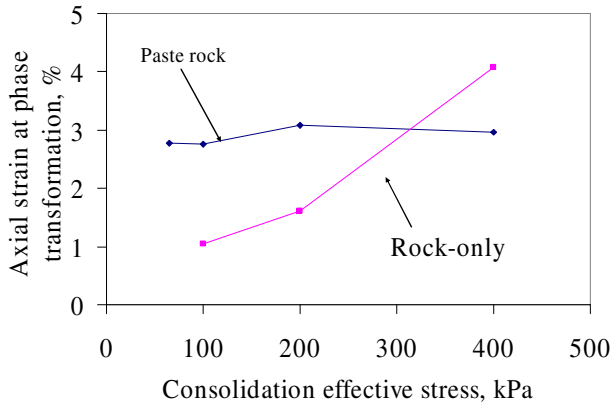


Figure 4.10. Axial strain at phase transformation vs. consolidation stress.

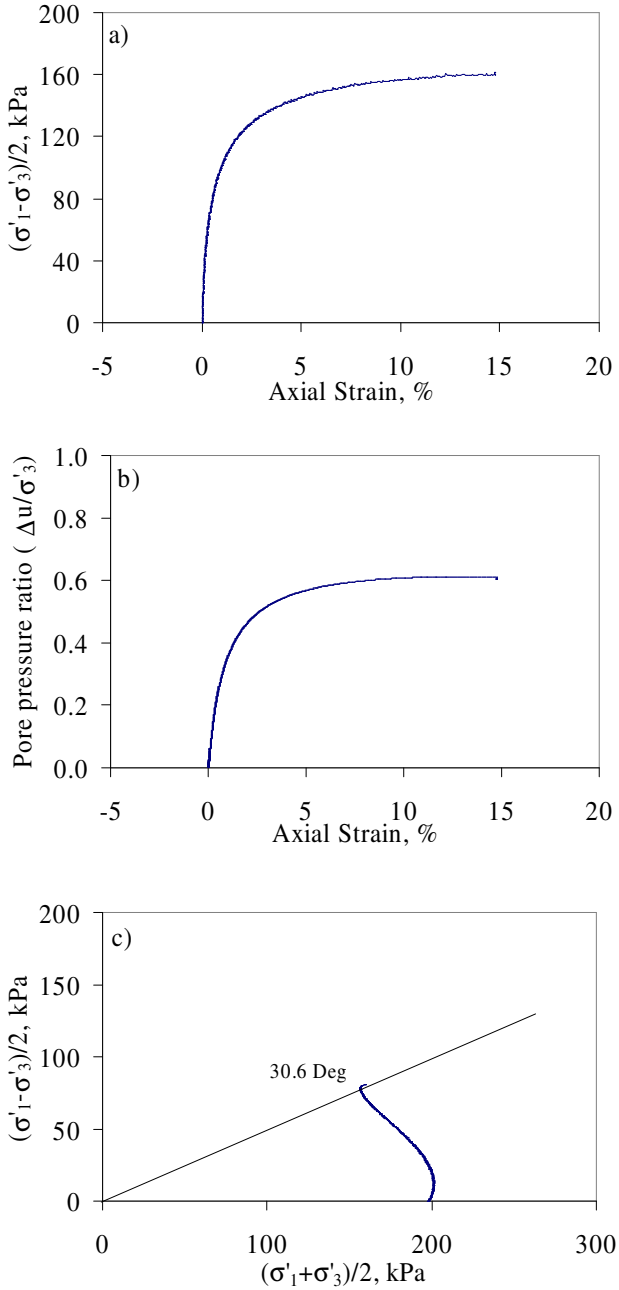


Figure 4.11. Undrained monotonic shear response of tailings-only material during triaxial compression testing: (a) deviator stress vs. axial strain; (b) excess pore water pressure ratio vs. axial strain; and (c) effective stress path.

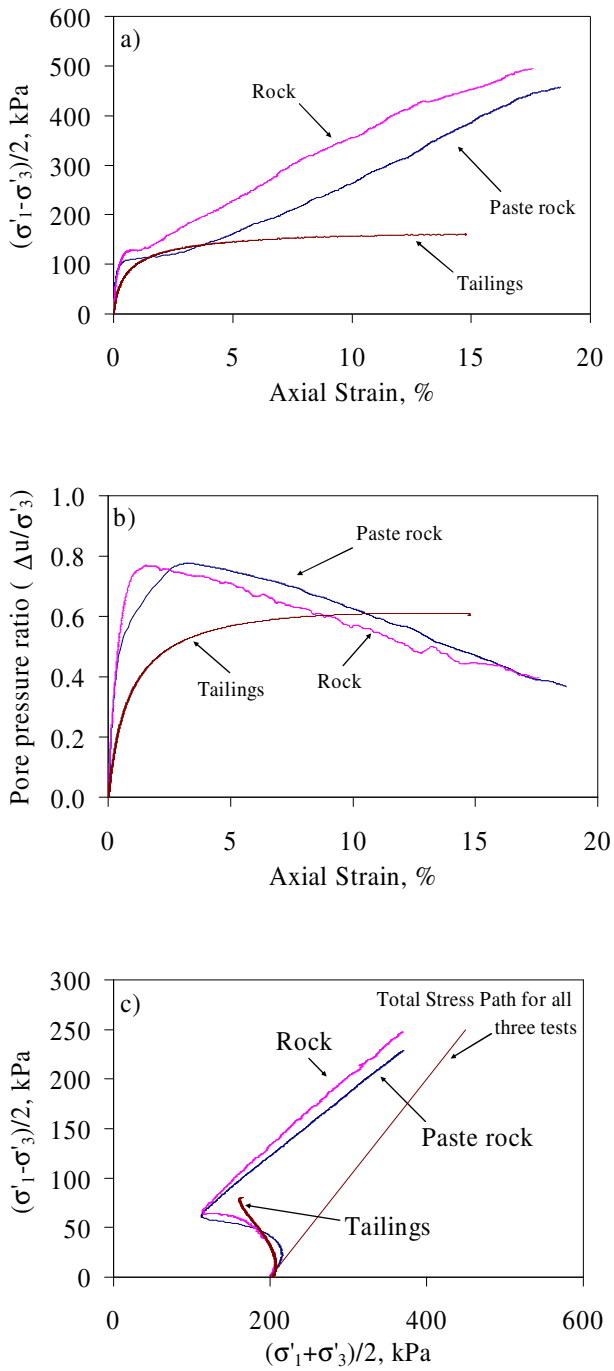


Figure 4.12. Comparison of typical shear response of paste rock, rock-only, and tailings-only materials at consolidation effective stress of 200 kPa: (a) deviator stress vs. axial strain; (b) excess pore water pressure ratio vs. axial strain; and (c) effective stress path.

4.6. References

ASTM. 2006. Standard D 4318-05. In Annual Book of ASTM Standards. *Edited by* ASTM International, West Conshohocken, PA.

ASTM. 2006. Standard test method for specific gravity of soil solids by water pycnometer (standard D854-06). In Annual Book of ASTM Standards. *Edited by* ASTM International, West Conshohocken, PA.

ASTM. 2002. Standard test method for particle-size analysis of soils (standard D422-63). In Annual Book of ASTM Standards. *Edited by* ASTM International, West Conshohocken, PA.

Bishop, A.W. and Henkel, D.J. 1962. The measurement of soil properties in the triaxial test. Edward Arnold Ltd., UK.

Donaghe, R.T. and Townsend, F.C. 1976. Scalping and replacement effects on the compaction characteristics of earth-rock mixtures. *In* Soil specimen preparation for laboratory testing: ASTM Special Technical Publication 599 *Edited by* J.B. Wheeler, H.M. Hoersch, C.E. DeFranco and E.J. McGlinchy. American Society for Testing and Materials, Philadelphia, US., pp. 248-277.

Fannin, R.J. and Moffat, R. 2006. Observations on internal stability of cohesionless soils, Geotechnique, **56**: 497-500.

Fines, P., Wilson, G.W., Landreault, D., Langetine, L., and Hulett, L. 2003. Co-mixing of tailings, waste rock and slag to produce barrier cover systems. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, pp. 14-17.

Furnas, C.C. 1928. Relations between specific volume voids and size composition in systems of broken solids of mixed sizes. US Bureau of Mines.

Holtz, W.G. and Ellis, W. 1961. Triaxial shear characteristics of clayey gravel soils. *In* Proceedings of the fifth international conference on soil mechanics and foundation engineering, Paris, Vol. 1, pp. 143-149.

Jamiolkowski, M., Kongsukprasert, L., and Lo Presti, D. 2005. Characterization of gravelly geomaterials. *In* Proceedings of The Fifth International Geotechnical Conference, Cairo, Egypt, pp. 1-27.

Junqueira, F., Rykaart, M., Weeks, B., Wickland, B., and and Wilson, G.W. 2007. Emerging technologies in mine waste management. *In* Proceedings of the Professional Development Seminar on Emerging Technologies in Mine Waste Management, Vancouver, BC.

Khalili, A. and Wijewickreme, D. 2008. Method for reconstitution of highly gap-graded specimens for laboratory element testing, ASTM Geotechnical Testing Journal, **31:** 424-432.

Khalili, A., Wijewickreme, D., and Wilson, G.W. 2007. Consolidation response of mixtures of waste rock and tailings with different particle size of coarse fraction. *In* Proceedings of 60th Canadian Geotechnical Conference, Ottawa, Canada.

Khalili, A., Wijewickreme, D., and Wilson, G.W. 2005. Some observations on the mechanical response of mixtures of mine waste and tailings. *In* Proceedings of 58th Canadian Geotechnical Conference - GeoSask2005, Saskatoon, SA.

Kuerbis, R., Negussey, D., and Vaid, Y.P. 1988. Effect of gradation and fines content on the undrained response of sand. *In* Proceedings of Hydraulic Fill Structures, pp. 330-345.

Lade, P.V. and Yamamuro, J.A. 1997. Effects of nonplastic fines on static liquefaction of sands, Canadian Geotechnical Journal, **34**: 918-928.

Lee, K.L. and Seed, H.B. 1967. Dynamic strength of anisotropically consolidated sand, Journal of the Soil Mechanics and Foundations Division, ASCE, **93**: 169-190.

Leroueil, S. and Hight, D.W. 2003. Behaviour and properties of natural soils and soft rocks. *In* Characterization and engineering properties of natural soils *Edited by* T.S.

Tan, K.K. Phoon, D.W. Hight and S. Leroueil. A.A. Balkema Publishers, Rotterdam, pp. 29-254.

Muir Wood, D. 1991. Soil behaviour and critical state soil mechanics. Cambridge University Press.

Poulos, S.J., Robinsky, E.I., and Keller, T.O. 1985. LIQUEFACTION RESISTANCE OF THICKENED TAILINGS. Journal of Geotechnical Engineering, **111:** 1380-1394.

Rowe, P.W. 1962. The stress-dilatancy relation for static equilibrium of an assembly of particles in contact. *In* Proceedings of the Royal Society of London.Series A, Mathematical and Physical SciencesJSTOR, pp. 500-527.

Sanin, M. and Wijewickreme, D. 2006. Influence of initial confining stress on the mechanical response of natural fraser river delta silt. *In* Proceedings of the 59 th Canadian Geotechnical Conference, Vancouver, BC.

Siddiqi, F.H., Seed, R.B., Chan, C.K., Seed, H.B., and Pyke, R.M. 1987. Strength evaluation of coarse-grained soils. UCB/EERC-87/22, Earthquake Engineering Research Center - University of California Berkley, California, USA.

Sivathayalan, S. and Vaid, Y.P. 1998. Truly undrained response of granular soils with no membrane-penetration effects, Canadian Geotechnical Journal, **35**: 730-739.

Thevanayagam, S. 1998. Effect of fines and confining stress on undrained shear strength of silty sands, Journal of Geotechnical and Geoenvironmental Engineering, **124**: 479-491.

Vaid, Y.P. and Chern, J.C. 1985. Cyclic and monotonic undrained response of saturated sands. *In* Advances in the Art of Testing Soils Under Cyclic Conditions: Proceedings of a session held in conjunction with the ASCE Convention., pp. 120-147.

Vaid, Y.P. and Negussey, D. 1984. A critical assessment of membrane penetration in the triaxial test, ASTM Geotechnical Testing Journal, **7**: 70-76.

Wagg, B.T. and Konrad, J.M. 1990. Index properties of clay silt mixtures. *In* Proceedings of the 43rd Canadian Geotechnical Conference. Part 2, Canada, pp. 705-710.

Wijewickreme, D., Sanin, M.V., and Greenaway, G.R. 2005. Cyclic shear response of fine-grained mine tailings. Canadian Geotechnical Journal, **42:** 1408-1421. Vallejo, L.E. 2001. Interpretation of the limits in shear strength in binary granular mixtures. Canadian Geotechnical Journal, **38**: 1097-1104.

Wilson, W.G. 2001. Co-disposal of tailings and waste rock, Geotechnical News, **19**: 44-49.

Wickland, B.E. 2006. Volume change and permeability of mixtures of waste rock and fine tailings. Ph.D., University of British Columbia, Canada.

Wickland, B.E., Wilson, G.W., Wijewickreme, D., and Klein, B. 2006. Design and evaluation of mixtures of mine waste rock and tailings, Canadian Geotechnical Journal, **43**: 928-945.

Williams, D.J., Wilson, G.W., and Panidis, C. 2003. Waste rock and tailings mixtures as a possible seal for potentially acid forming waste rock. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, pp. 14-17.

Williams, D.J., Gowan, M., and Keefer, P. 1995. Practical co-disposal deposition. *In* Proceedings of the 7th Australian Coal Preparation Conference, Mudgee, New South Wales, pp. 9-15.

Wilson, G.W., Plewes, H.D., Williams, D., and Robertson, J. 2003. Concepts for co-mixing of tailings and waste rock. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, pp. 14–17.

Chapter 5.

Cyclic Shear Response of Highly Gap-graded Mixtures of Waste Rock and Tailings [i]

With particular reference to "paste rock", research has been undertaken to develop methods for mixing of waste rock and tailings (Wilson 2001; Wilson et al. 2003; Fines et al. 2003; Williams et al. 2003) and determination of optimum mix ratios of waste rock and tailings (Williams et al. 1995; Wickland et al. 2006). Williams et al. (1995) concluded that the ideal mix ratio for paste rock is the one which causes the voids of the coarser fraction to be "just filled" with tailings particles while the coarser particles are still in contact with each other. Wickland (2006) confirmed that "just filling" the waste

[i] A version of this paper has been submitted for publication. Wijewickreme, D., Khalili, A. and Wilson, G. W. Cyclic Shear Response of Highly Gap-graded Mixtures of Waste Rock and Tailings.

rock particle matrix with tailings would result in a mixture with low compressibility (close to that of the waste rock matrix) while giving rise to the maximum tailings storage and reduced potential for ARD. Wickland et al. (2006) also found that compressibility of paste rock was constrained by the load bearing 'waste rock skeleton.' It was concluded that paste rock can offer the hydraulic conductivity of tailings alone and the compressibility characteristics of waste rock alone.

In spite of the above research work, and except for the studies on sand/silt mixtures, no detailed studies have been undertaken to understand the shear response of gap-graded mixtures of waste rock and tailings. In particular, many mine sites are located in seismically active areas. As such, in addition to the monotonic shear response, understanding of the cyclic response of paste rock is of great importance in the design of engineered dumps and/or barriers. Geotechnical laboratory element testing plays a key role in predicting the behaviour of the paste rock.

With this background, a laboratory triaxial element testing program was undertaken at the University of British Columbia (UBC) to study the mechanical behaviour of paste rock. The research program was developed primarily from the point of view of understanding the stress-strain response (stiffness and strength) and the development of shear-induced volume changes and/or excess pore water pressures under static/cyclic shear loading.

The observed performance of re-constituted paste rock for the "static" (monotonic) loading component is presented and discussed in the companion paper by the authors (Khalili et al. 2008) which forms the stand alone chapter 4 in this document. Laboratory findings related to the cyclic shear response, which is an important consideration in the use of paste rock in seismically active regions, forms the scope of research presented herein. The results pertaining to the cyclic loading response of paste rock and waste rock alone (hereinafter referred to as rock-only), along with those derived from limited testing work on tailings alone (hereinafter referred to as tailings-only), are presented. Some observations made during post-cyclic shearing of the specimens of paste rock and rock-only materials are also discussed.

5.1. Experimental Aspects

5.1.1 Material tested

The test material used in this study is identical to that used in the counterpart research (Khalili et al. 2008) on the monotonic response of tailings/waste-rock mixtures. The raw material used for the mixture includes blasted waste rock and Carbon in Pulp (CIP) tailings from a gold extraction process at the Porgera Gold Mine, Enga Province, Papua New Guinea.

The waste rock has an altered sedimentary origin and, as expected from a mining process, the particles are very angular in nature, with sharp edges. The specific gravity of rock particles was determined to be 2.7 (ASTM Standard D854-06). In order to obtain a gradation suitable for the triaxial shear testing herein, particles of waste rock larger than 9.4 mm were removed using a Tyler standard sieve with 9.42 mm openings (ASTM Standard D422). This resulted in a ratio of specimen diameter to maximum grain size of greater than 8 for the 76-mm diameter triaxial test specimens used in this study. Jamiolkowski (2005) has suggested that a minimum ratio of specimen size to maximum grain size of 5 is necessary to eliminate particle size effects during laboratory element testing, with any ratio equal or greater than 8 being ideal. Minimum and maximum void ratios of waste rock after scalping were determined to be 0.5 and 0.9 respectively (using ASTM Standards D4253 and D4254).

The CIP tailings material consisted of primarily silt and clay size particles, with more than 90% of the particles less than 75 μm in size. The tailings originate from an extraction process that involves autoclaving to oxidize sulphide minerals. The material had been treated with calcium hypochlorite to destroy cyanide and yield a chemically inert material for the purpose of this study. Specific gravity of tailings particles was found to be 2.93 using ASTM Standard D854-06. The material has a red-brown colour with a plasticity index of 12% (ASTM D4318-05).

Particle size distribution curves of crushed rock and tailings are represented in Figure 5.1. A mixture ratio of around 4.8:1 waste rock to tailings was previously shown to provide the maximum density for the material with tailings particles just filling the voids among rock grains (Wickland et al. 2006), and this ratio, therefore, was used for the reconstitution of test specimens. The same mixture ratio was used in a counterpart study at UBC to examine the compressibility and air-entry characteristics of paste rock (Wickland et al. 2006).

5.1.2 Specimen preparation and laboratory testing considerations

The commonly available specimen reconstitution methods (i.e., water pluviation, air pluviation and moist tamping) are not suitable to prepare uniform/homogeneous specimens of waste rock and tailings mixtures in a saturated condition. In recognition of this need, a new "slurry displacement" method was developed for preparing saturated, uniform, and repeatable specimens of highly gap-graded paste rock as detailed in Khalili and Wijewickreme (2008) and the companion paper (Khalili et al. 2008). The method is capable of forming specimens with very good uniformity and degree of saturation.

A computer-controlled cyclic triaxial testing device at UBC (capable of testing specimens with dimensions of 76 mm in diameter and approximately 160 mm in height) was employed for the testing program. Although the direct simple shear (DSS) device is considered to simulate the seismic loading modes prevalent in the field (Finn et al. 1978),

the cyclic triaxial shear device was considered suitable for this study because of its ability to accommodate relatively larger particles in the paste rock material, in comparison to testing in a DSS device where the specimen is relatively smaller (70 mm diameter and ~20 mm height NGI-type DSS tests). All tests were conducted using stress-controlled loading with double drainage (top and bottom) during the hydrostatic consolidation phase except for those tests performed on tailings-only specimens. For tailings-only specimens, single side drainage was used initiating from the bottom of the specimen. Tests were undertaken with appropriate "real-time" corrections applied with respect to membrane compliance using a fluid injection system (Khalili et al. 2008; Khalili and Wijewickreme 2008). Real-time fluid injections were not undertaken during undrained shearing after reaching excess pore pressure ratios ($\Delta u/\sigma'_{vc}$) levels of about 0.8; this was due to the practical limitations of the test apparatus capability to meet significantly large-volume pore water injection requirements that arise at high ($\Delta u/\sigma'_{vc}$) > 0.8 levels as a result of low effective stresses within the specimen.

The loading time-rates/frequencies were chosen based on the commonly used guidelines for equalization of shear-induced pore pressure for undrained triaxial testing (Bishop and Henkel 1962) by making use of coefficient of consolidation (C_v) values obtained during hydrostatic consolidation part of the triaxial testing. Care was taken to account for at least 90% equalization in shear-induced pore pressure within specimens. The time spans required for reaching equilibrium in pore water pressure readings during B-value determinations were also useful in assessing the suitability of the selected time-rates of

shearing. Cyclic loading of paste rock specimens was conducted in a stress-controlled manner at a frequency of 0.002 Hz (or 8.33 min/cycle). This consisted of applying a deviator stress ($\sigma_{d,cyc}$) in a symmetrical sinusoidal manner at constant cyclic stress ratio (CSR = $\sigma_{d,cyc}/2\sigma'_{3c}$) amplitude, where σ'_{3c} is initial effective confining stress. In spite of the relatively high coefficient of compressibility (C_v) for rock-only specimens, in order to be consistent with the tests on paste rock, cyclic loading of rock-only specimens were also conducted using a frequency of 0.002 Hz. It is to be noted that despite tailings material controlling the hydraulic conductivity of the paste rock, the coefficient of consolidation (C_v) of tailings-only material is much less compared to paste rock. Since criteria for equalization of pore pressures within specimens is dependent on C_v (and not , hydraulic conductivity), it was decided to conduct cyclic loading of tailings-only specimens at a frequency of 0.000139 Hz (or about 120 min/cycle).

A stress-controlled loading rate of about 480 kPa/hour were used for the limited post-cyclic monotonic loading tests that were undertaken on selected paste rock and rock-only specimens.

5.2. Test Program

Since earthquake loading would generally involve configurations of level-ground as well as slopes, the undrained cyclic shear response was examined for cases with no initial static shear stress (i.e., simulating level-ground) and those with initial static shear stress

bias (i.e., simulating sloping ground). A total of 42 cyclic triaxial tests were undertaken consisting of 23 tests on paste rock, 16 tests on rock-only specimens and 3 tests on tailings-only specimens (see Section 4.1 for underlying reasons for limiting the number tests on tailings-only material). The test program including parameters pertaining to each of the tests is shown in Tables 5.1 and 5.2.

As may be noted from Tables 5.1 and 5.2, a series of undrained cyclic triaxial tests performed on reconstituted paste rock forms the key component of the test program. In addition, another series of tests were undertaken on reconstituted rock-only specimens. Wickland et al. (2006) noted that the 'waste rock skeleton' has a major role to play in governing the compressibility of mixtures; as such, a comparison of the rock-only material response with that of paste rock was considered prudent. All tests were conducted to explore the cyclic response of specimens hydrostatically consolidated to initial effective confining stresses (σ'_{3c}) approximately between 100 and 400 kPa, with the exception of one additional test performed at σ'_{3c} = 64 kPa only on a paste rock specimen. In addition to the tests on paste rock and rock-only specimens, limited triaxial testing was also performed on tailings-only material using specimens cut from a 300-mm diameter cylindrical block sample of tailings that was prepared in advance; the block sample was reconstituted by slurry consolidation of tailings to a stress level slightly lower than that used for the triaxial test program.

Cyclic shear testing was undertaken with constant CSR (= $\sigma_{d,cyc}/2\sigma'_{3c}$) amplitudes ranging between 0.1 and 0.25 (Note: $\sigma_{d,\ cyc}$ = cyclic deviator stress, and $\sigma_d = \sigma'_1 - \sigma'_3$ = deviator stress). For those tests with a static shear stress bias, two different static shear bias levels [$(\sigma_{d,stat})/(\sigma'_{1c} + \sigma'_{3c})$] of ~0.1, and ~0.18 were employed, where, $\sigma_{d,stat}$ = static deviator stress used to consolidate the specimens prior to cyclic loading.

The as-placed saturated density, $\rho_{(sat)}$, of paste rock specimens used in this test program ranged between 2.102 and 2.199 g/cm^3, with waste rock to tailings ratios varying between 4.57 and 4.97. This density range was specifically chosen since it represented a condition close to the tailings fraction "just filling" the voids in the rock particle matrix, as determined by Wickland et al. (2006) and Khalili et al. (2007) for this mine waste type. The rock-only specimens had as-placed $\rho_{(sat)}$ ranging between 1.877 and 1.973 g/cm^3 as summarized in Table 5.2. The rock skeleton void ratios (e_{Rock}) calculated (knowing the bulk density, mixture ratio, and specific gravity of materials) for paste rock specimens are also given in Table 5.2. As may be noted, rock skeleton void ratios are comparable to the void ratios of rock-only specimens.

Post-cyclic undrained shear strength $S_{u(LIQ)}$ of potentially liquefiable soil zones is a key parameter that is required for seismic slope stability analysis. Seed and Harder (1990), Stark and Mesri (1992), and Olson and Stark (2002) have developed correlations between liquefied strength $S_{u(LIQ)}$, or liquefied strength ratio $S_{u(LIQ)}/\sigma'_{vc}$ (where σ'_{vc} is initial vertical overburden effective stress) computed by back-analyses of field case histories,

and in-situ standard penetration values. Such back-analysis for estimation of $S_{u(LIQ)}$ has been considered more suitable, since laboratory testing is not able to simulate the void redistributions, or water film effects, that take place after liquefaction, particularly in layered deposits with contrasting permeability (Kokusho 2003). Nevertheless, the evaluation of laboratory data provides important information for understanding the soil response in a fundamental manner as well as to support and confirm field-based approaches. With this background, to investigate the post-cyclic stress-strain response and strength aspects, several post-cyclic monotonic loading tests (as indicated in Tables 5.1 and 5.2) were also conducted on selected paste rock and rock-only specimens after subjecting them to a pre-defined level of cyclic straining.

5.3. Test Results and Discussion

5.3.1 Cyclic shear response with no initial static shear stress

Typical stress-strain response [$(\sigma'_1-\sigma'_3)$ vs. ε_a] as well as stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$], excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles, and axial strain (ε_a) vs. number of cycles obtained from three paste rock specimens initially hydrostatically consolidated to a stress (σ'_{3c}) of 200 kPa and then subjected to cyclic shear loading with constant cyclic stress ratio (CSR) amplitudes of 0.075, 0.15, and 0.25 (Tests No. MCT200-2, MCT-200-1, and MCT200-3, respectively) are presented in Figures 5.2 through 5.4. For a given specimen, the void ratio of rock skeleton after initial

consolidation phase - which is also the void ratio during cyclic loading - has been denoted by e_{Rock} in the figures.

It may be noted that initially all three specimens display excess pore water pressure (Δu) accumulation in a progressive manner with increasing number of loading cycles, with significant excess pore water pressure generation during the first half-cycle of loading. Moreover, except for the specimen that was subjected to a CSR of 0.075, all the specimens eventually experienced relatively low transient vertical effective stress conditions [i.e., $r_u = (\Delta u/\sigma'_{3c})$ approaching unity], during cyclic loading. This increasing r_u, or movement of effective stress path towards the origin of the plot, in essence increases the mobilized effective stress ratio ($R = \sigma'_1/\sigma'_3$) or the mobilized friction angle. Examination of the data reveals that specimens MCT200-1 and MCT200-3 accumulated significantly large strains after arriving at these high effective stress ratio levels (see Figures 5.3 through 5.4). It is also notable that there was more strain development when the specimen was in the extension mode of the loading cycle (i.e., negative ε_a) than during the compression mode.

In an overall sense, all three specimens exhibited "cyclic mobility type" strain development mechanism. This has been well observed and documented during laboratory research on the undrained cyclic shear response of dense sands (Wijewickreme et al. 2005a; Kammerer et al. 2002; Vaid and Sivathayalan 1998; Vaid and Sivathayalan 1999), natural fine-grained soils (e.g., Zergoun and Vaid 1994; Sanin and Wijewickreme 2006;

Boulanger and Idriss 2006; Bray and Sancio 2006), and fine-grained mine tailings (Wijewickreme et al. 2005b). Clearly, liquefaction in the form of strain softening accompanied by loss of shear strength did not manifest itself in paste rock, regardless of the applied CSR value, or the level of r_u. In spite of this, it is fair to state that there is an overall reduction of shear modulus with the development of excess pore water pressure which is an important consideration from an engineering design/performance point of view.

Typical mechanical response observed from cyclic shear testing of a specimen of rock-only material is presented in Figure 5.5 (presented in a format similar to that used for paste rock). In this Test (No. RCT200-1), the rock-only specimen was initially hydrostatically consolidated to a stress (σ'_{3c}) of 205 kPa, and then it was subjected to cyclic shear loading with constant cyclic stress ratio (CSR) amplitude of 0.16. As noted, the void ratio of the rock skeleton (e_{cR}) of this rock-only specimen (after hydrostatic consolidation) is reasonably close to the corresponding e_{Rock} values of the paste rock skeleton (0.754 for rock-only specimen vs. 0.749 for paste rock specimen). As explained earlier, pore water injection for compensating the membrane penetration effect was terminated after a pore pressure ratio of about 0.8 was reached.

In an overall context, the observed response for the rock-only specimen is similar to those observed for paste rock. For example, accumulation of Δu in a progressive manner with increasing number of loading cycles, with r_u approaching unity with increasing number of

cycles can be noted. Again, larger strain development occurred after arriving at a certain effective stress ratio (typically between 0.70 and 0.85) with more strain development in the extension mode in comparison to the compression mode. No strain softening, accompanied by loss of shear strength, was observed during the loading process. In spite of these general similarities, it is important to note that significantly larger number of cycles of loading were required to cause significant strain development in the rock-only specimen when compared with that required for the similarly loaded paste rock specimen MCT200-1 (σ'_{3c} = 198 kPa subjected to CSR = 0.15); this aspect is further examined in the next section titled "Cyclic shear resistance". Although there was no strain softening and no loss of shear strength, once the rock-only specimen had reached high r_u levels, significant axial strain development in the extensional mode was noted during subsequent cycles ($\Delta\varepsilon_a$ of over 3% per cycle as shown in Figure 5.5). Although not shown from the point of view of brevity, the cyclic response observed from the testing of other specimens of rock-only material (as per Tables 5.1 and 5.2) were generally similar in form to that presented in Figure 5.5 for Test No. RCT200-1. All cyclic test results are presented in Appendix 1.

The mechanical response observed during cyclic shear testing of a tailings-only specimen (Test No. TCT200-1, σ'_{3c} = 200 kPa subjected to CSR = 0.18) is presented in Figure 5.6 for comparison with the behaviour of paste rock material. The tailings specimen also displayed contractive response during both loading and unloading parts of the first half-cycle of loading, with r_u increasing further with increasing number of loading cycles.

Once again, "cyclic mobility type" strain development mechanism is observed with no liquefaction in the form of strain softening accompanied by loss of shear strength. The observed response is well in accord with those observed by Sanin and Wijewickreme (2006) and Bray and Sancio (2007) for natural fine-grained soils and Wijewickreme et al. (2005b) for fine-grained mine tailings. Strain development during cyclic loading in the tailings-only specimen was significantly more gradual compared to the paste rock and rock-only specimens.

Wickland et al. (2006) noted that, in terms of compressibility, paste rock behaves more like waste rock alone, rather than tailings alone. Similar observations have been made in chapter 4 as well as Khalili et al. (2008), based on data from monotonic triaxial shear tests conducted on paste rock, rock-only, and tailings-only specimens. The above observations during undrained cyclic loading is very much in accord with this notion, further confirming the role of the 'waste rock skeleton' in controlling the stress-strain-shear strength response of paste rock.

5.3.1.1 Cyclic shear resistance

It is of interest to examine the cyclic shear resistance of paste rock by comparing the response observed from different tests under different applied CSR levels. For this purpose, the number of load cycles required to reach a single-amplitude axial strain ε_a = 2.5%, in a given undrained triaxial test under a given applied CSR, was defined as $N_{2.5\%}$.

An ε_a = 2.5% has been previously used as a deformation criterion to assess the cyclic shear resistance of sands by the U.S. National Research Council (NRC 1985), and it also has been adopted in many previous liquefaction studies at UBC. An arbitrarily selected strain level is not necessarily an appropriate measure of "liquefaction"; however, the author believes that it is reasonable to consider the development of $\varepsilon_a \geq 2.5\%$ as an indicator of unacceptable performance in a triaxial specimen, primarily from the point of view of comparing cyclic resistance between tests.

The variation of cyclic resistance ratio (CRR) vs. $N_{2.5\%}$ related to data from all the cyclic triaxial tests (with zero static shear stress bias) conducted on paste rock is presented in Figure 5.7. As described in Youd et al. (2001), the value of CSR in this interpretation is called CRR since it represents the capacity of the soil to resist cyclic loading. As may be noted one test did not reach 2.5% of axial strain even after 50 cycles of loading. With allowance made for experimental scatter, it can be argued that the CRR vs. $N_{2.5\%}$ relationship for all the tests seems to follow a common trend line (see Figure 5.7). It appears that the CRR of tested paste rock does not appear to be significantly sensitive to the overburden stress for the tested σ'_{3c} levels between 100 kPa and 400 kPa. Similar to that for paste rock, the variation of cyclic resistance ratio (CRR) with respect to $N_{2.5\%}$ derived from the testing of rock-only specimens is presented in Figure 5.8. Again two of the tests did not reach 2.5% of axial strain even after 50 cycles of loading. Similar to paste rock, CRR appears to be not sensitive to the σ'_{3c} levels between 100 kPa and 400 kPa. In the evaluation of liquefaction susceptibility of sands, the effect of confining

pressure on CRR is accounted by the use of empirical factors K_σ (Seed and Harder 1990), based on limited database from laboratory investigations. K_σ is defined as:

[1] $$(CRR)_{\sigma', Drc} = (CRR)_{100, Drc} \times K_\sigma$$

where $(CRR)_{\sigma', Drc}$ is the cyclic resistance ratio of a soil specimen of a given density D_{rc} consolidated to an initial confining stress of σ', and $(CRR)_{100,Drc}$ is the cyclic resistance ratio of a specimen of the same soil at the same density consolidated to an initial confining stress of 100 kPa. The value of K_σ has been noted to decrease with increasing confining pressure, for a given relative density. In particular, the reported information by Seed and Harder (1990), Vaid and Thomas (1995), and Vaid et al. (2001) suggests that K_σ is close to unity for the loose sand and for stress levels not exceeding about 500 kPa. The relative insensitivity of CRR to the σ'_{3c} levels (i.e., $K_\sigma \cong 1$ for $\sigma'_{3c} < 400$ kPa) observed for uncompacted (relatively loose) rock-only specimens as presented in Figure 5.8 seems to be in accord with these observations for sands. The insensitivity of CRR to the σ'_{3c} levels observed for paste rock (Figure 5.6) can also be considered reasonable since paste rock behaviour appears to be controlled by the waste rock skeleton.

The CRR vs. $N_{2.5\%}$ for the three materials (paste rock, rock-only, tailings-only), obtained from specimens initially consolidated to σ'_{3c} of 200 kPa, are compared in Figure 5.9. As may be noted, the tests on tailings-only material yielded the highest cyclic shear resistance which was then followed by rock-only material. This is generally in accord

with the previous observations that the cyclic resistance of low-plastic fine-grained soils is higher than coarse-grained sandy soils, as noted by Wijewickreme and Sanin (2004) for natural silts and Wijewickreme et al. (2005b) for mine tailings. It is of relevance to highlight that the three tests on tailings-only material in this study were conducted on specimens initially consolidated to 200 kPa. In practical situations, a long time is required for the tailings material to reach self-weight consolidation levels yielding effective stresses of 200 kPa as indicated by Wickland (2006).

As shown, paste rock seems to have the lowest cyclic shear resistance of the three materials – i.e., in relative terms, paste rock has a higher potential for strain development under a given cyclic stress ratio and number of load cycles in comparison to tailings-only and rock-only materials. The presence of tailings (a material with a low hydraulic conductivity) "just filling" the pore spaces in between rock particles appears to have decreased the ability of the rock particles to engage and develop inter-particle stresses in comparison to the case with rock-only material. Wickland et al (2006) have shown that tailings particles in paste rock has a coefficient of consolidation (C_v) in the range of 0.002 to 0.0095 cm^2/s and, thus, could remain unconsolidated for long periods of time under real-life configurations. This may be one of the reasons for the observed relatively lower CRR.

It is important to recall that "just filled" paste rock, regardless of the applied cyclic stress level, exhibited cyclic mobility - gradual strain development and not strain softening and

loss of shear strength leading to situations of catastrophic failure - under undrained cyclic loading (see Figures 5.2 through 5.4). The observed relatively higher potential for strain development indicated in Figure 5.9, therefore, should be viewed with these considerations in mind and not be overemphasized as an insurmountable drawback with respect to the performance of paste rock material from an engineering point of view.

5.3.1.2 Post-cyclic shear stress strain response

Typical post-cyclic monotonic shear response observed from testing of paste rock and rock-only specimens (Tests No. MPCT200-4 and RPCT200-3) are presented in Figures 5.10 and 11, respectively. In each test, monotonic undrained compression loading was commenced after the specimens accumulated single amplitude-cyclic strain ε_a in excess of 2.5% (which is also the strain level used for comparison of cyclic shear resistance throughout this chapter).

As may be noted, due to previous cyclic shearing, both the paste rock and rock-only specimens had reached r_u values in excess of 0.8 at the time of commencement of post-cyclic monotonic loading. During the very initial stages of monotonic loading, both the specimens registered further increase in excess pore water pressure with r_u approaching close to 1. As a result, the post-cyclic shear stress-strain response begins with very low initial shear stiffness; this response is typical of soils that have developed significantly high excess pore water cyclic loading. With increasing post-cyclic monotonic shear

strain and phase transformation occurring, the paste rock and rock-only specimens exhibited dilative response along with increasing shear stiffness. Such post-cyclic stress strain response, with initially very low and subsequent build up of shear stiffness, is also similar to those reported by Vaid and Sivathayalan (1999) from their simple shear tests on water-pluviated sands and Wijewickreme et al. (2005b) for fine-grained tailings under simple shear loading.

5.3.2 Effect of initial static shear stress

Typical cyclic response of paste rock specimens that were initially consolidated to σ'_{3c} = 200 kPa (Tests No. MSCT200-2 and MSCT200-6) with initial static shear stress bias (i.e. normalized static shear stress level = $\alpha = (\sigma'_{1c} - \sigma'_{3c})/(\sigma'_{1c} + \sigma'_{3c})$), and then subjected to cyclic loading are presented in Figures 5.12 and 5.13. The test presented in Figure 5.12 is for a specimen (Test No. MSCT200-2, α = 0.1) subjected to cyclic loading with stress reversal (or transient $\sigma_{d,cyc}$ = 0 condition), and that in Figure 5.13 is for a specimen (Test No. MSCT200-6, α = 0.2) subjected to cyclic loading without stress reversal (i.e., without σ_d cross-over of zero line).

As may be noted from the figures, the two specimens exhibited reduction in effective stress (or equivalent rise in pore water pressure) with increasing number of cycles. If shear stress reversal does not occur during cyclic loading, the specimens would not have had the opportunity to experience high r_u values (i.e., high excess pore water pressure

generation). On the other hand, with cyclic stress reversal, or zero transient shear stress condition, the samples were subjected to liquefaction with transient r_u values approaching unity. It can also be noted that the specimen that was consolidated to a relatively higher value of static bias (α = 0.2) exhibited a mild strain softening with significant strains developing in the first half-cycle. These trends are, again, qualitatively similar to those observed from cyclic tests on sand (Vaid et al. 2001).

5.3.2.1 Cyclic shear resistance

The cyclic resistant ratios versus number of cycles to initiate 2.5% single amplitude axial strain ($N_{2.5\%}$) for paste rock specimens consolidated to σ'_{3c} = 200 kPa under two initial static shear bias (α) levels are plotted in Figure 5.14. The results are compared with those obtained from tests without static bias, but otherwise with essentially identical initial stress level and density conditions.

The effect of normalized initial static shear stress α on the cyclic resistance can be examined in the context of commonly used K_α factor defined as:

[2] $$(CRR)_{\sigma',\alpha} = (CRR)_{\sigma',0}\, K_\alpha$$

where $(CRR)_{\sigma',\alpha}$ is the cyclic resistance ratio of a given sand at an arbitrary initial confining stress σ' and static bias of α, and $(CRR)_{\sigma',0}$ is the cyclic resistance ratio of a

sample of the same soil at the same density/initial confining stress, but with no static shear bias ($\alpha = 0$). It can be seen that cyclic shear resistance (CRR) of specimens having an initial static bias of $\alpha = 0.1$ is larger than those without static bias, and further increase in static bias, say $\alpha = 0.2$, would cause a reduction in CRR (or increase in relative potential for shear strain development).

Figure 5.15 shows the variation of K_α with the variable α obtained from different tests on paste rock material. Presented in the same figure are data from Vaid and Sivathayalan (1998) on Fraser River sand at various relative densities. It can be seen that general trend of change in K_α for paste rock material is similar to loose sand.

5.4. Summary

A detailed laboratory research program was undertaken to investigate the cyclic shear response of mixtures of waste rock and tailings (i.e. paste rock). Because of the low permeability inherited from the fine tailings leading to potential reduction of ARD, and high shear strength of coarse waste rock leading to increased mechanical stability, this idea of combining waste rock and tailings to form paste rock for disposal is now receiving significant attention from the point of view of sustainable mine waste management practices.

Triaxial element tests were performed on hydrostatically consolidated specimens of paste rock, rock-only and tailings-only materials (i.e. no static shear) as well as paste rock and rock-only specimens consolidated under two different values of static bias (0.1 and 0.18). Paste rock specimens were prepared to represent a state at which fine particles "just filled" the voids formed in the larger rock particle matrix.

Undrained cyclic triaxial tests indicated that reconstituted paste rock displayed "cyclic-mobility-type" strain development. Strain-softening accompanied by loss of shear strength did not occur regardless of the applied cyclic stress ratio (CSR). The cyclic shear resistance of paste rock appeared to be insensitive to the initial effective confining stress (σ'_{3c}) for the tested stress range of $\sigma'_{3c} < 400$ kPa.

Comparison of the cyclic response of paste rock specimens consolidated under different initial static shear bias conditions indicates that cyclic resistance of material increases initially with increase of static bias up to $\alpha = 0.1$ and decreases afterwards with further increase of static bias ($\alpha = 0.18$). High values of static bias ($\alpha = 0.18$) triggered a mild strain softening in paste rock material. Changes in the response of paste rock material due to changes in the value of static bias are similar to those observed from cyclic tests on loose sand (Vaid et al. 2001).

In an overall context, the laboratory findings indicate that general cyclic behavioural patterns of paste rock is similar to those observed for rock-only material. This suggests

that the rock (coarse) particle skeleton plays a dominant role in carrying the applied loads; this observation is in accord with the findings from previous research on this material from consolidation testing (Wickland 2006, Khalili et al. 2008). In relative terms, paste rock has a higher potential for strain development under a given cyclic stress ratio and number of load cycles in comparison to tailings-only and rock-only materials. The presence of tailings in the pore space in rock particles appears to have decreased the ability of the rock particles to engage and develop inter-particle stresses in comparison to the case with rock-only material.

Data from post-cyclic monotonic tests indicated a shear stress-strain response with very low initial shear stiffness as typical of soils that have developed significantly high excess pore water cyclic loading. With increasing post-cyclic monotonic shear strain and the occurrence of phase transformation, the paste rock and rock-only specimens exhibited dilative response along with increasing shear stiffness. Such behaviour is similar to those previously documented for Fraser river sand (Vaid and Sivathayalan 1999) and fine grained tailings (Wijewickreme et al. 2006). The dilative nature of post cyclic response, again, suggests that paste rock prepared to meet just filled conditions is not likely to experience catastrophic flow failure. Again, it should be noted that the data presented herein are the results of the element testing on reconstituted specimens of paste rock. Extrapolation of these results to real life scenarios should be undertaken with caution and with due attention paid to the variations between laboratory and field conditions.

Limitations in the use of results from element test for the prediction of field behaviour are discussed in Section 6.5.

Table 5.1. Test Program including key parameters.

Test No.	Material	Cell Pressure kPa	Back Pressure kPa	σ'_{3c} kPa [1]	Static Bias	Post Cyclic	CSR
MCT64	M	190.3	126.6	63.7			0.220
MCT100-1	M	249.3	150.5	98.8			0.166
MCT100-2	M	230.2	131.2	99.0			0.116
MCT100-3	M	300.9	199.8	101.1			0.173
MCT100-4	M	324.6	224.9	99.7			0.231
MCT100-5	M	233.0	133.0	100.0			0.205
MCT200-1	M	334.1	136.6	197.5			0.152
MCT200-2	M	390.9	194.4	196.5			0.079
MCT200-3	M	345.5	153.1	192.4			0.255
MCT400-1	M	550.0	161.9	388.1			0.155
MCT400-2	M	528.6	145.1	383.5			0.180
MCT400-3	M	540.7	162.9	377.8			0.109
MSCT200-1	M	354.9	156.9	198.0	0.109		0.114
MSCT200-2	M	329.9	134.2	195.7	0.105		0.171
MSCT200-3	M	347.9	143.9	204.0	0.096		0.213
MSCT200-4	M	359.8	162.8	197.0	0.178		0.114
MSCT200-5	M	343.9	145.9	198.0	0.179		0.202
MSCT200-6	M	307.8	104.0	203.8	0.177		0.162
MSCT200-7	M	344.9	143.4	201.5	0.177		0.127
MPCT200-1	M	351.5	147.9	203.6		Yes	0.157
MPCT200-2	M	377.4	177.2	200.2		Yes	0.190
MPCT200-3	M	334.3	135.8	198.5		Yes	0.126
MPCT200-4	M	386.9	185.5	201.4		Yes	0.154
RCT100-1	R	326.3	225.9	100.4			0.199
RCT100-2	R	272.3	171.1	101.2			0.168
RCT100-3	R	273.5	172.4	101.1			0.143
RCT200-1	R	351.9	146.6	205.3			0.156
RCT200-2	R	349.4	147.5	201.9			0.188
RCT200-3	R	319.4	116.7	202.7			0.227
RCT400-1	R	499.4	93.3	406.1			0.199
RCT400-2	R	470.4	68.1	402.3			0.172
RCT400-3	R	594.9	195.3	399.6			0.173
RCT400-4	R	599.2	195.0	404.2			0.156
RCT400-5	R	554.4	154.1	400.3			0.162
RCT400-6	R	614.5	214.2	400.3			0.135
RPCT200-1	R	344.5	142.1	202.4		Yes	0.193
RPCT200-2	R	288.8	88.0	200.8		Yes	0.179
RPCT200-3	R	312.5	109.4	203.1		Yes	0.158
RPCT200-4	R	355.2	153.7	201.5		Yes	0.223
TCT200-1	T	314.2	114.2	200.0			0.175
TCT200-2	T	326.0	126.0	200.0		Yes	0.225
TCT200-3	T	324.0	124.0	200.0			0.275

(1) Effective consolidation stress (σ'_{3c} in kPa)

Table 5.2. Test Program including key parameters.

Test No. [1]	$N_{2.5\%}$ [2]	Density (g/cm3) [4]	Rock : tailings ratio [5]	e^f_{total} [10]	e_{Rock} [11]	$e^f_{tailings}$ [12]	$(R_d)^f_{rock}$ [13]
MCT64	4	2.167	4.71	0.490	0.781	2.995	29.8
MCT100-1	5	2.142	4.80	0.517	0.809	3.213	22.8
MCT100-2	50	2.102	4.91	0.569	0.863	3.597	9.3
MCT100-3	3	2.146	4.82	0.515	0.804	3.206	24.0
MCT100-4	1	2.164	4.97	0.488	0.764	3.117	34.0
MCT100-5	4	2.167	4.96	0.487	0.763	3.109	34.3
MCT200-1	9	2.164	4.90	0.472	0.749	2.979	37.8
MCT200-2	50	2.171	4.93	0.459	0.732	2.912	42.0
MCT200-3	0.75 [3]	2.166	4.82	0.459	0.738	2.861	40.5
MCT400-1	10	2.151	4.80	0.467	0.749	2.900	37.8
MCT400-2	2	2.149	4.80	0.461	0.741	2.862	39.8
MCT400-3	36	2.169	4.74	0.428	0.706	2.632	48.5
MSCT200-1	50	2.171	4.91	0.460	0.734	2.910	41.5
MSCT200-2	16	2.191	4.91	0.428	0.696	2.710	51.0
MSCT200-3	2	2.136	4.79	0.487	0.773	3.017	31.8
MSCT200-4	50	2.184	4.76	0.440	0.719	2.715	45.3
MSCT200-5	1	2.165	4.89	0.458	0.733	2.887	41.8
MSCT200-6	3	2.189	4.79	0.422	0.695	2.616	51.3
MSCT200-7	9	2.156	4.57	0.470	0.766	2.803	33.5
MPCT200-1	9	2.199	4.77	0.432	0.709	2.667	47.8
MPCT200-2	2	2.111	4.91	0.542	0.832	3.428	17.0
MPCT200-3	24	2.163	4.87	0.466	0.744	2.930	39.0
MPCT200-4	9	2.172	4.84	0.461	0.739	2.881	40.3
RCT100-1	2	1.949			0.790		27.5
RCT100-2	7	1.952			0.776		31.0
RCT100-3	50	1.933			0.797		25.8
RCT200-1	21	1.965			0.754		36.5
RCT200-2	5	1.973			0.746		38.5
RCT200-3	1	1.955			0.754		36.5
RCT400-1	2	1.934			0.742		39.5
RCT400-2	24	1.929			0.766		33.5
RCT400-3	2	1.885			0.796		26.0
RCT400-4	30	1.877			0.816		21.0
RCT400-5	9	1.949			0.735		41.3
RCT400-6	50	1.929			0.751		37.3
RPCT200-1	3	1.935			0.779		30.3
RPCT200-2	5	1.912			0.807		23.3
RPCT200-3	36	1.913			0.799		25.3
RPCT200-4	1	1.935			0.759		35.3
TCT200-1	19	1.798				1.071	
TCT200-2	3	1.814				1.091	
TCT200-3	0.75 [2]	1.797				1.107	

(1) Test designation numbers as in Table 5.1;
(2) Number of cycles to reach a single-amplitude axial strain $\varepsilon_a = 2.5\%$, in a given constant volume Triaxial test under a given applied CSR defined as $N_{2.5\%}$;
(3) Specimen failed in one cycle with $\varepsilon_a \geq 2.5\%$ in that cycle;
(4) Satutared density of specimen (g/cm3);
(5) Rock to tailings ratio by dry mass;
(6) Initial overall void ratio before consolidation;
(7) Initial void ratio of rock skeleton before consolidation;
(8) Initial void ratio of tailings component before consolidation;
(9) Initial relative density of rock skeleton before consolidation;
(10) Final overall void ratio after consolidation;
(11) Final void ratio of rock skeleton after consolidation;
(12) Final void ratio of tailings component after consolidation;
(13) Final relative density of rock skeleton after consolidation.

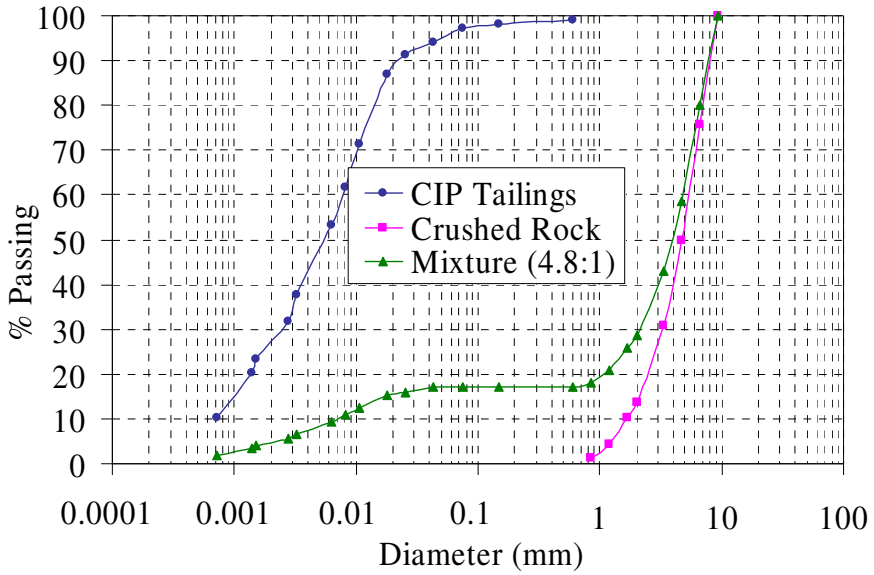


Figure 5.1. Particle size distributions of paste rock, rock-only, and tailings materials.

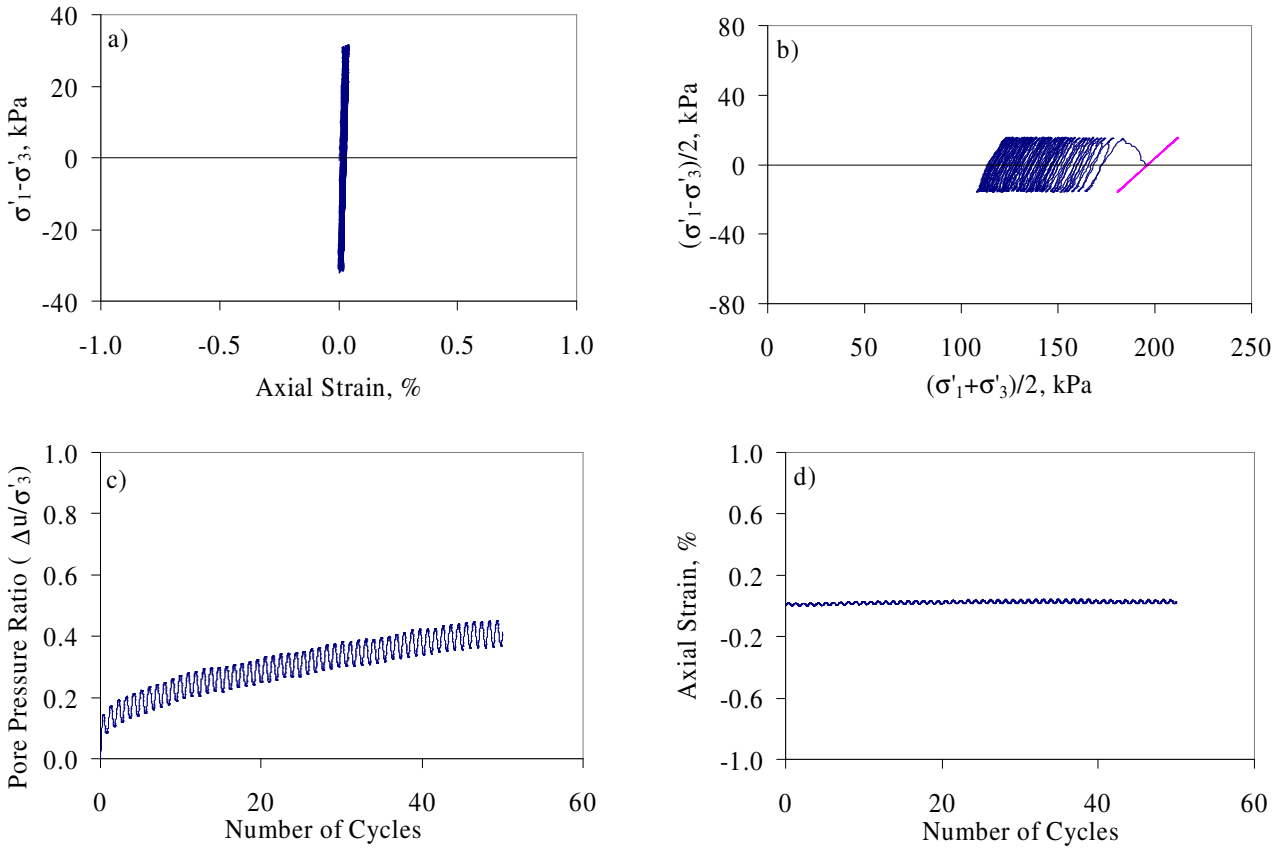


Figure 5.2. Typical response from cyclic shear testing of paste rock - Test MCT200-2; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 200 kPa; CSR= 0.079; e_{Rock} = 0.732; $e_{Tailings}$ = 2.912).

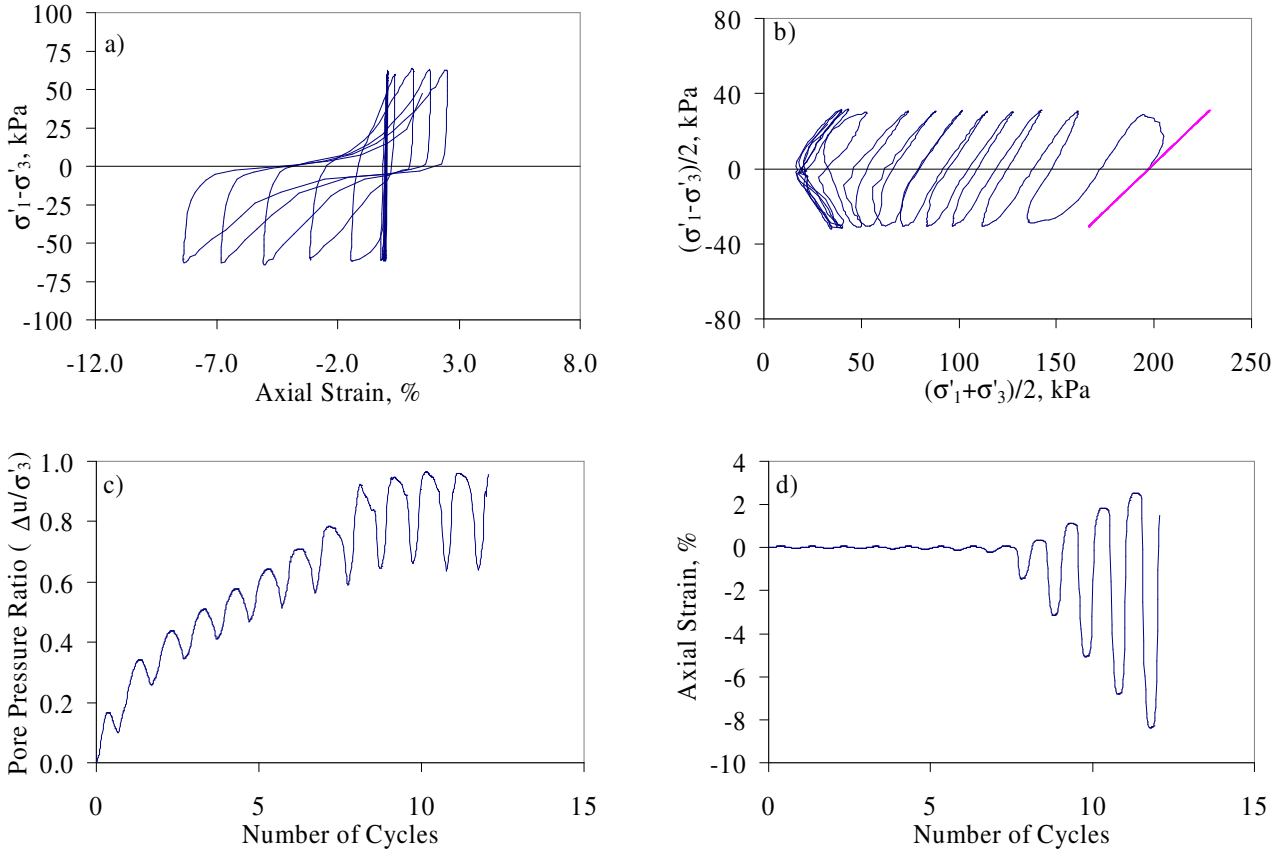


Figure 5.3. Typical response from cyclic shear testing of paste rock - Test MCT200-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 200 kPa; CSR= 0.152; e_{Rock} = 0.749; $e_{Tailings}$ = 2.979).

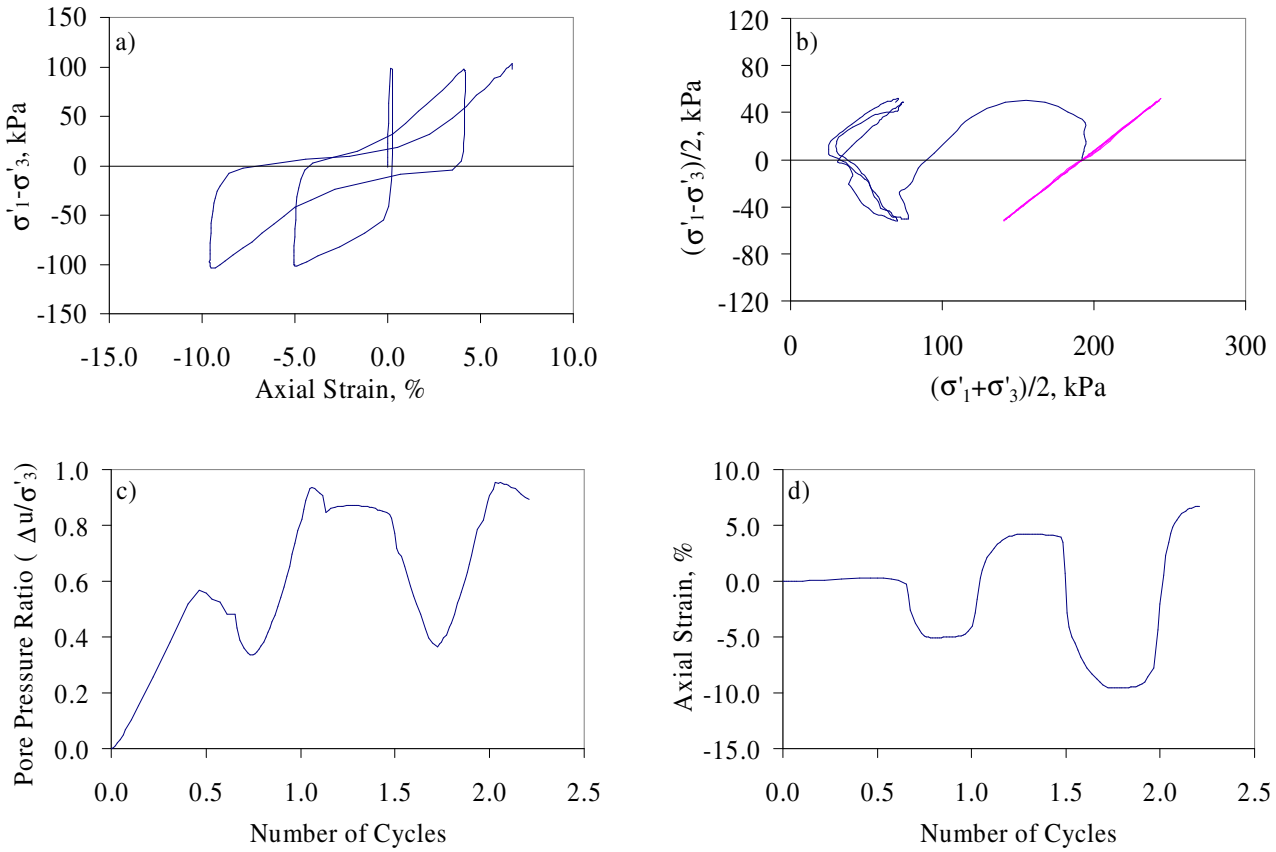


Figure 5.4. Typical response from cyclic shear testing of paste rock - Test MCT200-3; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 200 kPa; CSR= 0.255; e_{Rock} = 0.738; $e_{Tailings}$ = 2.861).

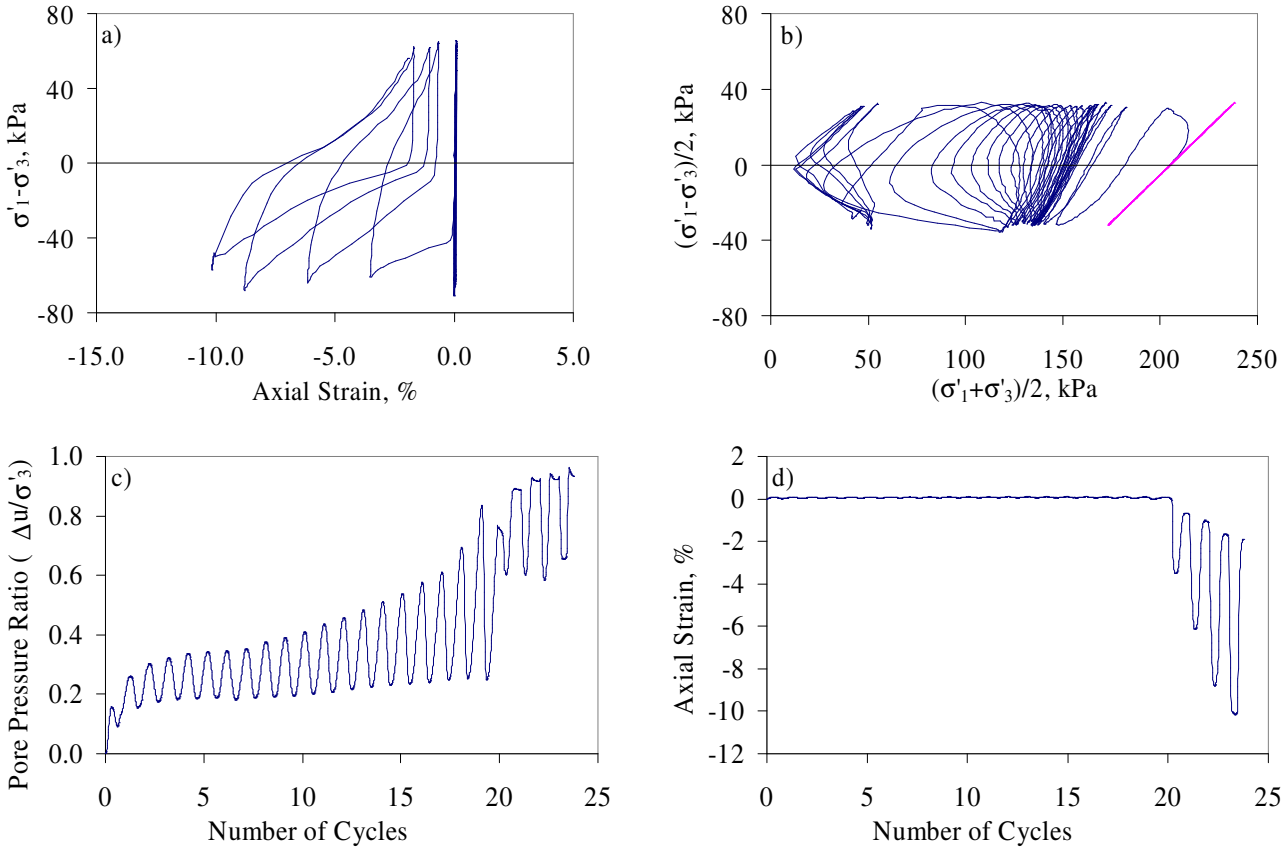


Figure 5.5. Typical response from cyclic shear testing of rock-only material - Test RCT200-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (CSR = 0.156; e_{Rock} = 0.754).

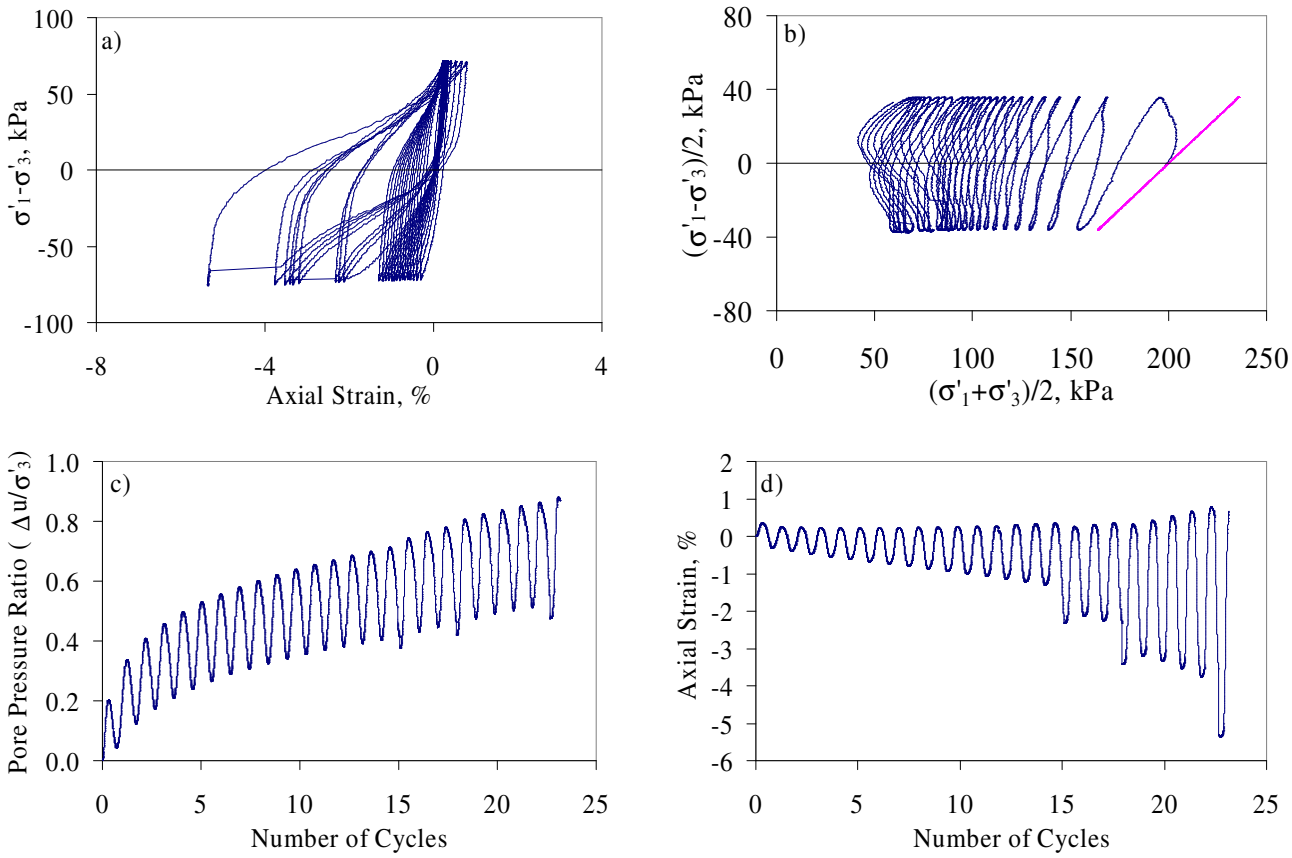


Figure 5.6. Typical response from cyclic shear testing of tailings-only material - Test TCT200-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (CSR = 0.175; $e_{Tailings}$ = 1.071).

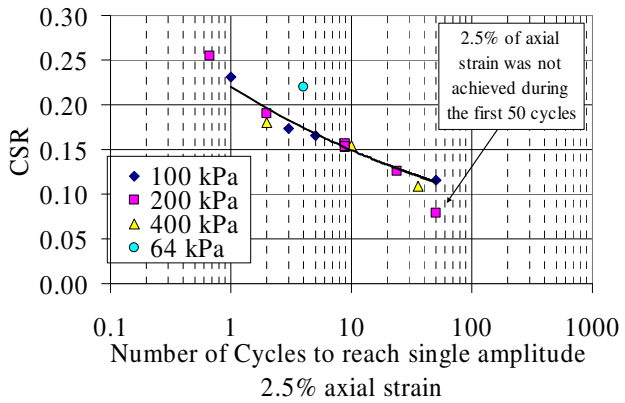


Figure 5.7. Cyclic resistance ratio (CRR) of paste rock material from tests conducted with no initial static shear bias.

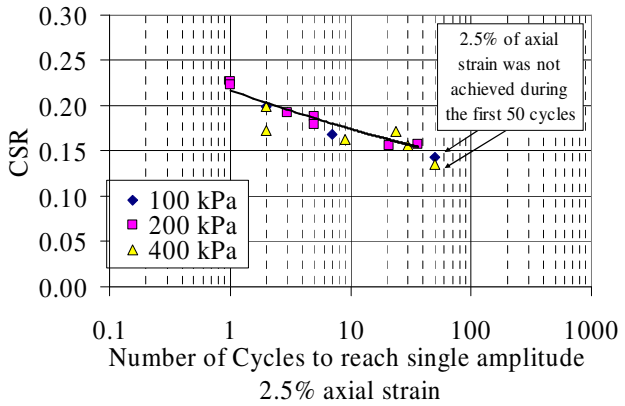


Figure 5.8. Cyclic resistance ratio (CRR) of rock-only material from tests conducted with no initial static shear bias.

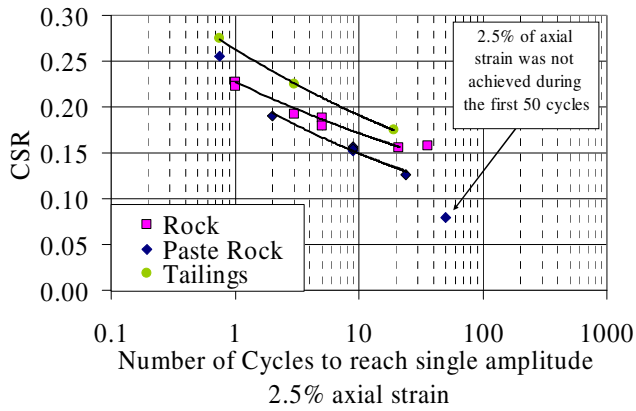


Figure 5.9. Cyclic resistance ratio of rock-only, tailings-only and paste rock materials from tests conducted with no initial static shear bias (tests conducted after consolidating specimens to σ'_{3c} =200 kPa).

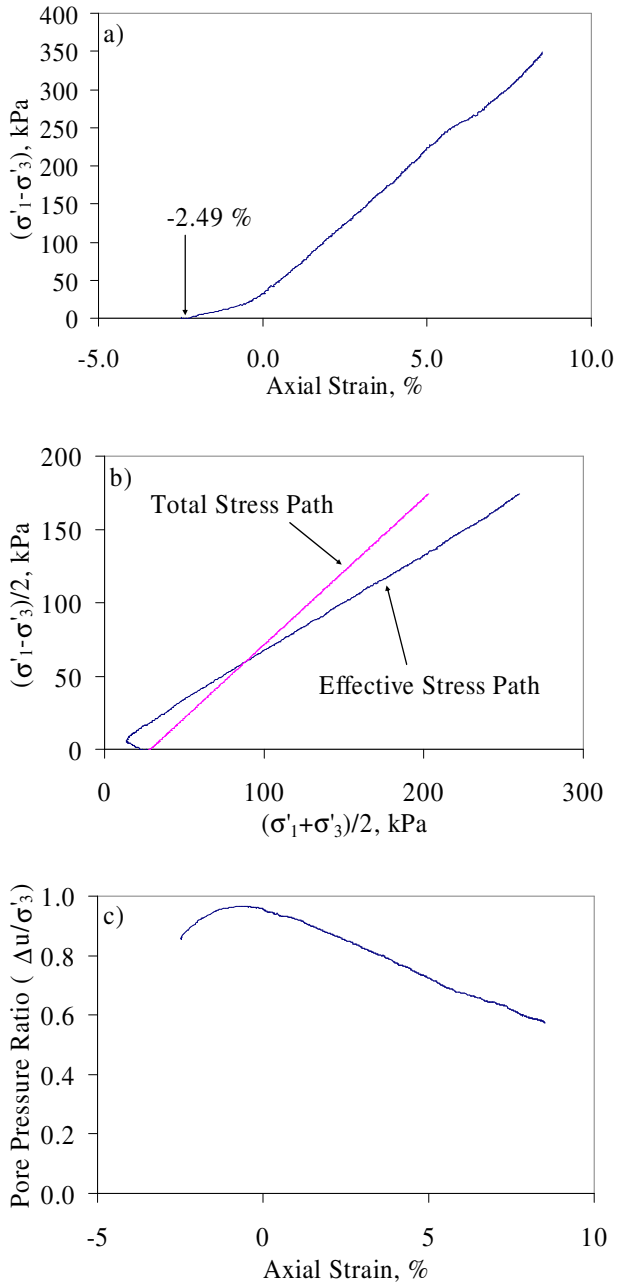


Figure 5.10. Typical post-cyclic shear test on paste rock specimen (MPCT200-4) after reaching 2.5% axial strain during cyclic shear – a) Stress-strain response b) Stress path c) Excess Pore Water Pressure.

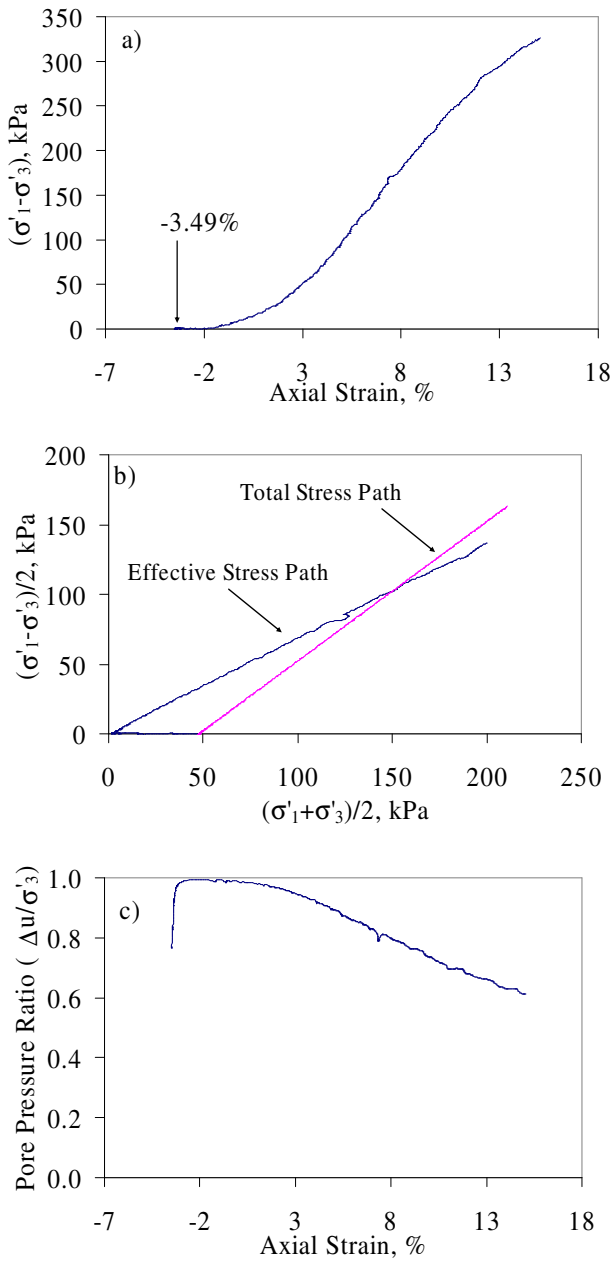


Figure 5.11. Typical post-cyclic monotonic shear response of rock specimen (RPCT200-3) after reaching 2.5% axial strain during cyclic shear – a) Stress-strain response b) Stress path c) Excess Pore Water Pressure.

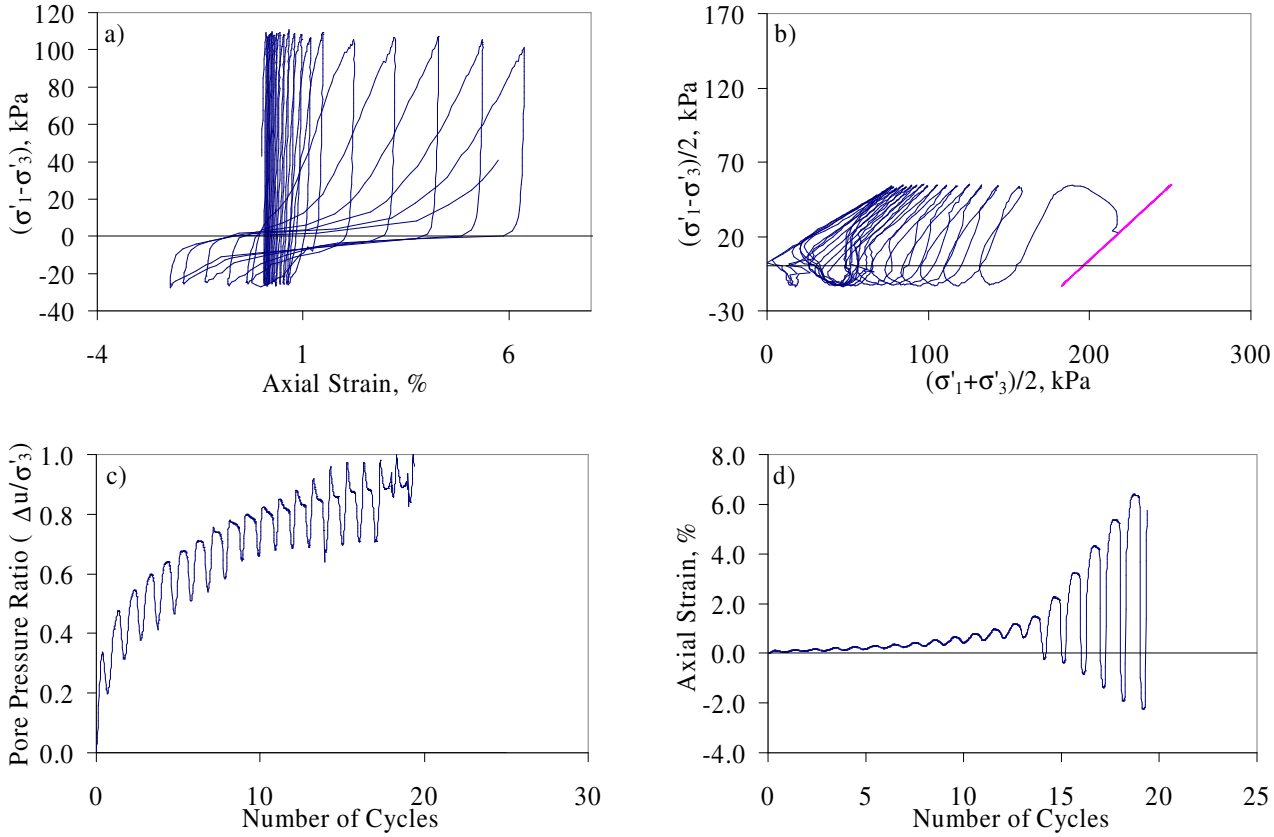


Figure 5.12. Typical response from cyclic shear testing of paste rock material with static bias (α = 0.1)- Test MSCT200-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (CSR = 0.171; e_{Rock} = 0.696; $e_{Tailings}$ = 2.710).

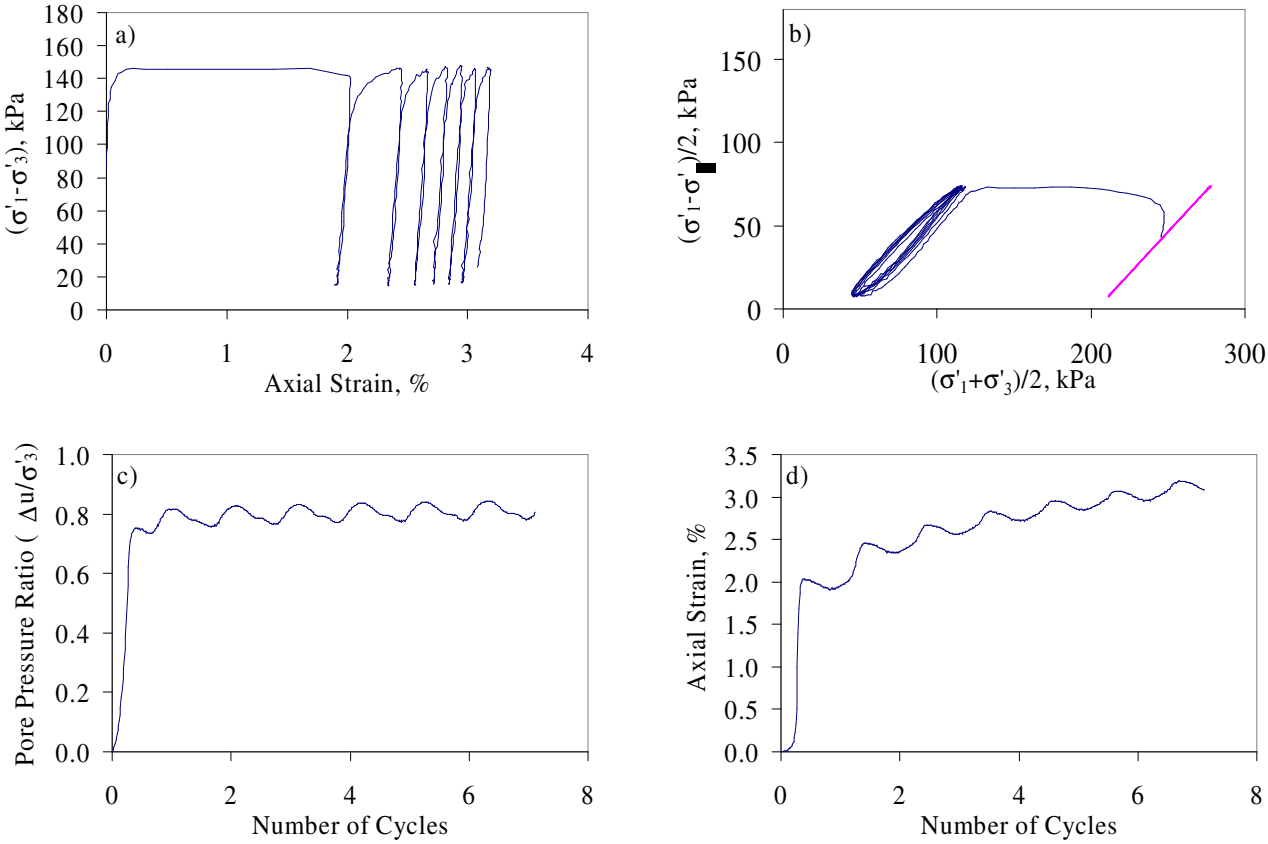


Figure 5.13. Typical response from cyclic shear testing of paste rock material with static bias (α = 0.18)- Test MSCT200-6; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (CSR = 0.162; e_{Rock} = 0.695; $e_{Tailings}$ = 2.616).

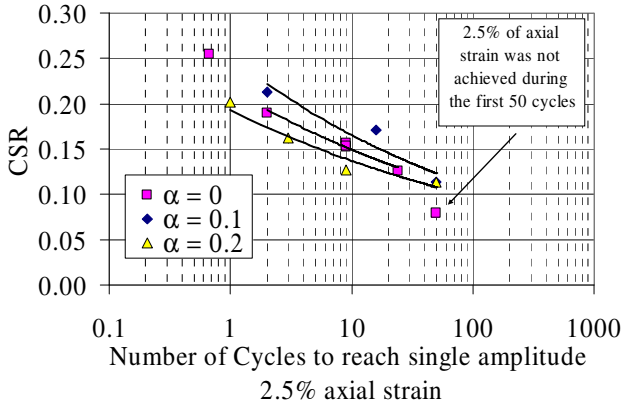


Figure 5.14. Effect of static bias on cyclic resistance of tailings/waste-rock mixtures.

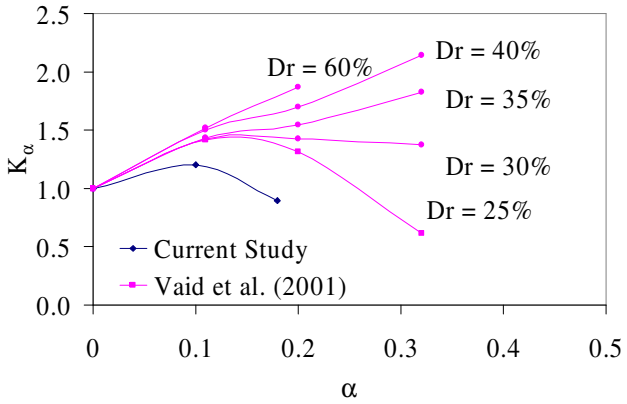


Figure 5.15. Comparison of K_α of tailings/waste-rock mixture with Fraser river delta sand. © 2008 NRC Canada or its licensors. Reproduced by permission.

5.5. References

ASTM. 2006. Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (standard D4318-05). *In* Annual Book of ASTM Standards. ASTM International, West Conshohocken, PA.

ASTM. 2006. Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer (standard D854-06). *In* Annual Book of ASTM Standards. ASTM International, West Conshohocken, PA.

ASTM. 2006. Standard test methods for maximum index density and unit weight of soils using a vibratory table (standard D4253-00). *In* Annual Book of ASTM Standards. ASTM International, West Conshohocken, PA.

ASTM. 2006. Standard test methods for minimum index density and unit weight of soils and calculation of relative density (standard D4254-00). *In* Annual Book of ASTM Standards. ASTM International, West Conshohocken, PA.

ASTM. 2002. Standard test method for particle-size analysis of soils (standard D422-63). *In* Annual Book of ASTM Standards. ASTM International, West Conshohocken, PA.

Bishop, A.W. and Henkel, D.J. 1962. The measurement of soil properties in the triaxial test. Edward Arnold Ltd., UK.

Boulanger, R.W. and Idriss, I.M. 2006. Liquefaction susceptibility criteria for silts and clays. Journal of Geotechnical and Geoenvironmental Engineering, **132:** 1413-1426.

Bray, J.D. and Sancio, R.B. 2006. Assessment of the liquefaction susceptibility of fine-grained soils. Journal of Geotechnical and Geoenvironmental Engineering, **132**: 1165-1177.

Fines, P., Wilson, G.W., Landreault, D., Langetine, L., and Hulett, L. 2003. Co-mixing of tailings, waste rock and slag to produce barrier cover systems. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, 12-18 July 2003. pp. 14-17.

Finn, W., Vaid, Y., and Bhatia, S. 1978. Constant volume cyclic simple shear testing. *In* Proceedings of the 2nd International Conference on Microzonation for Safer Construction Research and Application, San Francisco, USA, pp. 839–851.

Jamiolkowski, M., Kongsukprasert, L., and Lo Presti, D. 2005. Characterization of gravelly geomaterials. *In* Proceedings of The Fifth International Geotechnical Conference, Cairo, Egypt, 2005. Cairo University, Cairo, Egypt, pp. 1-27.

Junqueira, F., Rykaart, M., Weeks, B., Wickland, B., and Wilson, G.W. 2007. Emerging technologies in mine waste management. *In* Proceedings of the Professional Development Seminar on Emerging Technologies in Mine Waste Management, Vancouver, BC, April 2007. Association of Professional Engineers and Geoscientists of BC, Canada.

Kammerer, A., Wu, J., Pestana, J., Riemer, M., and Seed, R. 2002. Undrained response of monterey 0/30 sand under multidirectional cyclic simple shear loading conditions: Electronic data files. Report No. UCB/GT/02-02, University of California, Berkeley, California.

Khalili, A. and Wijewickreme, D. 2008. New slurry displacement method for reconstitution of highly gap-graded specimens for laboratory element testing. ASTM Geotechnical Testing Journal, **31:** 424-432.

Khalili, A., Wijewickreme, D., and Wilson, G.W. 2007. Consolidation response of mixtures of waste rock and tailings with different particle size of coarse fraction. *In* Proceedings of 60th Canadian Geotechnical Conference, Ottawa, Canada, 2007.

Khalili, A., Wijewickreme, D., and Wilson, G.W. 2005. Some observations on the mechanical response of mixtures of mine waste and tailings. *In* Proceedings of 58th Canadian Geotechnical Conference, Saskatoon, 2005. Canada.

Kokusho, T. 2003. Current state of research on flow failure considering void redistribution in liquefied deposits. Soil Dynamics and Earthquake Engineering, **23**: 585-603.

Kramer, S.L. 1996. Geotechnical earthquake engineering. Prentice-Hall, Upper Saddle River, NJ.

National Research Council (NRC). 1985. Liquefaction of soils during earthquakes. Report CETS-EE-001, National Academic Press, Wahington, D.C.

Olson, S.M. and Stark, T.D. 2002. Liquefied strength ratio from liquefaction flow failure case histories. Canadian Geotechnical Journal, **39:** 629-647.

Poulos, S.J., Robinsky, E.I., and Keller, T.O. 1985. Liquefaction Resistance of Thickend Tailings. Journal of Geotechnical Engineering, **111:** 1380-1394.

Sanin, M.V. and Wijewickreme, D. 2006. Cyclic shear response of channel-fill fraser river delta silt. Soil Dynamics and Earthquake Engineering, **26:** 854-869.

Seed, R.B. and Harder, L.F. 1990. SPT-based analysis of cyclic pore pressure generation and undrained residual strength. *In* Proceedings of the H.Bolton Seed Memorial Symposium, **2:** 351–376.

Stark, T.D. and Mesri, G. 1992. Undrained shear strength of liquefied sands for stability analysis. Journal of Geotechnical Engineering, **118:** 1727-1747.

Vaid, Y.P. and Sivathayalan, S. 1999. Post liquefaction behaviour of saturated sand under simple shear loading. *In* Proceedings of the 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, Germany, September 1997. A. A. Balkema Publishers, Rotterdam, pp. 6-12.

Vaid, Y.P. and Sivathayalan, S. 1998. Fundamental factors affecting liquefaction susceptibility of sands. *In* Proceedings of the International Workshop on the Physics and Mechanics of Soil Liquefaction, Baltimore, MD, 10-11 Sep 1998. Hopkings University, USA, pp. 105-120.

Vaid, J.P., Stedman, J.D., and Sivathayalan, S. 2001. Confining stress and static shear effects in cyclic liquefaction. Canadian Geotechnical Journal, **38:** 580-591.

Vaid, Y.P. and Thomas, J. 1995. Liquefaction and postliquefaction behaviour of sand. Journal of Geotechnical Engineering, **121:** 163-173.

Wickland, B.E. 2006. Volume change and permeability of mixtures of waste rock and fine tailings. Ph.D., University of British Columbia, Canada.

Wickland, B.E., Wilson, G.W., Wijewickreme, D., and Klein, B. 2006. Design and evaluation of mixtures of mine waste rock and tailings. Canadian Geotechnical Journal, **43:** 928-945.

Wijewickreme, D., Sriskandakumar, S., and Byrne, P. 2005a. Cyclic loading response of loose air-pluviated fraser river sand for validation of numerical models simulating centrifuge tests. Canadian Geotechnical Journal, **42:** 550-561.

Wijewickreme, D., Sanin, M.V., and Greenaway, G.R. 2005b. Cyclic shear response of fine-grained mine tailings. Canadian Geotechnical Journal, **42:** 1408-1421.

Wijewickreme, D. and Sanin, M. 2004. Cyclic shear loading response of fraser river delta silt. *In* Proceedings of 13th World Conference on Earthquake Engineering (13th WCEE), Vancouver, Canada, 2004.

Williams, D.J., Wilson, G.W., and Panidis, C. 2003. Waste rock and tailings mixtures as a possible seal for potentially acid forming waste rock. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, 12-18 July 2003. pp. 14-17.

Williams, D.J., Gowan, M., and Keefer, P. 1995. Practical co-disposal deposition. *In* Proceedings of the 7th Australian Coal Preparation Conference, Mudgee, New South Wales, 1995. pp. 9-15.

Wilson, W.G. 2001. Co-disposal of tailings and waste rock, Geotechnical News, **19**: 44-49.

Wilson, G.W., Plewes, H.D., Williams, D., and Robertson, J. 2003. Concepts for co-mixing of tailings and waste rock. *In* Application of Sustainability of Technologies: Proceedings of the 6th International Conference on Acid Rock Drainage (ICARD), Cairns, Queensland, Australia, pp. 14–17.

Youd, T.L. and Idriss, I.M. 2001. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering, **127:** 297-313.

Zergoun, M. and Vaid, Y. 1994. Effective stress response of clay to undrained cyclic loading. Canadian Geotechnical Journal, **31:** 714-727.

Chapter 6.

Summary and Conclusion

The idea of disposing mine waste as a mixture of waste rock and tailings (also called paste rock) has received increased attention because of various advantages over the traditional methods of disposal in the form of waste rock dumps and tailings impoundments A thorough understanding of the mechanical behaviour of paste rock would constitute a key part in implementing this idea into real-life applications. With this background, a comprehensive laboratory research program was undertaken to study the mechanical behaviour of paste rock under monotonic and cyclic shear loading conditions, thus forming the main theme of this thesis.

A series of monotonic/cyclic and drained/undrained triaxial shear tests conducted on laboratory specimens prepared from paste rock and its constituents formed the core of the research program. All tests on paste rock were conducted using a waste rock to tailings

mixture ratio that would yield the maximum density for the mixture – i.e., with tailings particles "just filling" the voids among rock grains. In addition to the triaxial shear tests, a limited number of 1-D and hydrostatic consolidation tests were performed on the same materials to investigate the consolidation behaviour of the paste rock and to compare it with previous studies. All tests were conducted using paste rock prepared to meet an optimum mixture ratio (with tailings particles just filling the voids of the coarser skeleton) that has been suggested to be preferable in terms of resisting loads based on a previous study by Wickland (2006).

6.1. Major Topics of Research

1. A new computer-controlled triaxial shear device that is capable of performing stress path tests on soil specimens having diameters up to 150 mm (6") was designed and fabricated as a part of this research program. The device has built-in capabilities for real-time membrane penetration correction during undrained shearing phase.

2. Because of the highly gap-graded nature of the tested paste rock material, extensive time and effort was expended during the initial phase of this research to develop a method for preparing saturated, uniform, repeatable test specimens. This work led to a novel technique for the reconstitution of saturated soil specimens for laboratory testing. The technique can be practically extrapolated

for use with any highly gap-graded material such as gravel-silt, gravel-clay and sand-clay mixtures. The high angularity of the coarser fraction of the paste rock required the development of unique membrane configurations to overcome membrane puncture during specimen preparation; the process involved extensive trial and error approaches.

3. Study of hydrostatic and 1-D consolidation response of paste rock and effect of gradation and particle size on specimen response.

4. Study of monotonic shear response of paste rock; including behaviour of the material during compression and extension in both drained and undrained conditions.

5. Study of cyclic shear response of paste rock; including fundamental comparison between behaviour of paste rock, rock-only, and tailings-only materials as well as studying the effect of static bias on the shear response of paste rock.

6. Study of post cyclic shear response of paste rock and comparison of such behaviour with post cyclic shear response of rock-only material.

The following sections (i.e. section 6.2 to 6.4) summarize the conclusions derived from laboratory work in this research with respect items 3 to 6 above.

6.2. Consolidation Response – Summary of Findings

Examination of laboratory data from limited consolidation tests conducted on paste rock specimens reconstituted at "just filled" mixture ratio (i.e. the mixture ratio at which the tailings particles just fill the voids of the coarser skeleton) along with those from waste rock-only and tailings-only specimens suggest that a great proportion of the applied load is carried by the particle skeleton of waste rock. This finding is also in accord with previous research performed by Wickland (2006), and further confirms the importance of the waste rock particle matrix in governing the overall performance of a paste rock mass from an engineering point of view.

Jamiolkowski et al. (2005) has suggested that the specimen diameter to maximum particle size ratio (D_s/D_{max}) should be greater than 5, with a ratio of 8 being ideal, for minimizing stress non-uniformities arising from large particles inside a test specimen and, in turn, yielding good quality results from laboratory element testing. The data from consolidation tests performed on paste rock as a part of this study along with those from previous studies on consolidation behaviour of the same material indicated that changing the D_s/D_{max} from 6 to 15 did not affect the observed general trends in the consolidation response; this supports the observations by Jamiolkowski et al. (2005).

The compressibility index (C_c) and coefficient of consolidation (C_v) values obtained from hydrostatic consolidation of triaxial specimens were in line with those derived based on 1-D consolidation testing of paste rock.

6.3. Monotonic Shear Response – Summary of Findings

Monotonic triaxial element tests were performed on hydrostatically consolidated specimens of paste rock, rock-only, and tailings-only materials. Under both drained and undrained triaxial compression loading, all paste rock specimens displayed an initially contractive behaviour followed by dilative response. Moreover, the contractive response of the specimens was noted to increase with increasing σ'_{3c}. Limited liquefaction type of strain development and mild strain softening was observed during undrained loading in higher effective confining stress levels; no strain softening associated with flow failure and loss of strength was observed during triaxial tests on any of the material during compression or extension.

Comparison between the monotonic shear performance of paste rock, rock-only and tailings-only specimens reveals that the strength and deformation characteristics of the paste rock is slightly weaker than the rock-only specimens at comparable consolidation effective stresses. The research findings clearly suggest that the mechanical response of paste rock under monotonic triaxial shear loading is more similar to that observed for waste rock alone than that for tailings alone; this also highlighted the key role played by

the waste rock particle matrix in providing the shear resistance of paste rock at "just filled" state.

6.4. Cyclic Shear Response – Summary of Findings

Cyclic triaxial element tests were performed on hydrostatically consolidated specimens of paste rock, rock-only and tailings-only materials as well as specimens initially consolidated under a static shear stress bias (defined as static shear stress bias level $\alpha = (\sigma'_{1c} - \sigma'_{3c})/(\sigma'_{1c} + \sigma'_{3c})$). Cyclic mobility type of strain development was observed during cyclic tests on paste rock. No strain softening with sudden loss of shear stiffness was observed under any cyclic stress ratio (CSR). This suggests that the tested paste rock is not likely to experience flow failure under cyclic loading conditions. The cyclic resistance ratio of paste rock was found to be insensitive to initial effective confining stress (σ'_{3c}) for the tested stress range of $\sigma'_{3c} < 400$ kPa.

Cyclic shear resistance of paste rock was observed to initially increase with increasing static shear bias up to an α value of 0.1; the cyclic shear resistance then decreased with further increase of α. A mild strain softening behaviour was observed during cyclic tests on paste rock with initial static bias value of 0.18 although it was not accompanied by significant loss of stiffness (i.e. flow failure type of behaviour). The general behavioural trend of paste rock with respect to static bias is similar to those documented for the response of sand by others (Vaid et al. 2001).

Post-cyclic monotonic undrained tests performed on paste rock and rock-only specimens indicate that both materials initially exhibited low shear stiffness; with increased shearing, then displayed phase transformation followed by dilative response and increase of shear resistance. Such dilative nature of post cyclic response is further confirming the previous findings that paste rock prepared to meet "just filled" state is not likely to experience catastrophic flow failure.

Similar to the general response under monotonic shear loading, general cyclic performance and behaviour observed from paste rock specimens were found to be more similar to that from waste rock-only specimens than that from tailings-only specimens. This finding that the waste rock skeleton is responsible for providing a large proportion of the shear resistance of paste rock at "just filled" state is in accord with the findings from previous research on this material from consolidation testing (Wickland 2006, Khalili et al. 2007).

Comparison of the shear behaviour patterns, under a given cyclic stress ratio, suggests that paste rock has a higher potential for cyclic shear strain development than waste rock alone. The presence of tailings particles in paste rock appears to have reduced the ability of the rock particles to engage and effectively develop inter-particle stresses in comparison to the conditions in a waste rock-only matrix where no fine particles are present in the void between the coarse particles. In spite of the slightly higher tendency for cyclic shear strain development, the no strain softening tendency of the material (i.e.

no flow type of failure) suggests that, with proper engineering, there will be potential for the use of this material in the field applications. Additional laboratory and field research would be required prior to arriving at definitive conclusions.

6.5. Limitations

Conclusions derived from this research work are based on the results of laboratory element testing on one type of tailings, waste rock and their mixtures. It is important to recognize that there are many variables that may affect the behaviour of tailings/waste rock mixtures, particularly with respect to extending the experimental observations to predict possible field behaviour of structures constructed using paste rock. As such, it is suggested that future use of the data in this thesis for engineering purposes should be undertaken with due accounting of the following key considerations/limitations:

1- The grain size distribution of tailings and waste rock materials are likely to be highly variable, and the tailings and waste rock mixture ratios to achieve the "just filled" condition (which was used for the paste rock tested herein) may vary accordingly.
2- Depending on the mixture ratios, grain size distribution, and other properties of the paste rock constituents, both fine-grained and coarse-grained particles may participate in the load bearing and providing shear resistance. The mechanical

behaviour in such situations may be significantly different from those presented herein.

3- The chemical properties of the paste rock constituents, as well as weathering processes and aging effects, might lead to breakage of particles and/or creating bonds between particles over time. These effects are not reflected in tests conducted in this thesis on reconstituted specimens.

4- Consolidation, monotonic, and cyclic behaviour of the rock and tailings components in practice might be different from those presented herein depending on minerals present in the ore and extraction processes used.

6.6. Future Studies

The research presented in this thesis was essentially focused on the monotonic and cyclic performance of paste rock. To extend the knowledge on the mechanical behaviour of paste, and particularly to address issues arising from the present research work, it is recommended that the following work be undertaken as a part of future research on this subject:

1- Study of the grain size variability on the overall behaviour of paste rock,

2- Study of mechanical response of paste rock in mixture ratios at which coarser skeleton is not the main load bearing skeleton. Due attention should be paid to the behaviour of paste rock as primary load bearing skeleton transitions from waste rock to tailings.

3- Investigation of the effects of aging and chemical activities over time on the behaviour of paste rock.
4- Investigation of piping vulnerability and its consequences,
5- Climate interaction, weathering process and the effect of freeze/thaw cycles on the performance of the paste rock,
6- Investigation of practical methods of implementation on site,
7- Effect of rock and tailings fabric and gradation on the performance of the material in general.
8- Particle flow modeling to investigate the fabric anisotropy of paste rock and rock-only material.

In absence of material-specific data, careful consideration should be given when extrapolating the data presented in this thesis for other cases involving different materials. Cost/benefit assessments combined with value engineering approaches would also have a key role to play in extending the findings of this research to field applications.

6.7. References

Jamiolkowski, M., Kongsukprasert, L., and Lo Presti, D. 2005. Characterization of gravelly geomaterials. *In* Proceedings of The Fifth International Geotechnical Conference, Cairo, Egypt, 2005. Cairo University, Cairo, Egypt, pp. 1-27.

Khalili, A., Wijewickreme, D., and Wilson, G.W. 2007. Consolidation response of mixtures of waste rock and tailings with different particle size of coarse fraction. *In* Proceedings of 60th Canadian Geotechnical Conference, Ottawa, Canada.

Vaid, Y.P. and Chern, J.C. 1985. Cyclic and monotonic undrained response of saturated sands. *In* Advances in the Art of Testing Soils Under Cyclic Conditions: Proceedings of a session held in conjunction with the ASCE Convention.ASCE, New York, NY, USA, Detroit, MI, Engl, pp. 120-147.

Vaid, J.P., Stedman, J.D., and Sivathayalan, S. 2001. Confining stress and static shear effects in cyclic liquefaction. Canadian Geotechnical Journal, **38:** 580-591.

Wickland, B.E. 2006. Volume change and permeability of mixtures of waste rock and fine tailings. Ph.D., University of British Columbia, Canada.

Appendix 1.

Cyclic Triaxial Test Results

This appendix presents the results of all the cyclic triaxial tests including post-cyclic monotonic shear tests performed as part of this research. The monotonic test results are all presented in Chapter 4.

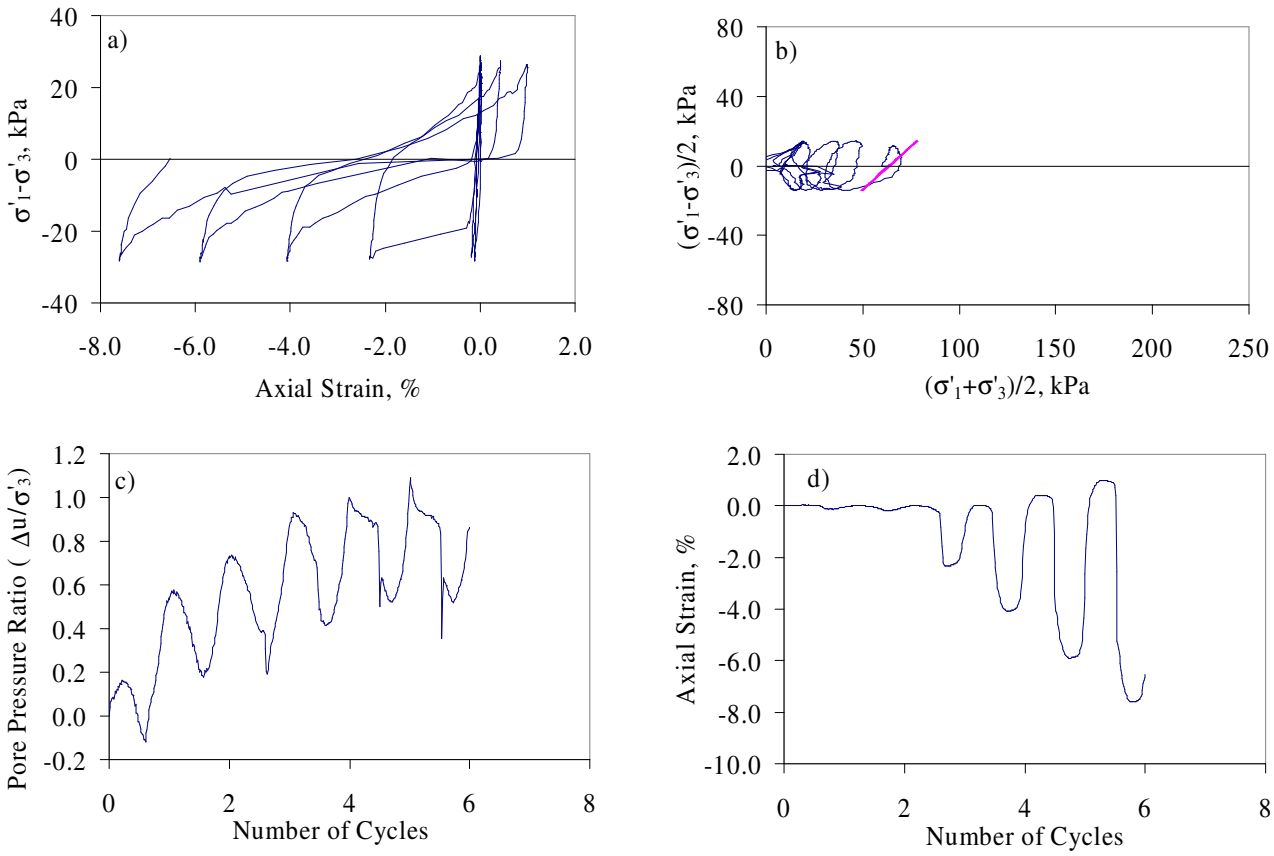


Figure A1.1. Cyclic shear response of paste rock material - Test MCT64; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 63.7 kPa; CSR= 0.220; e_{Rock} = 0.732).

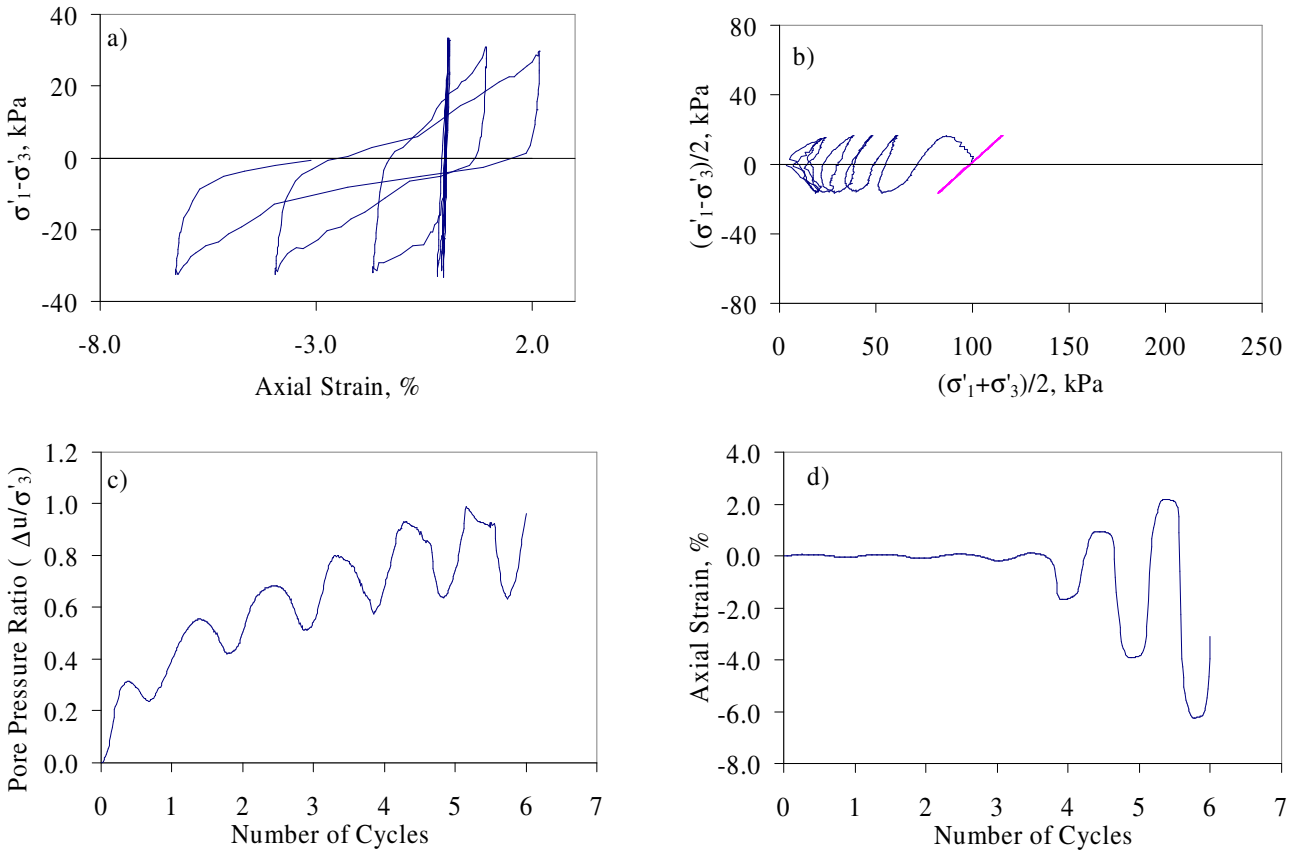


Figure A1.2. Cyclic shear response of paste rock material - Test MCT100-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 98.8 kPa; CSR= 0.166; e_{Rock} = 0.809).

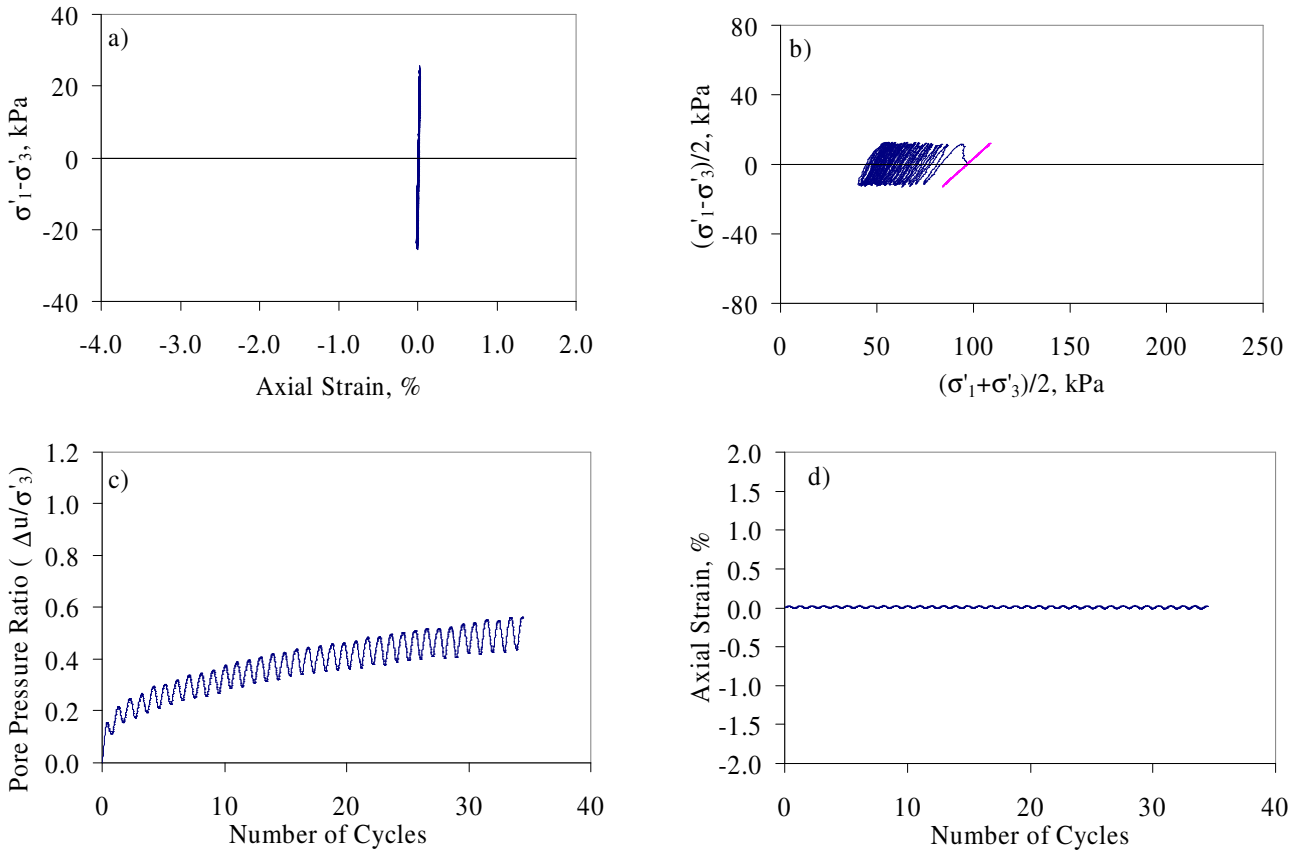


Figure A1.3. Cyclic shear response of paste rock material - Test MCT100-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 99.0 kPa; CSR= 0.116; e_{Rock} = 0.863).

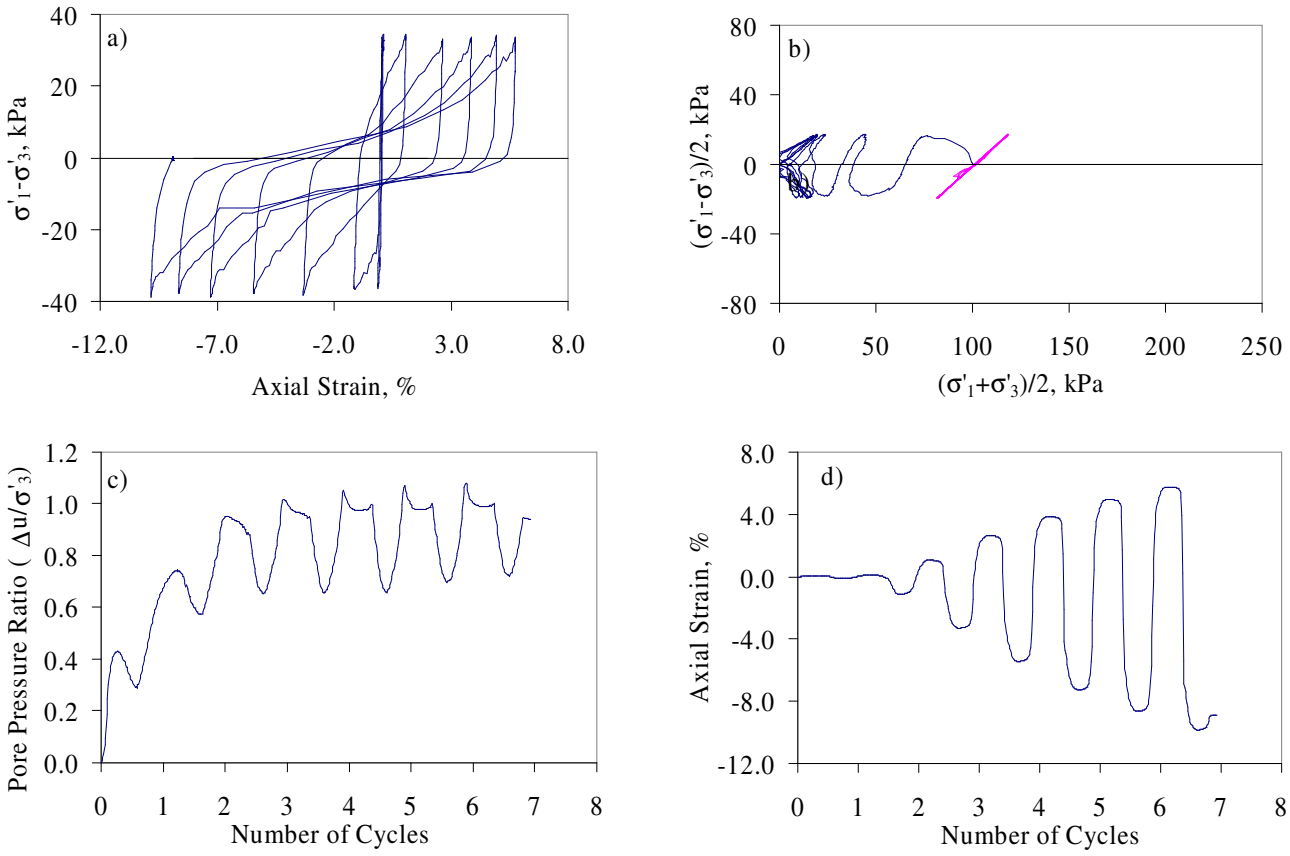


Figure A1.4. Cyclic shear response of paste rock material - Test MCT100-3; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 101.1 kPa; CSR= 0.173; e_{Rock} = 0.804).

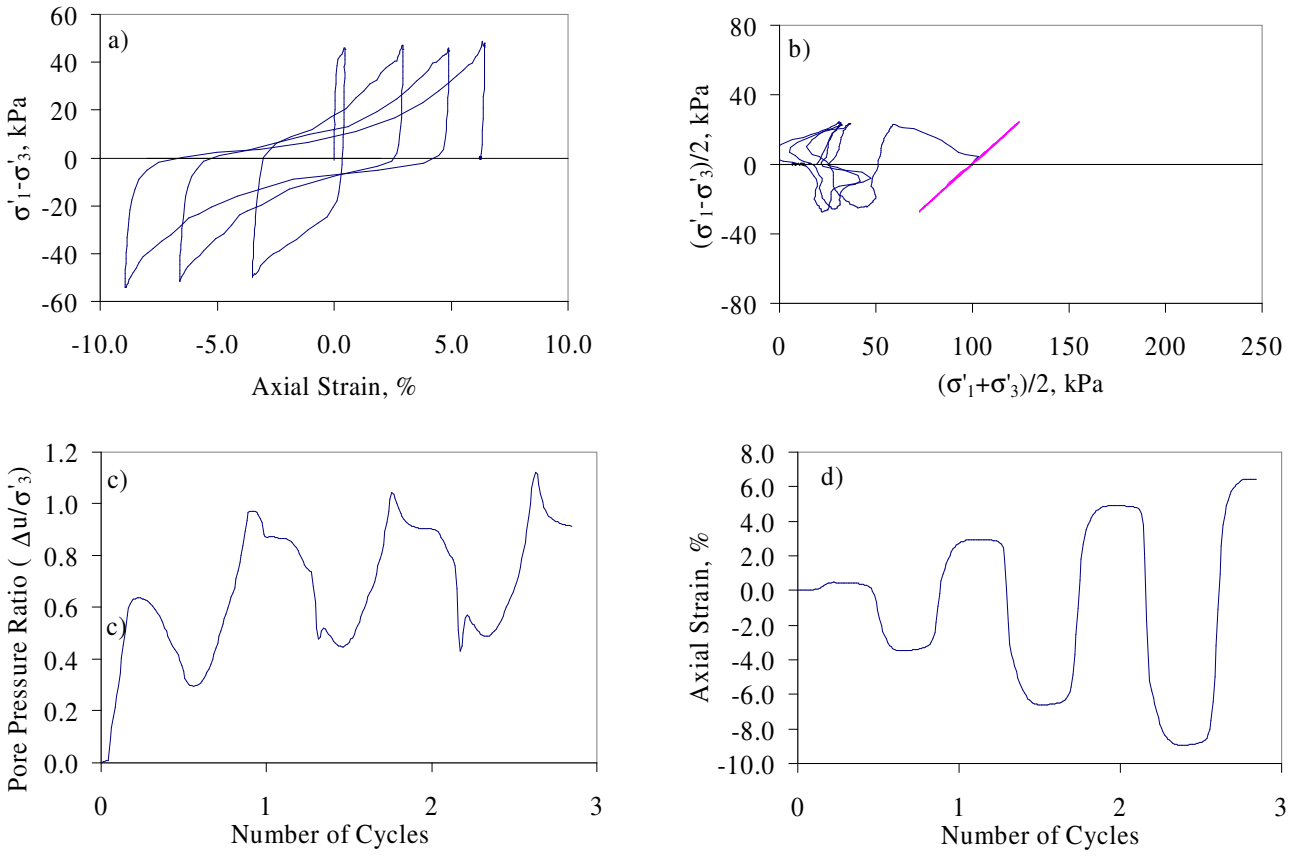


Figure A1.5. Cyclic shear response of paste rock material - Test MCT100-4; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 99.7 kPa; CSR= 0.231; e_{Rock} = 0.764).

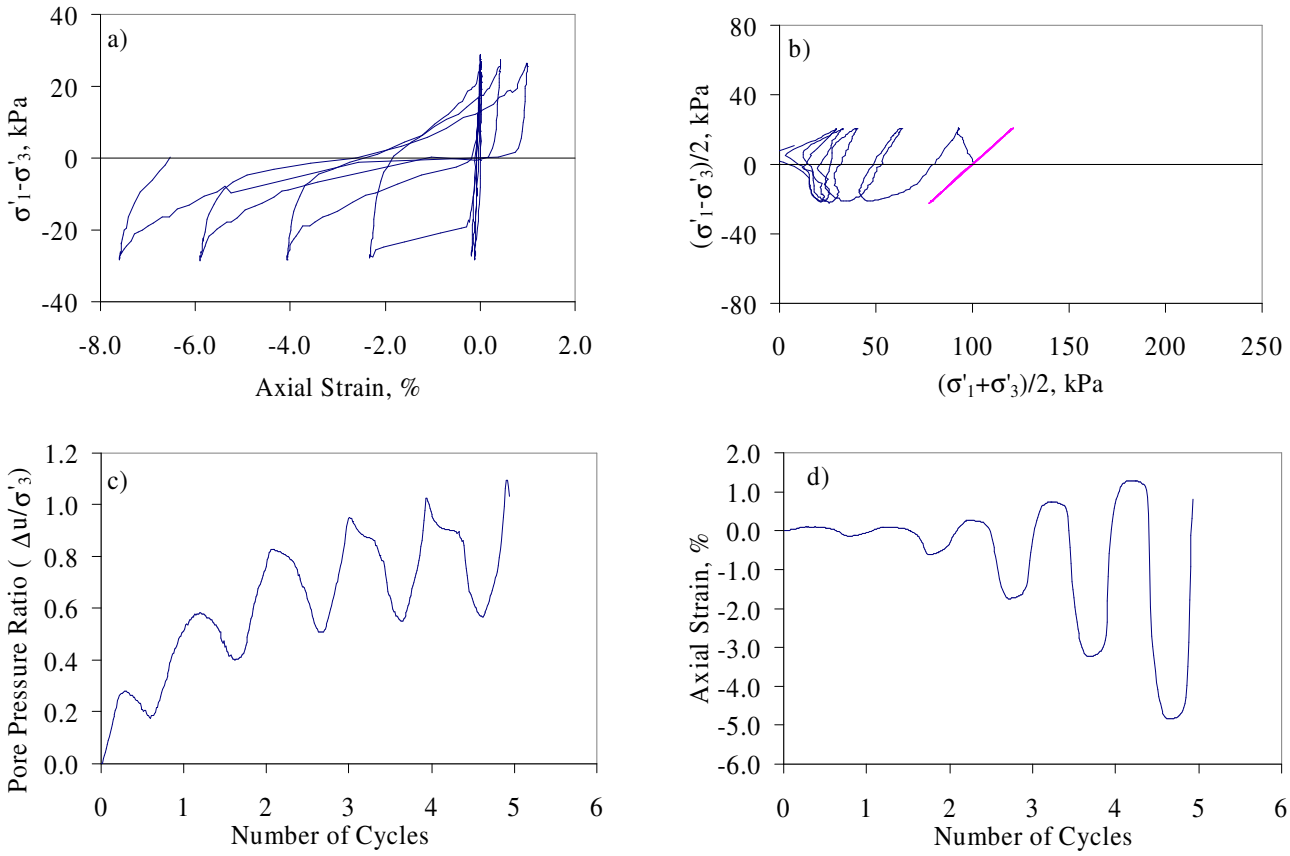


Figure A1.6. Cyclic shear response of paste rock material - Test MCT100-5; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 100.0 kPa; CSR= 0.205; e_{Rock} = 0.763).

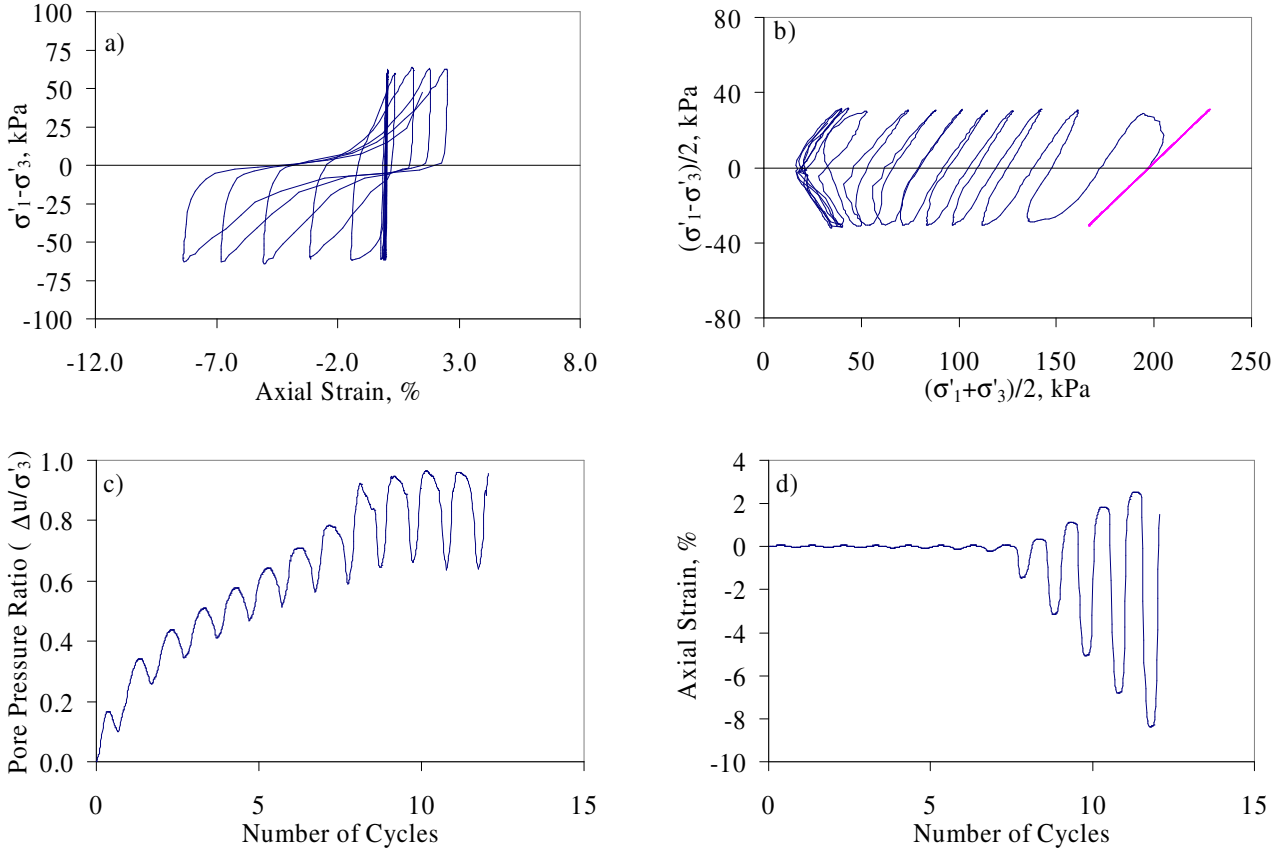


Figure A1.7. Cyclic shear response of paste rock material - Test MCT200-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 197.5 kPa; CSR= 0.152; e_{Rock} = 0.749).

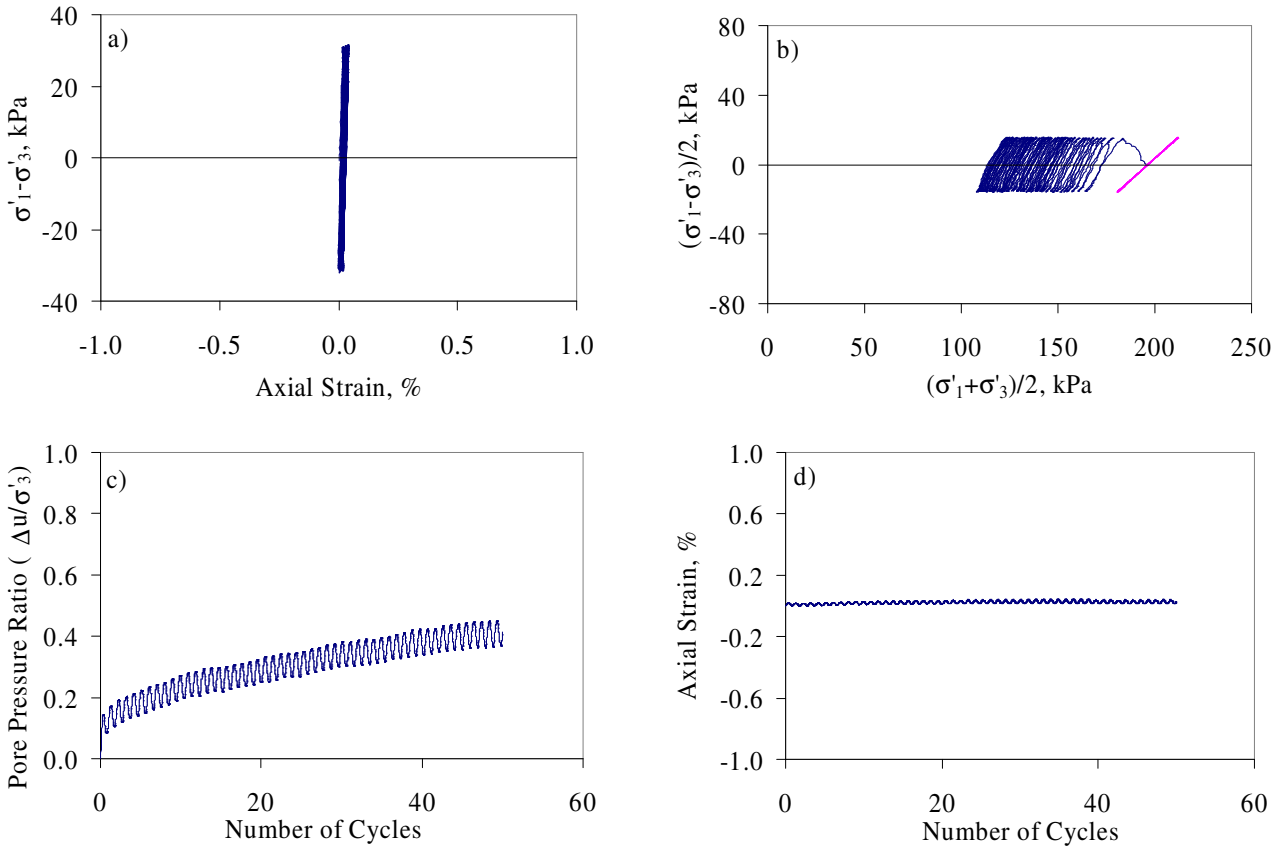


Figure A1.8. Cyclic shear response of paste rock material - Test MCT200-2; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 196.5 kPa; CSR= 0.079; e_{Rock} = 0.732).

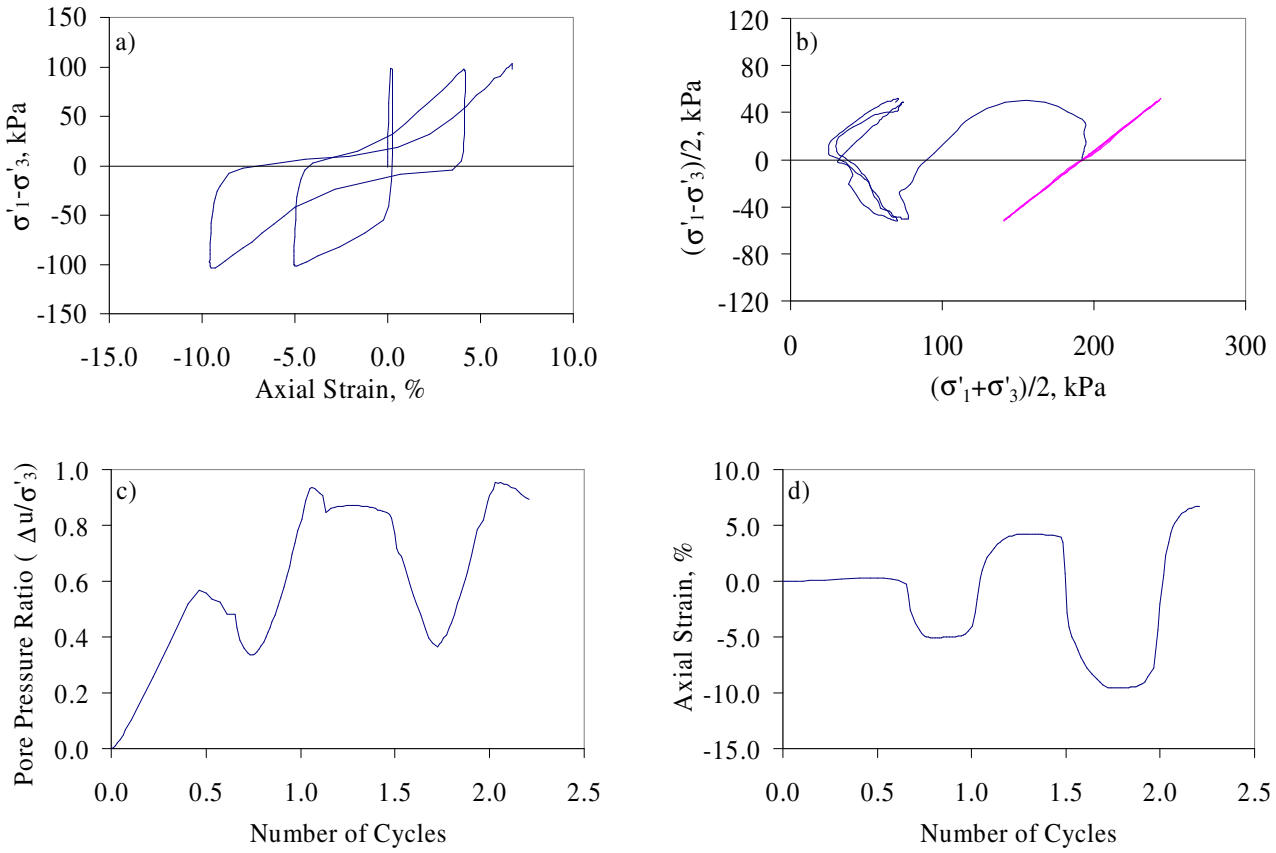


Figure A1.9. Cyclic shear response of paste rock material - Test MCT200-3; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 192.4 kPa; CSR= 0.255; e_{Rock} = 0.738).

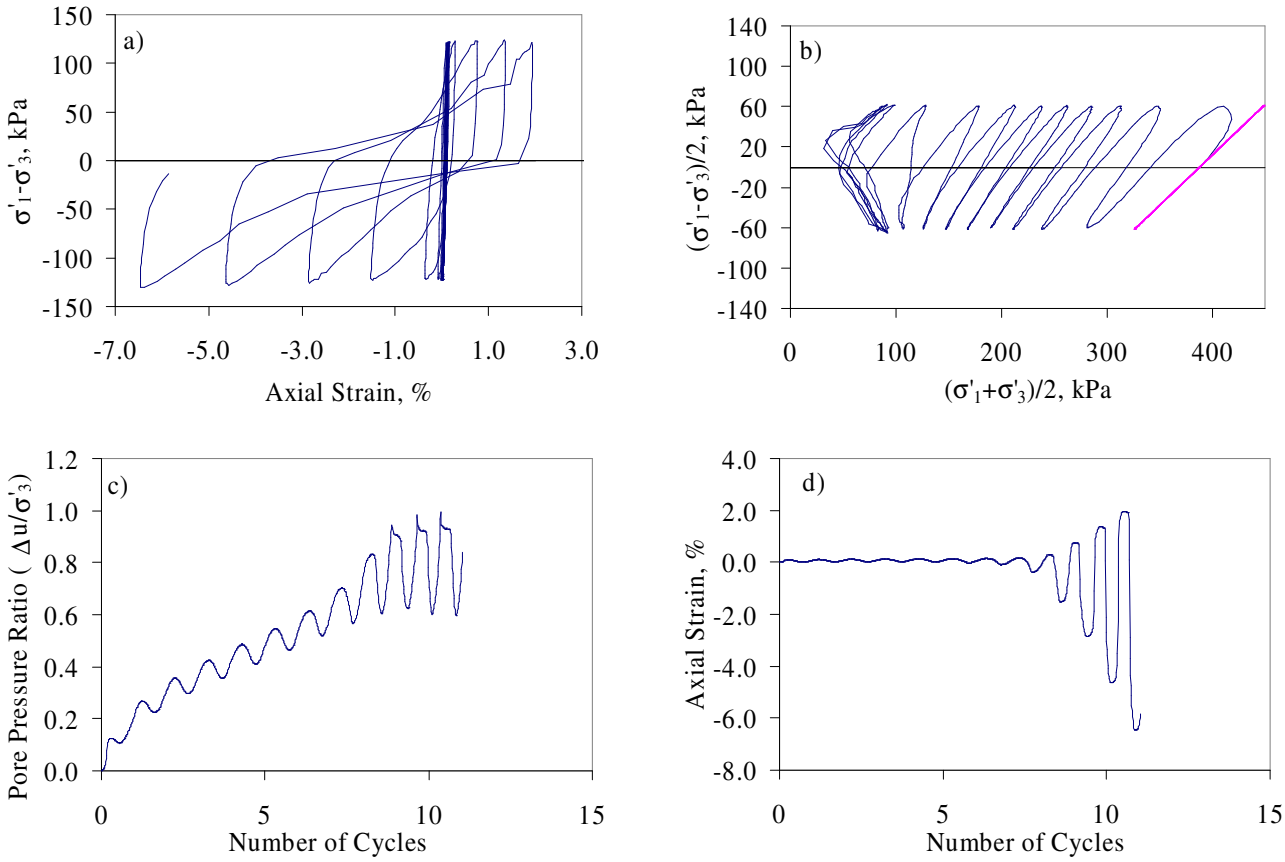


Figure A1.10. Cyclic shear response of paste rock material - Test MCT400-1; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 388.1 kPa; CSR= 0.155; e_{Rock} = 0.749).

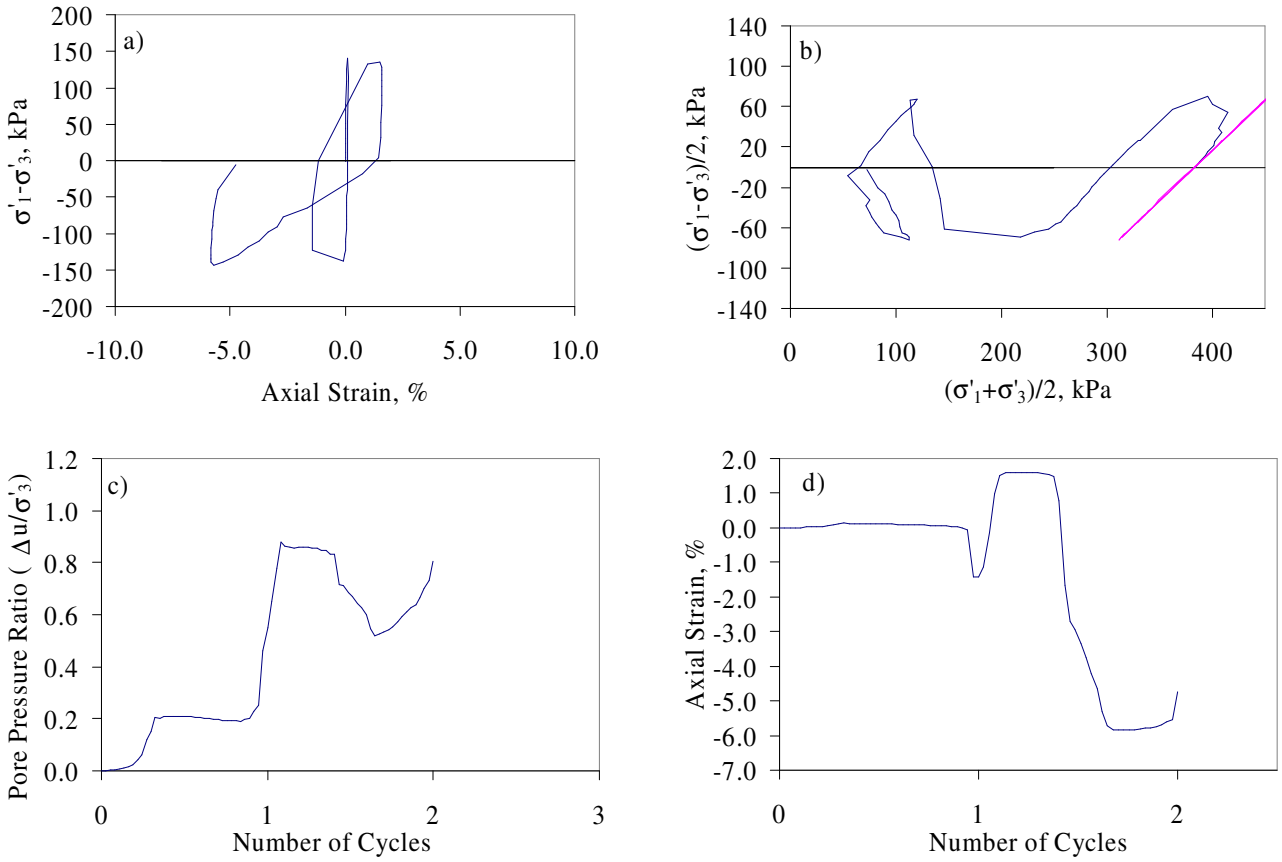


Figure A1.11. Cyclic shear response of paste rock material - Test MCT400-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 383.5 kPa; CSR= 0.180; e_{Rock} = 0.741).

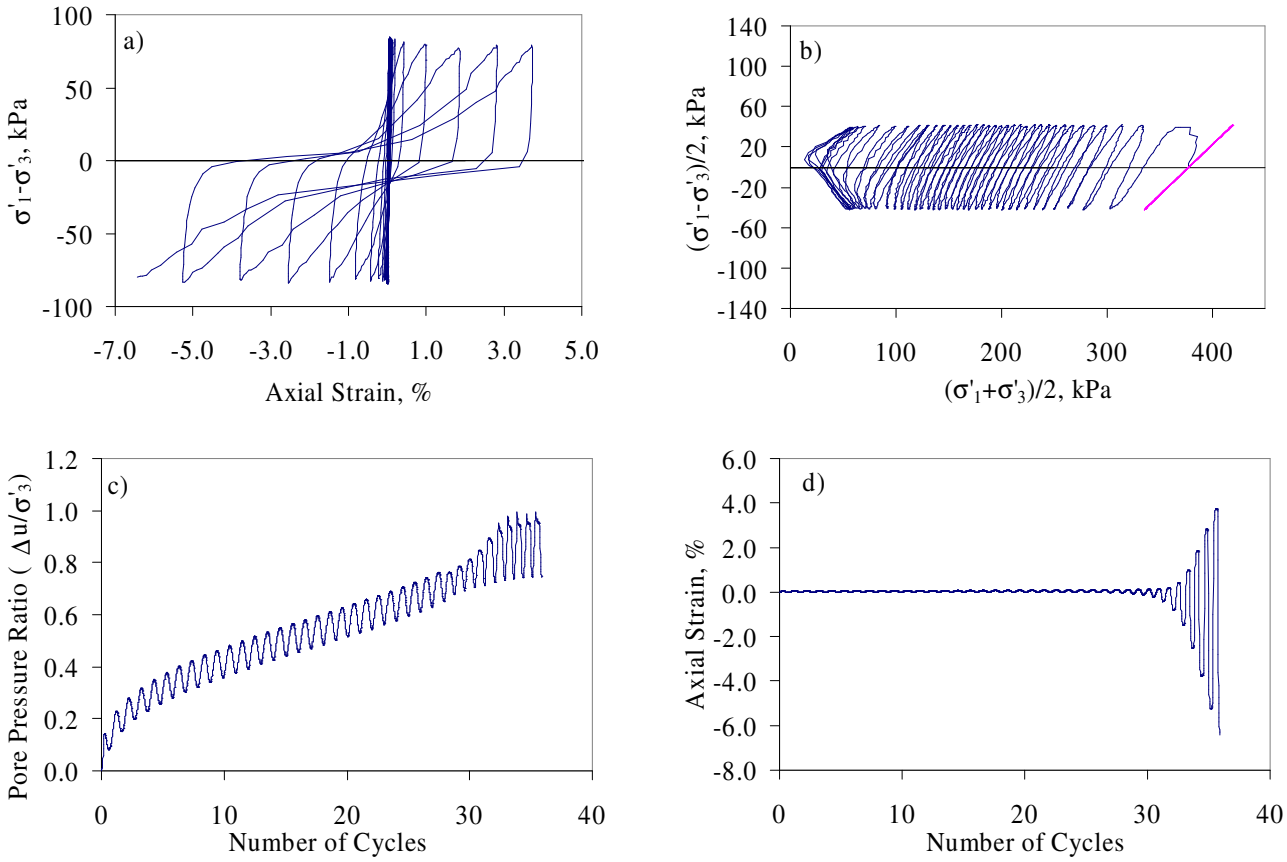


Figure A1.12. Cyclic shear response of paste rock material - Test MCT400-3; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 377.8 kPa; CSR= 0.109; e_{Rock} = 0.706).

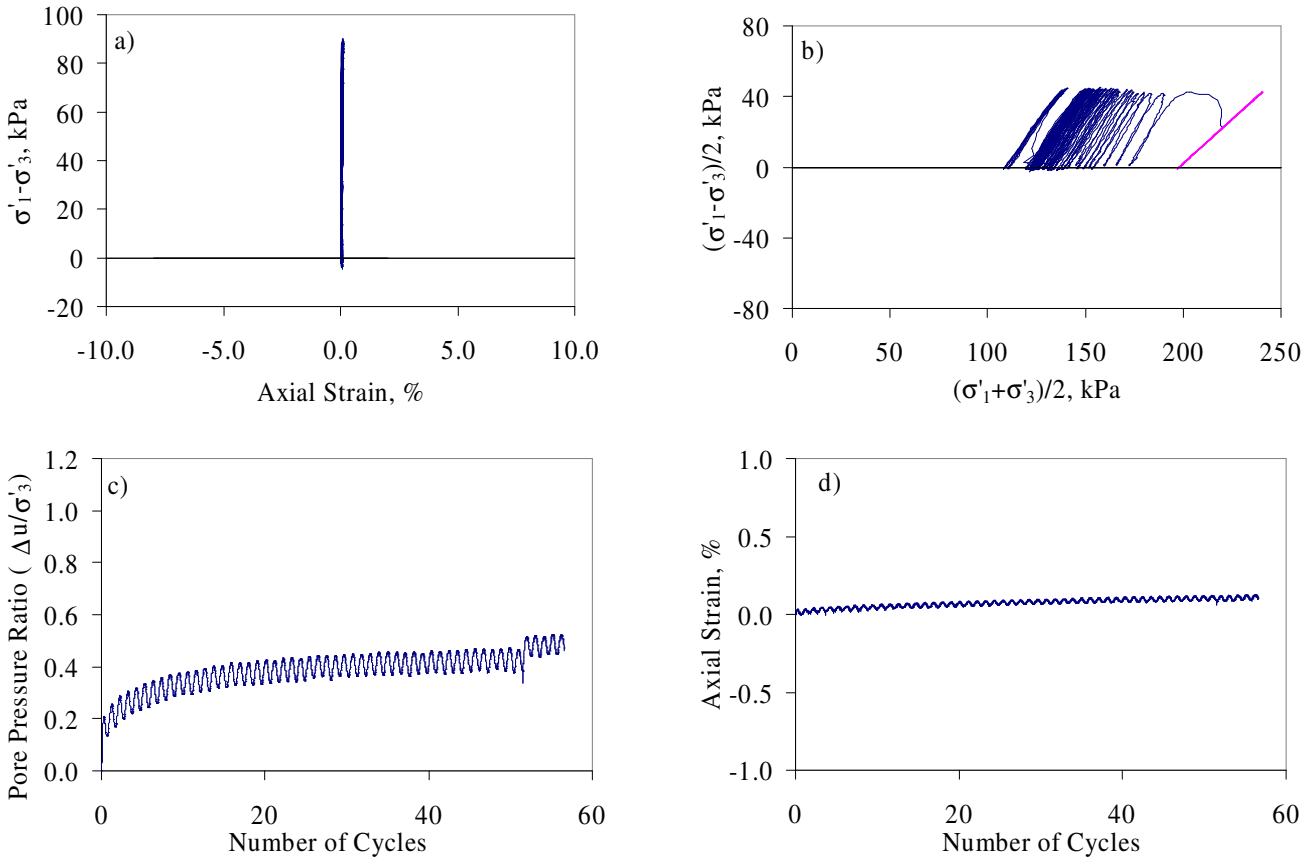


Figure A1.13. Cyclic shear response of paste rock material with static bias (α = 0.109)-Test MSCT200-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 198.0 kPa; CSR= 0.114; e_{Rock} = 0.734).

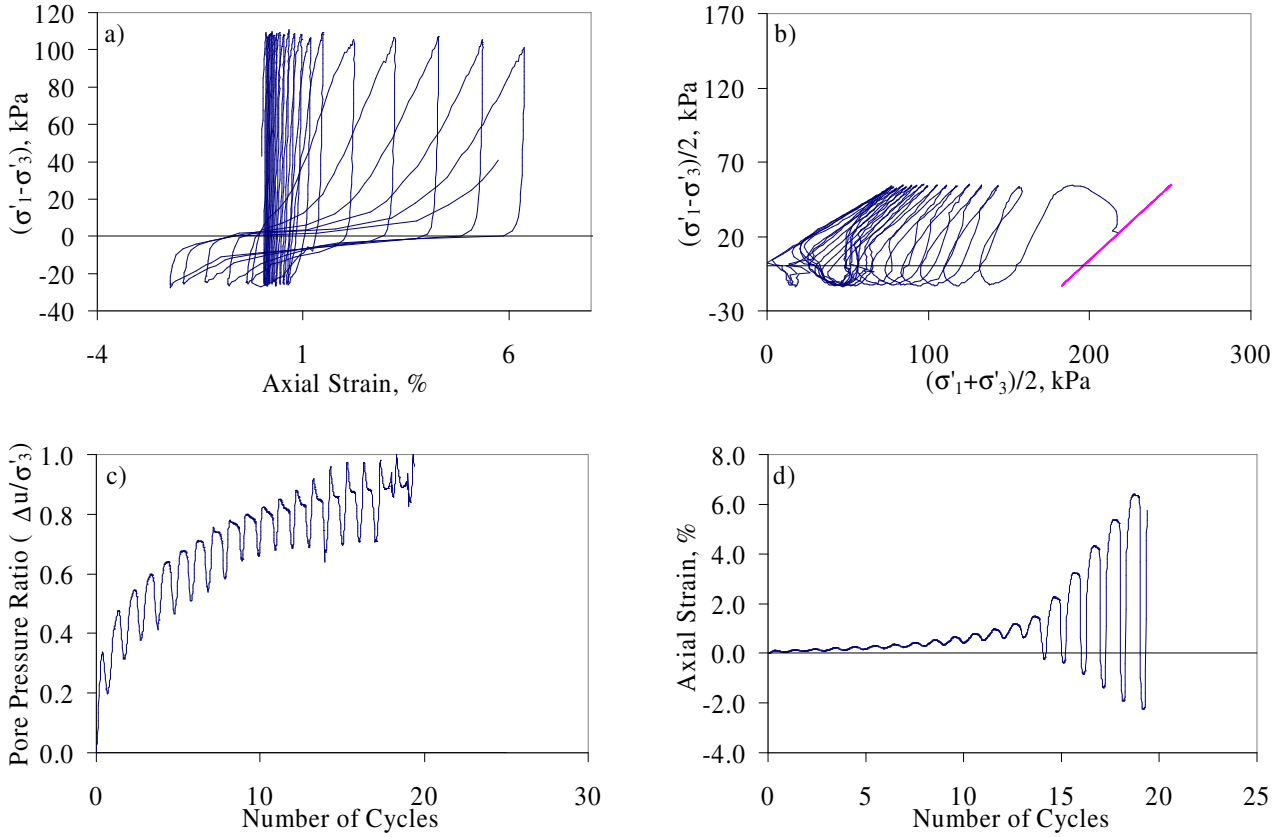


Figure A1.14. Cyclic shear response of paste rock material with static bias (α = 0.105)-Test MSCT200-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 195.7 kPa; CSR= 0.171; e_{Rock} = 0.696).

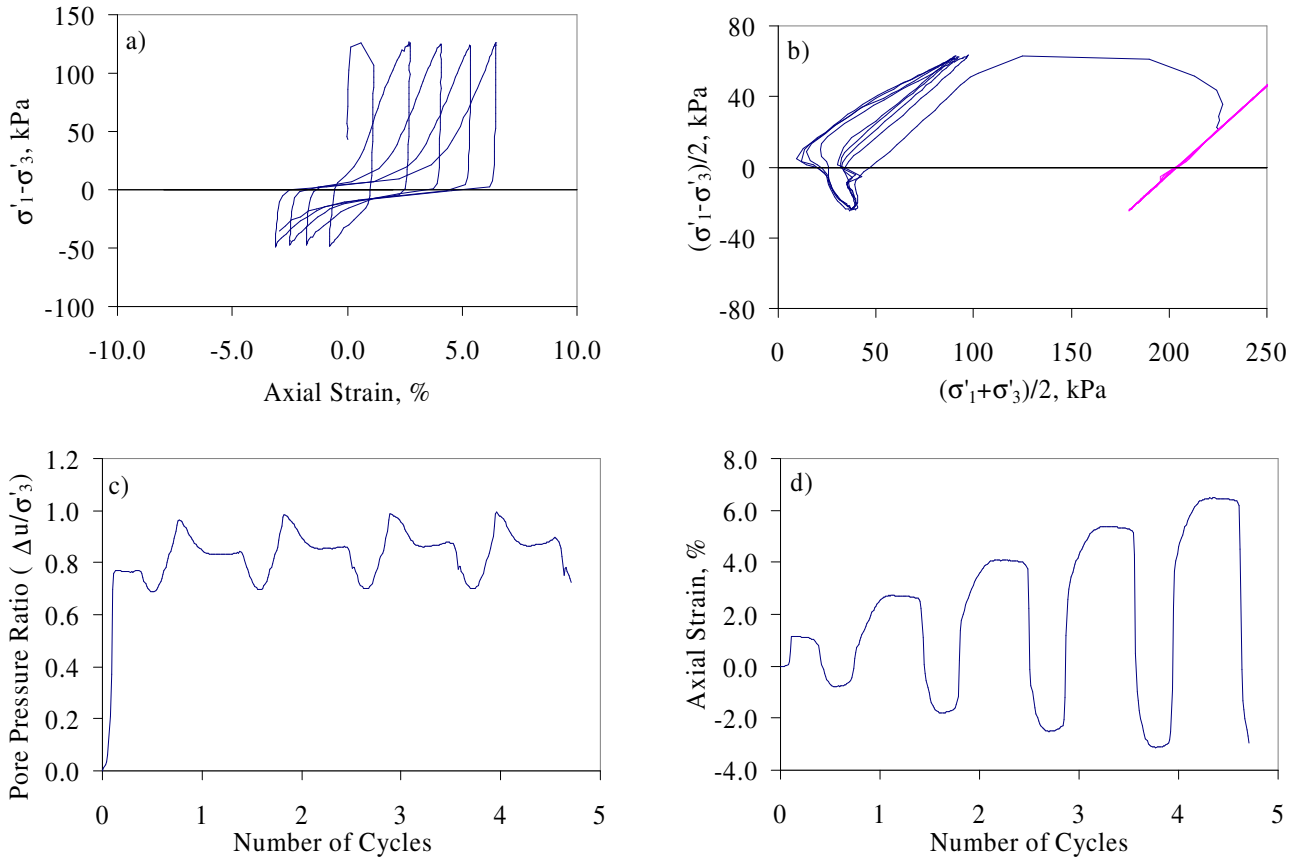


Figure A1.15. Cyclic shear response of paste rock material with static bias (α = 0.096)-Test MSCT200-3; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 204.0 kPa; CSR= 0.213; e_{Rock} = 0.773).

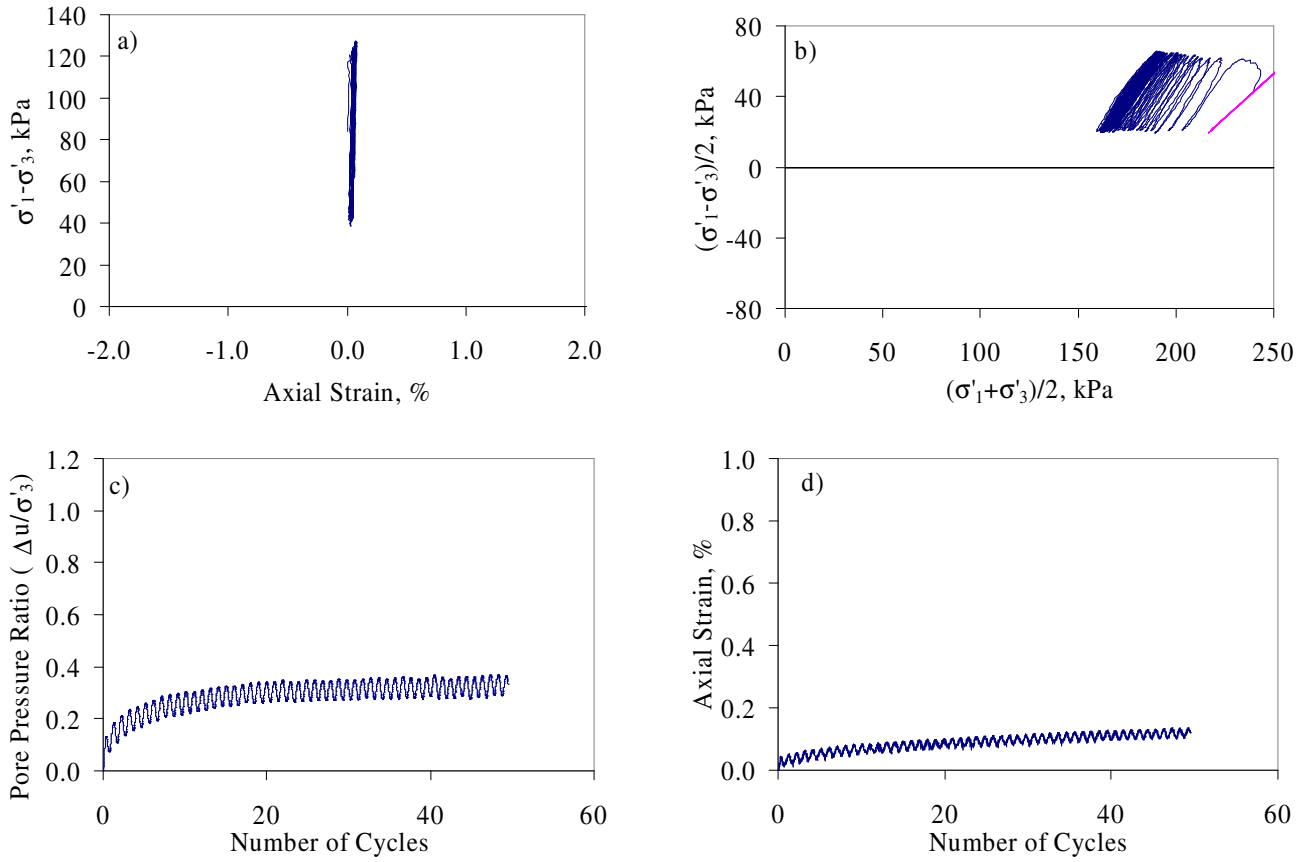


Figure A1.16. Cyclic shear response of paste rock material with static bias (α = 0.178)-Test MSCT200-4; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 197.0 kPa; CSR= 0.114; e_{Rock} = 0.719).

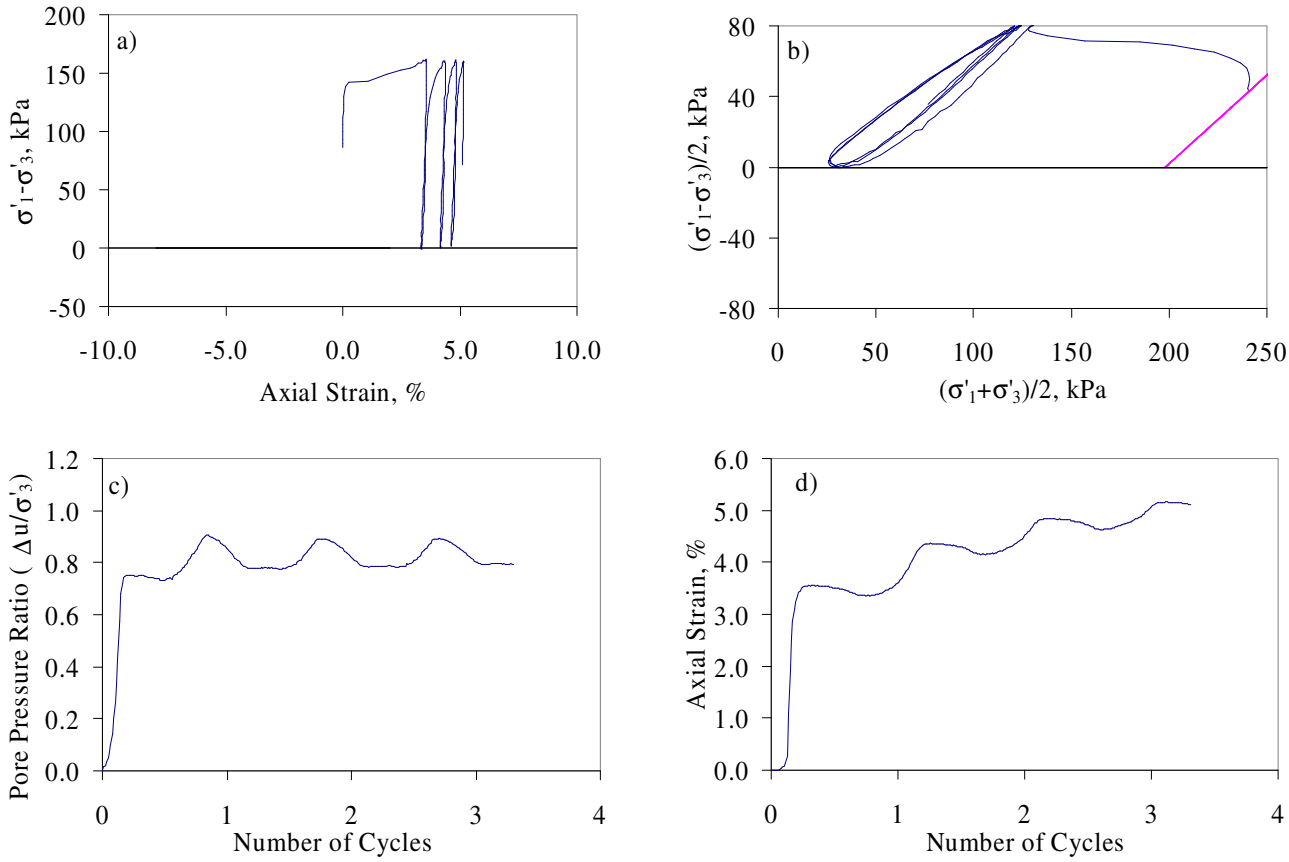


Figure A1.17. Cyclic shear response of paste rock material with static bias (α = 0.179)-Test MSCT200-5; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 198.0 kPa; CSR= 0.202; e_{Rock} = 0.733).

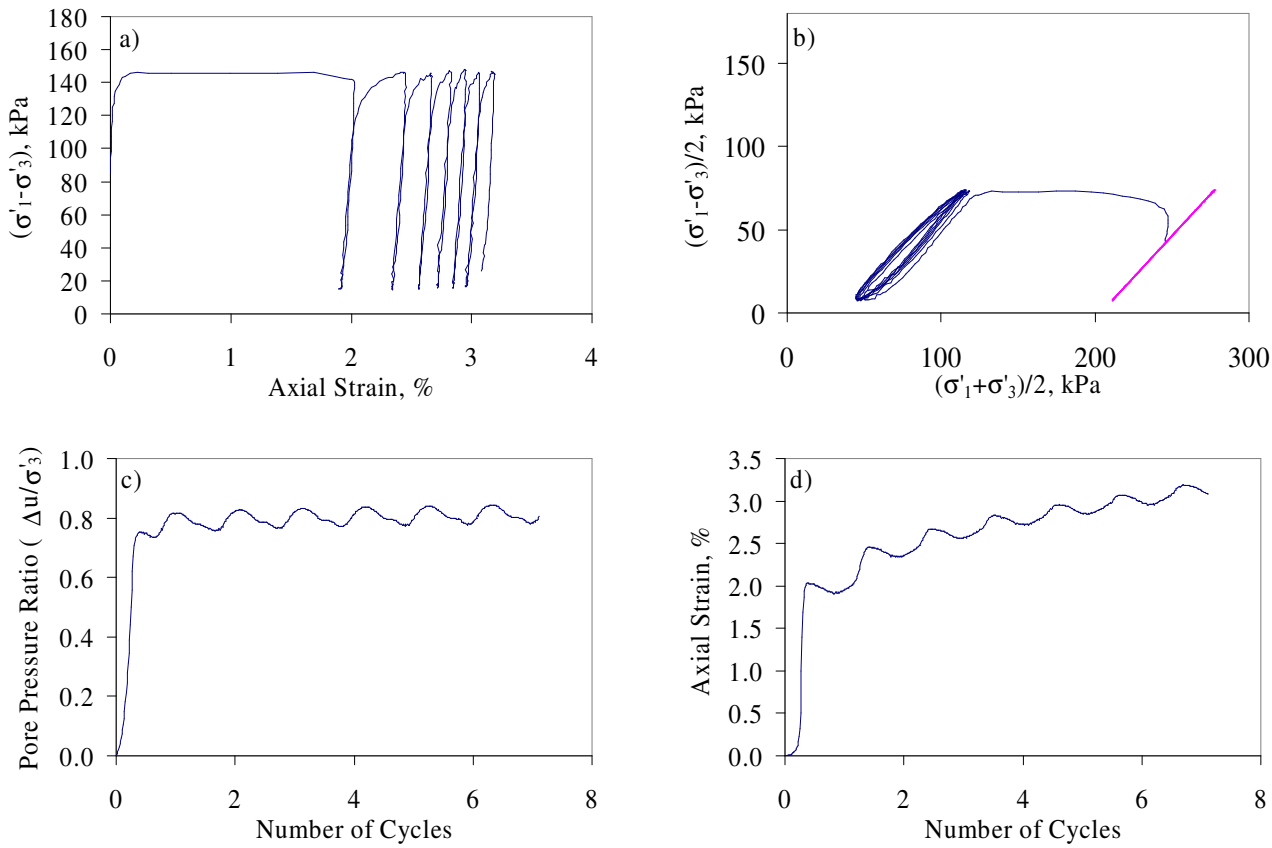


Figure A1.18. Cyclic shear response of paste rock material with static bias (α = 0.177)-Test MSCT200-6; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 203.8 kPa; CSR= 0.162; e_{Rock} = 0.695).

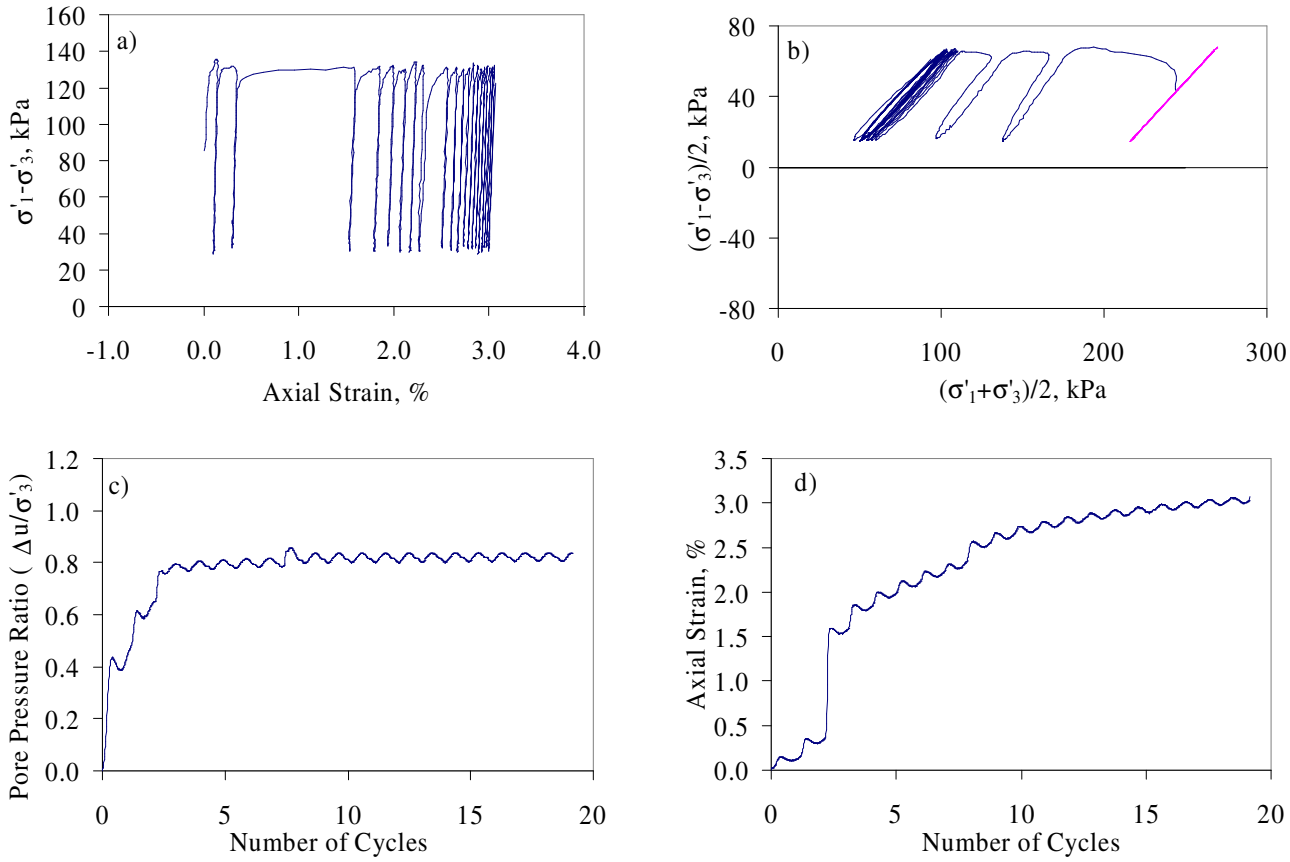


Figure A1.19. Cyclic shear response of paste rock material with static bias (α = 0.177)-Test MSCT200-7; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 201.5 kPa; CSR= 0.127; e_{Rock} = 0.766).

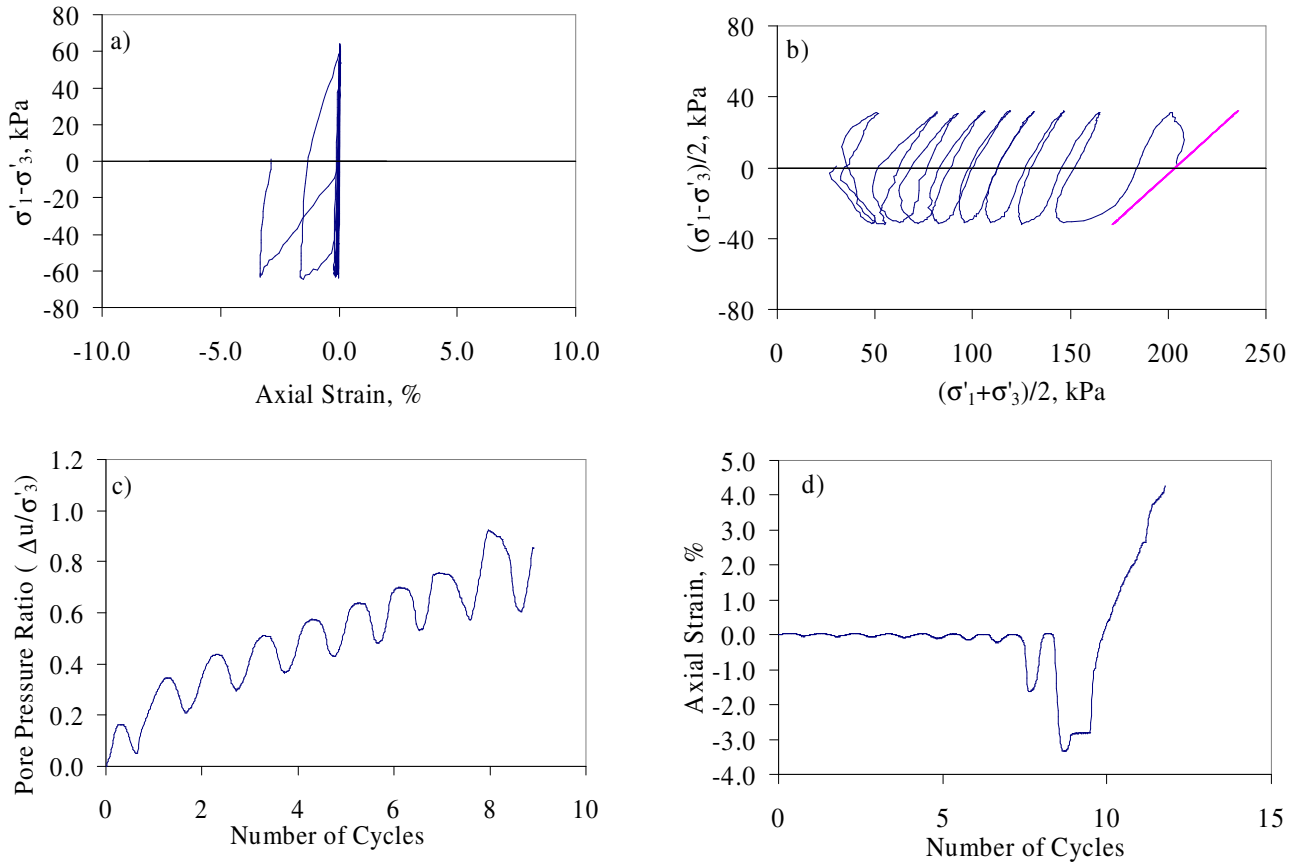


Figure A1.20. Cyclic shear response of paste rock material- Test MPCT200-1; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 203.6 kPa; CSR= 0.157; e_{Rock} = 0.709). (refer to Figure A1.21 for post cyclic shear part).

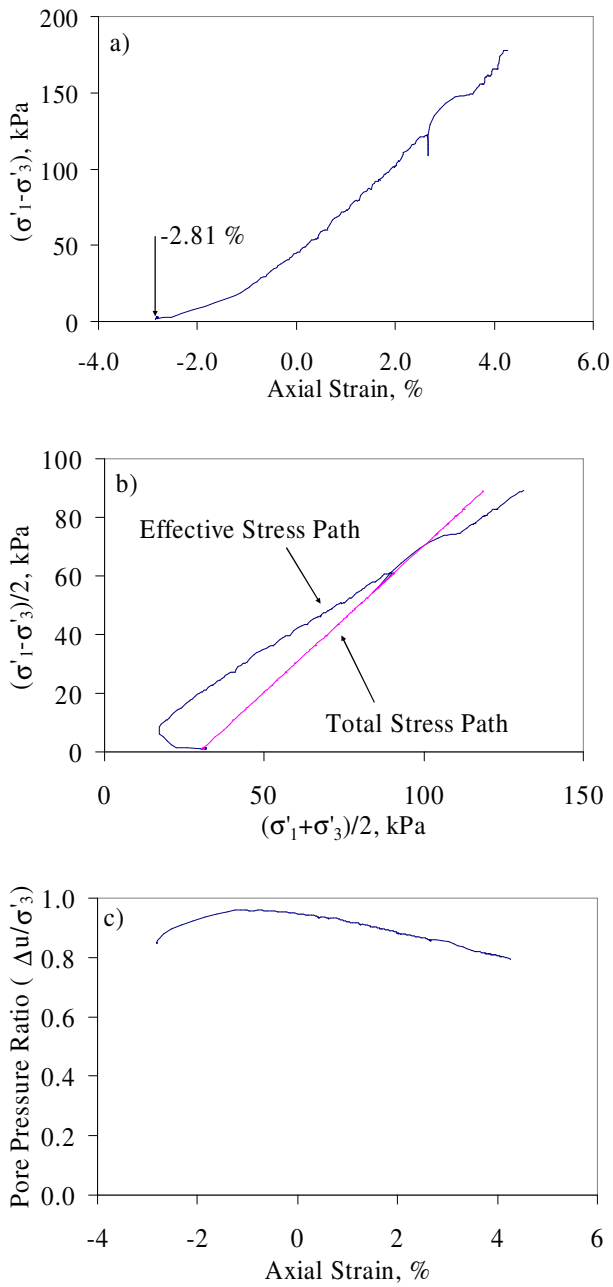


Figure A1.21. Post-cyclic shear test on paste rock specimen (MPCT200-1) after reaching 2.5% axial strain during cyclic shear – (a) Stress-strain response (b) Stress path (c) Excess Pore Water Pressure. (refer to Figure A1.20 for cyclic part).

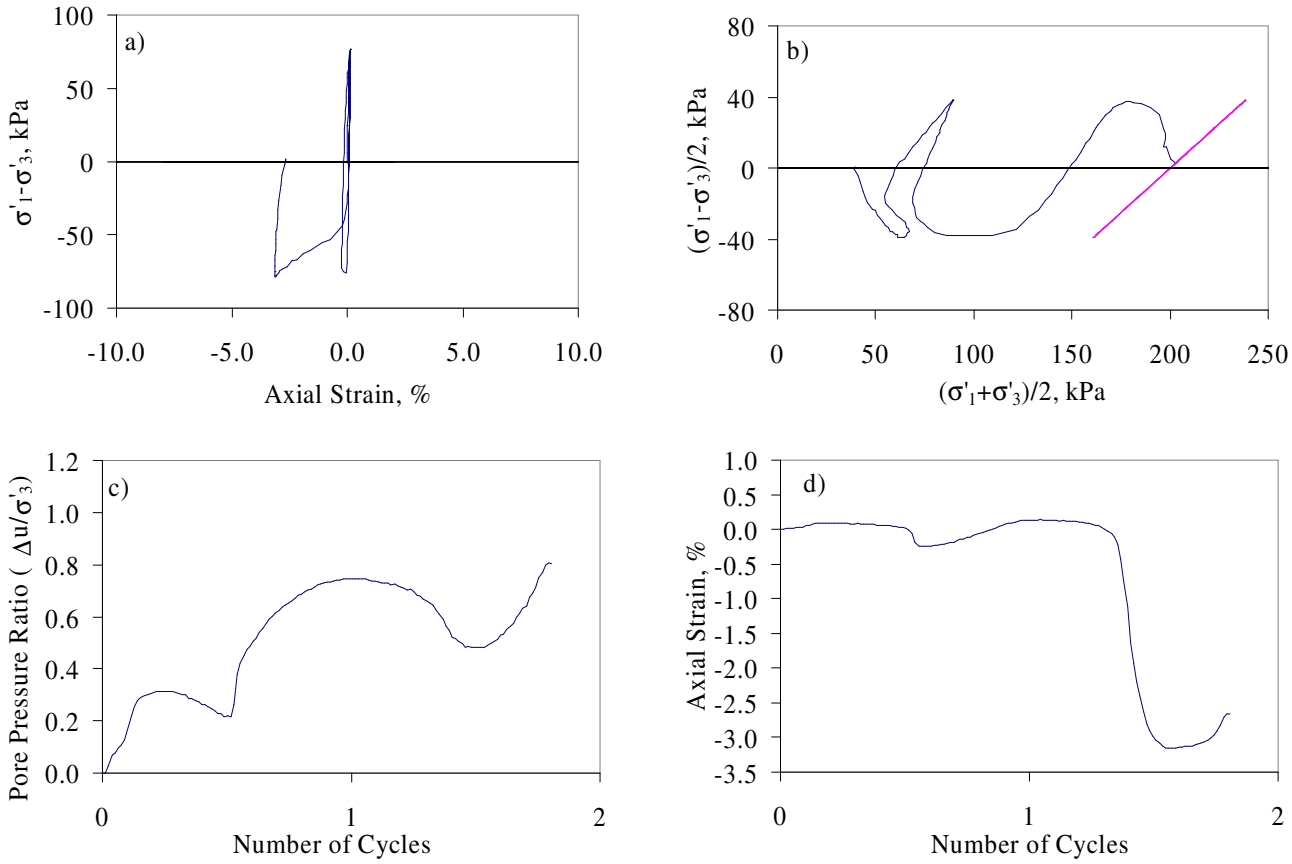


Figure A1.22. Cyclic shear response of paste rock material- Test MPCT200-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 200.2 kPa; CSR= 0.190; e_{Rock} = 0.832). (refer to Figure A1.23 for post cyclic shear part).

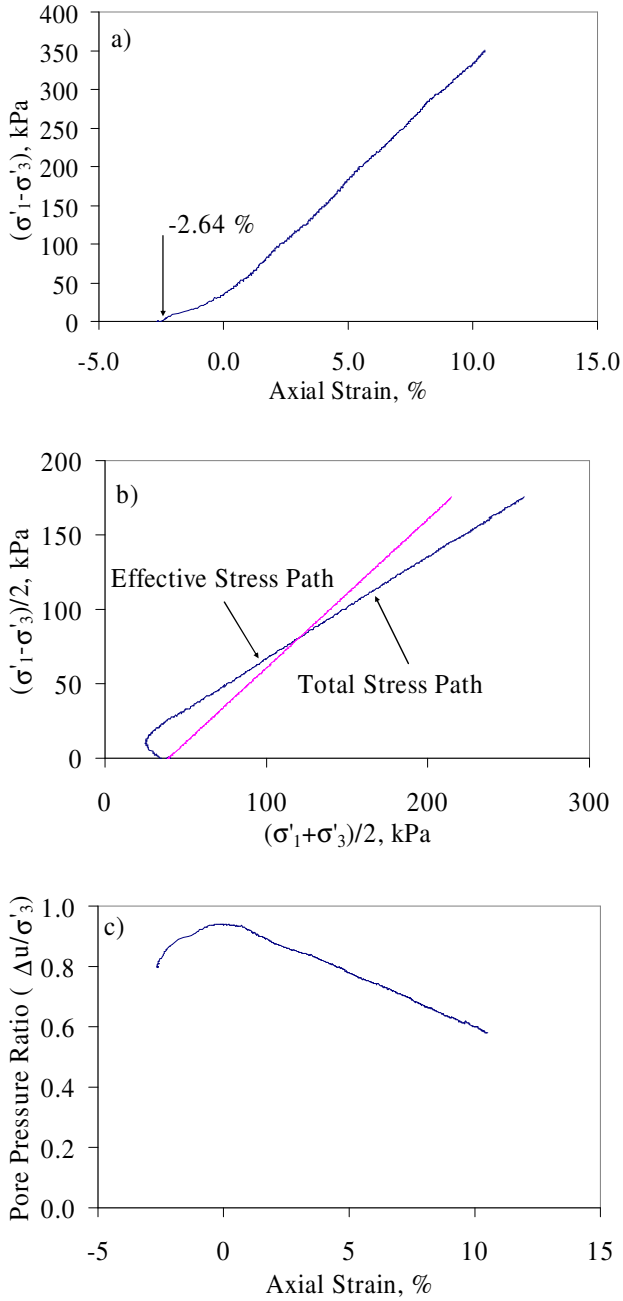


Figure A1.23. Post-cyclic shear test on paste rock specimen (MPCT200-2) after reaching 2.5% axial strain during cyclic shear – (a) Stress-strain response (b) Stress path (c) Excess Pore Water Pressure. (refer to Figure A1.22 for cyclic part).

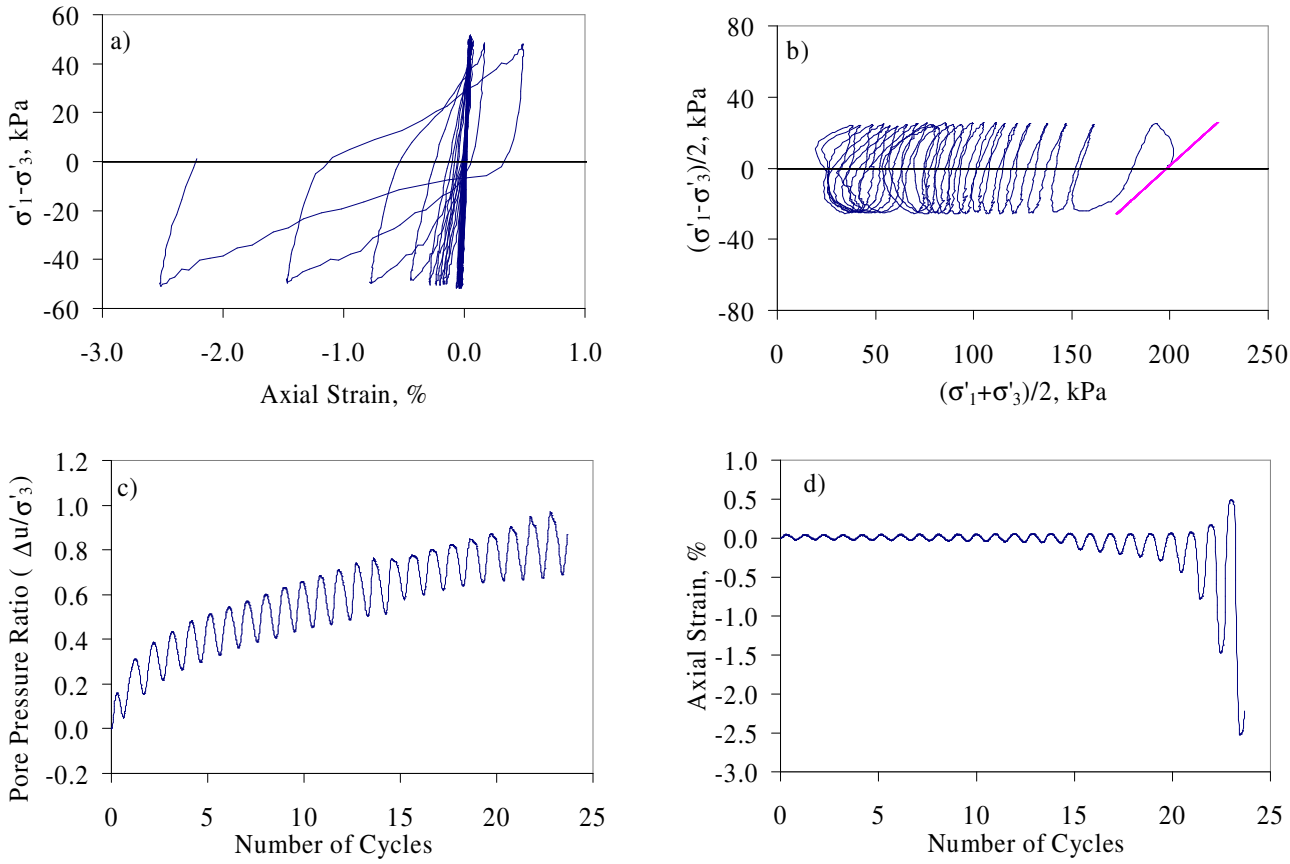


Figure A1.24. Cyclic shear response of paste rock material- Test MPCT200-3; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 198.5 kPa; CSR= 0.126; e_{Rock} = 0.744). (refer to Figure A1.25 for post cyclic shear part).

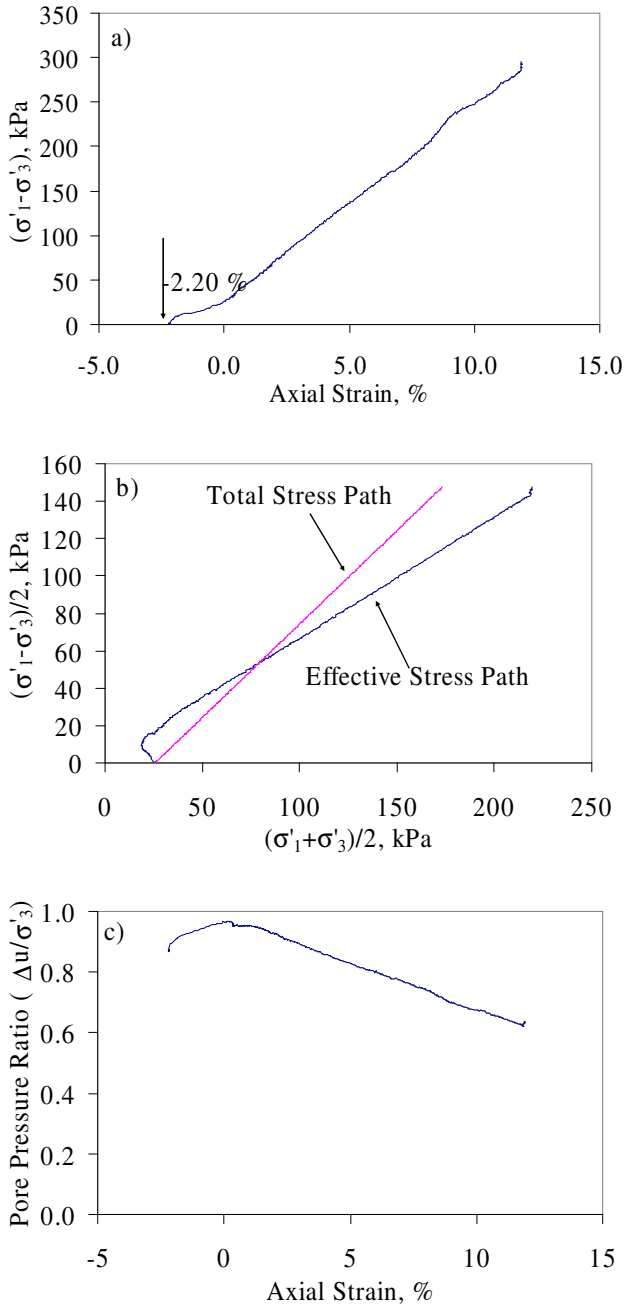


Figure A1.25. Post-cyclic shear test on paste rock specimen (MPCT200-3) after reaching 2.5% axial strain during cyclic shear – (a) Stress-strain response (b) Stress path (c) Excess Pore Water Pressure. (refer to Figure A1.24 for cyclic part).

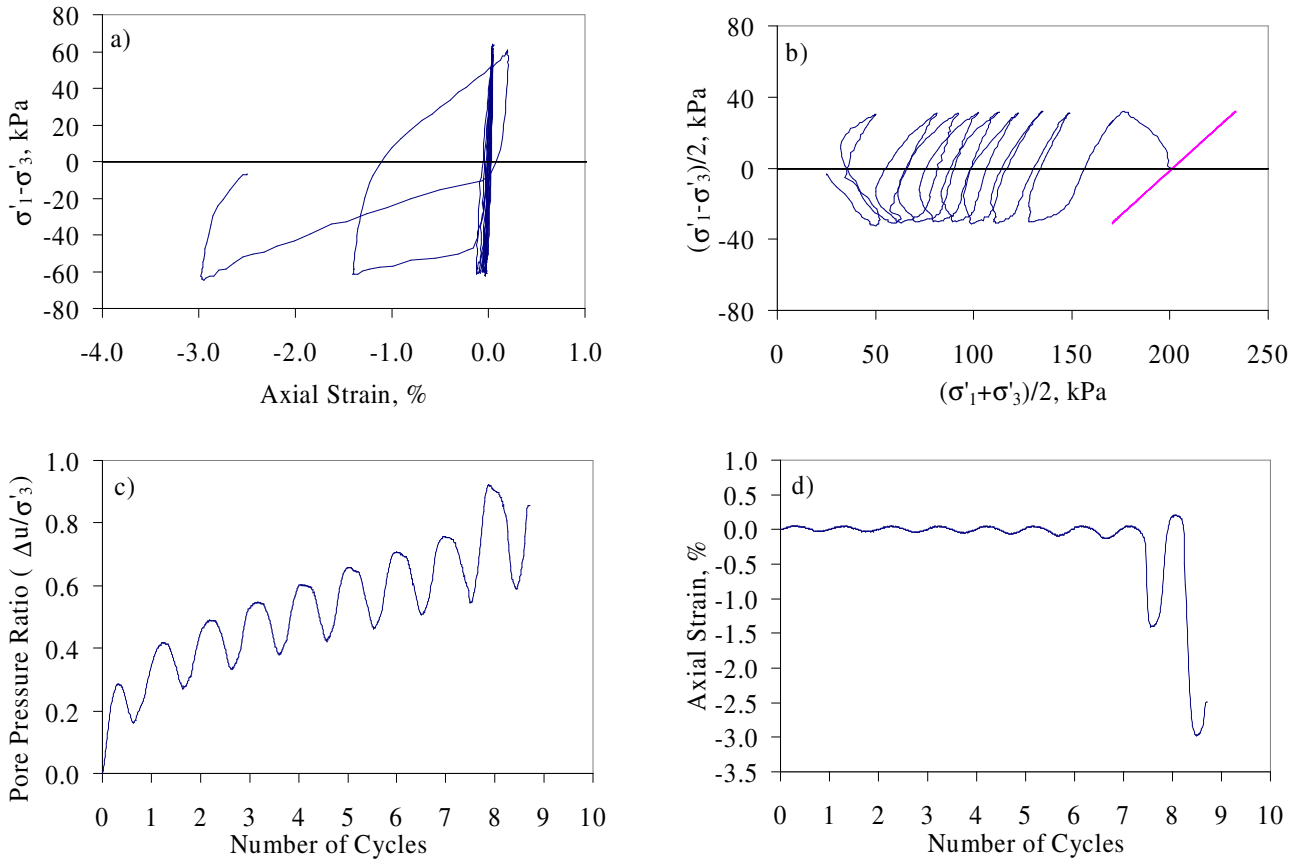


Figure A1.26. Cyclic shear response of paste rock material- Test MPCT200-4; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 201.4 kPa; CSR= 0.154; e_{Rock} = 0.739). (refer to Figure A1.27 for post cyclic shear part).

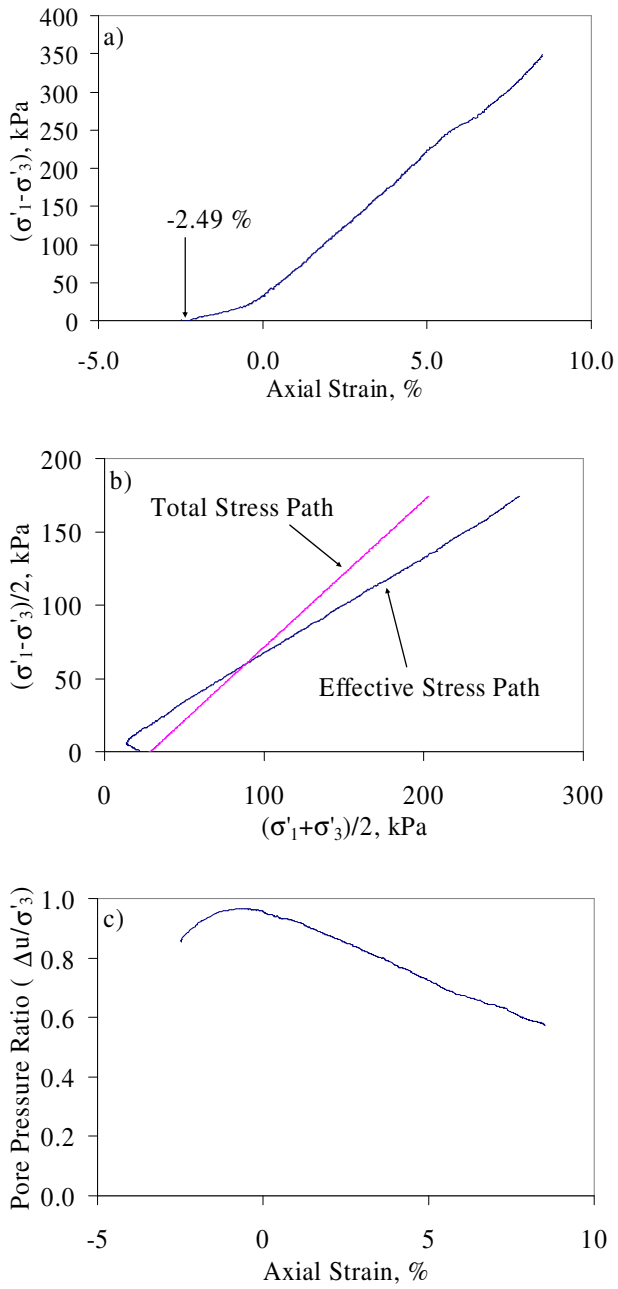


Figure A1.27. Typical post-cyclic shear test on paste rock specimen (MPCT200-4) after reaching 2.5% axial strain during cyclic shear – a) Stress-strain response b) Stress path c) Excess Pore Water Pressure. (refer to Figure A1.26 for cyclic part).

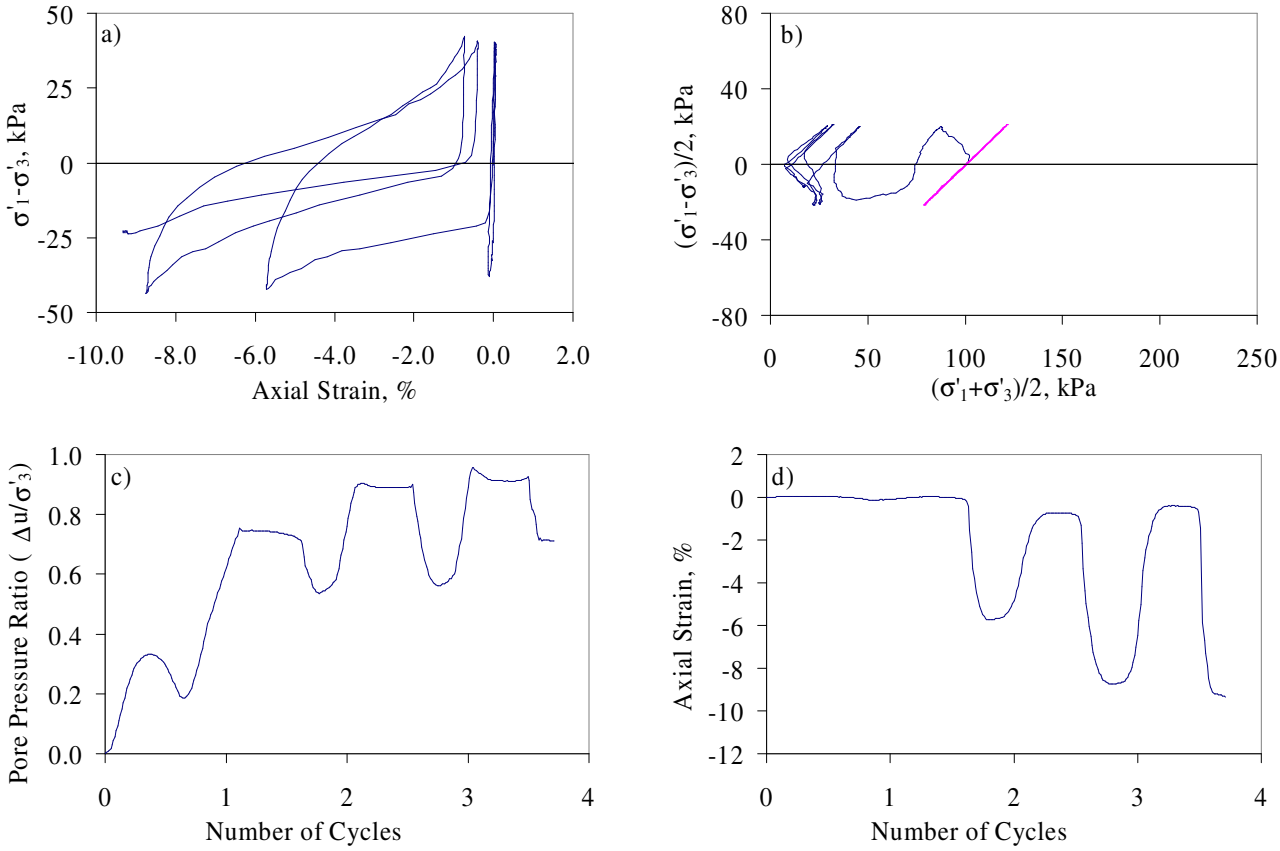


Figure A1.28. Cyclic shear response of rock-only material - Test RCT100-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 100.4 kPa; CSR= 0.199; e_{Rock} = 0.790).

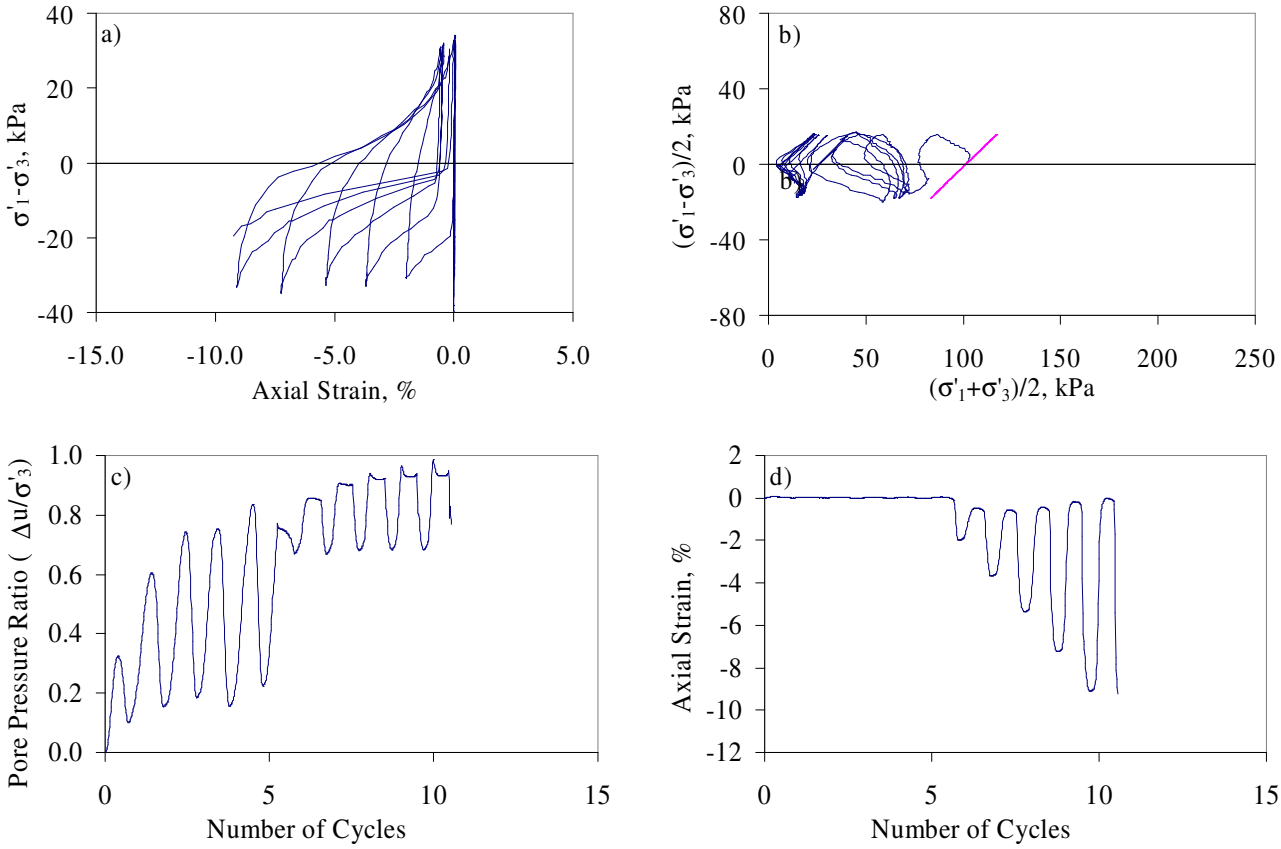


Figure A1.29. Cyclic shear response of rock-only material - Test RCT100-2; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 101.2 kPa; CSR= 0.168; e_{Rock} = 0.776).

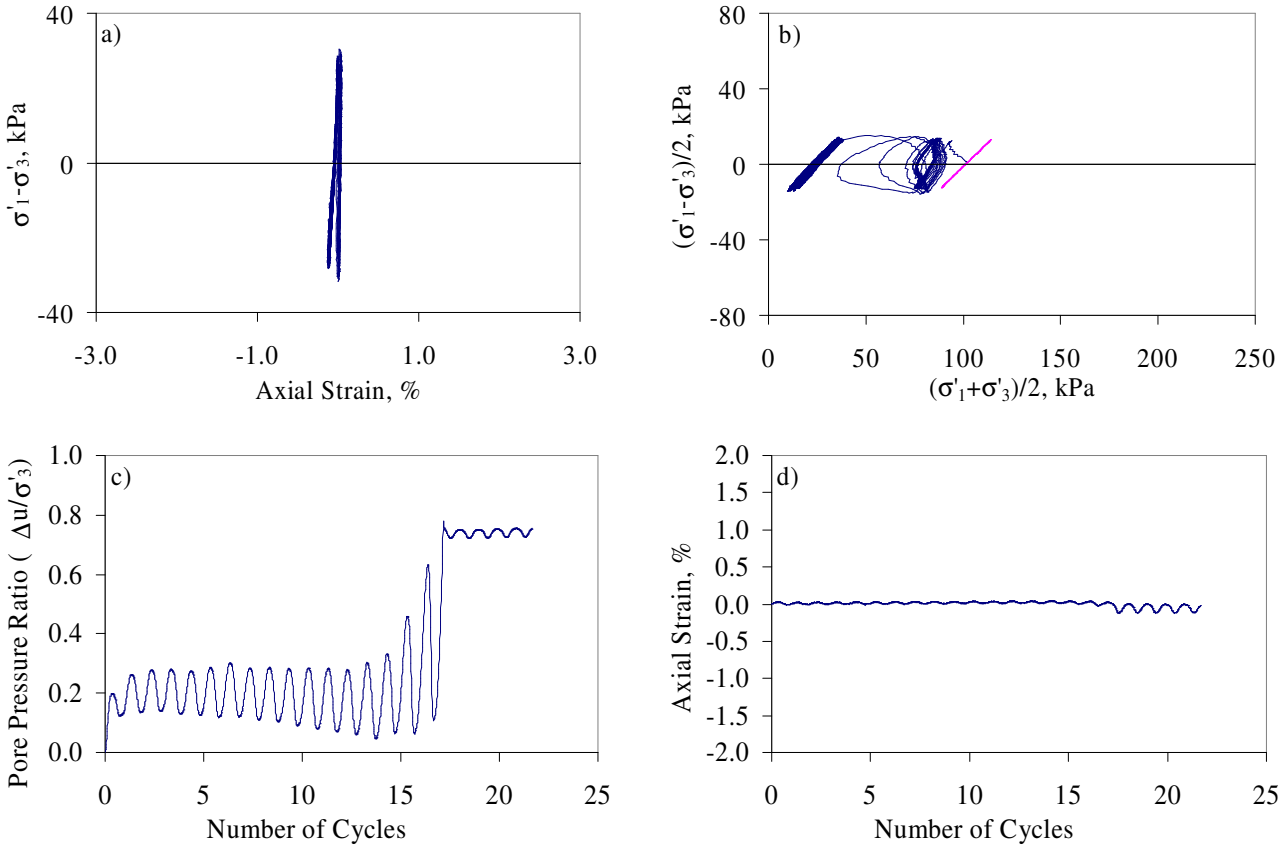


Figure A1.30. Cyclic shear response of rock-only material - Test RCT100-3; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 101.1 kPa; CSR= 0.143; e_{Rock} = 0.797).

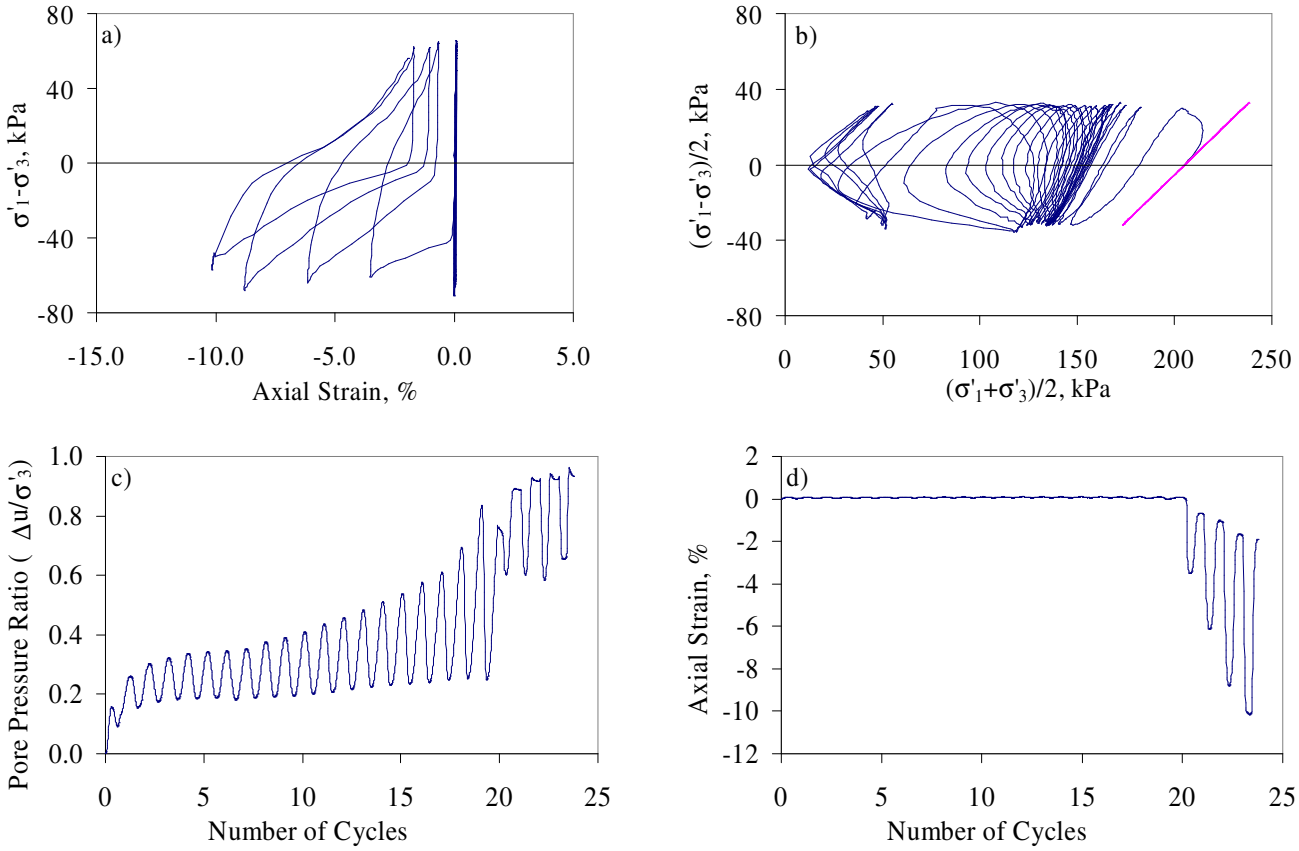


Figure A1.31. Cyclic shear response of rock-only material - Test RCT200-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 205.3 kPa; CSR= 0.156; e_{Rock} = 0.754).

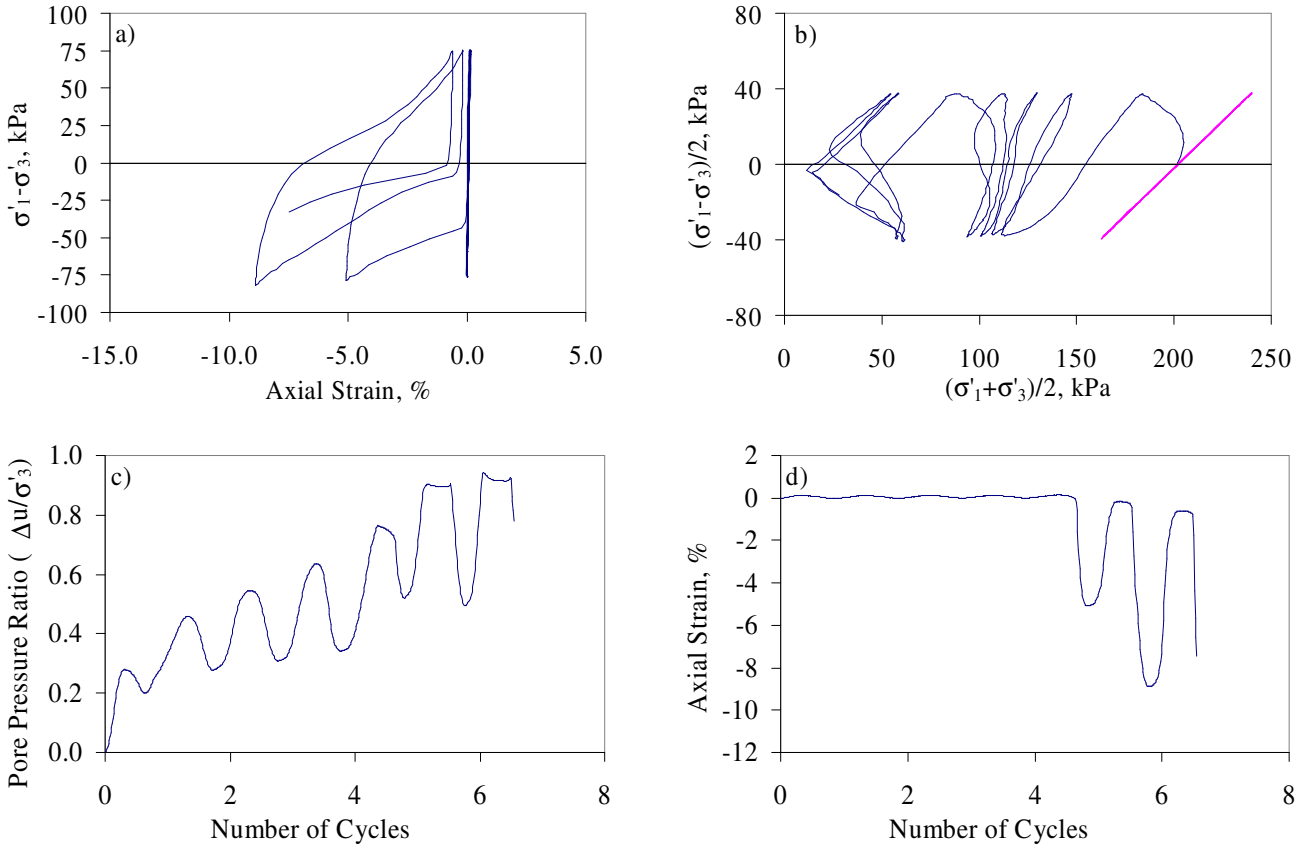


Figure A1.32. Cyclic shear response of rock-only material - Test RCT200-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 201.9 kPa; CSR= 0.188; e_{Rock} = 0.746).

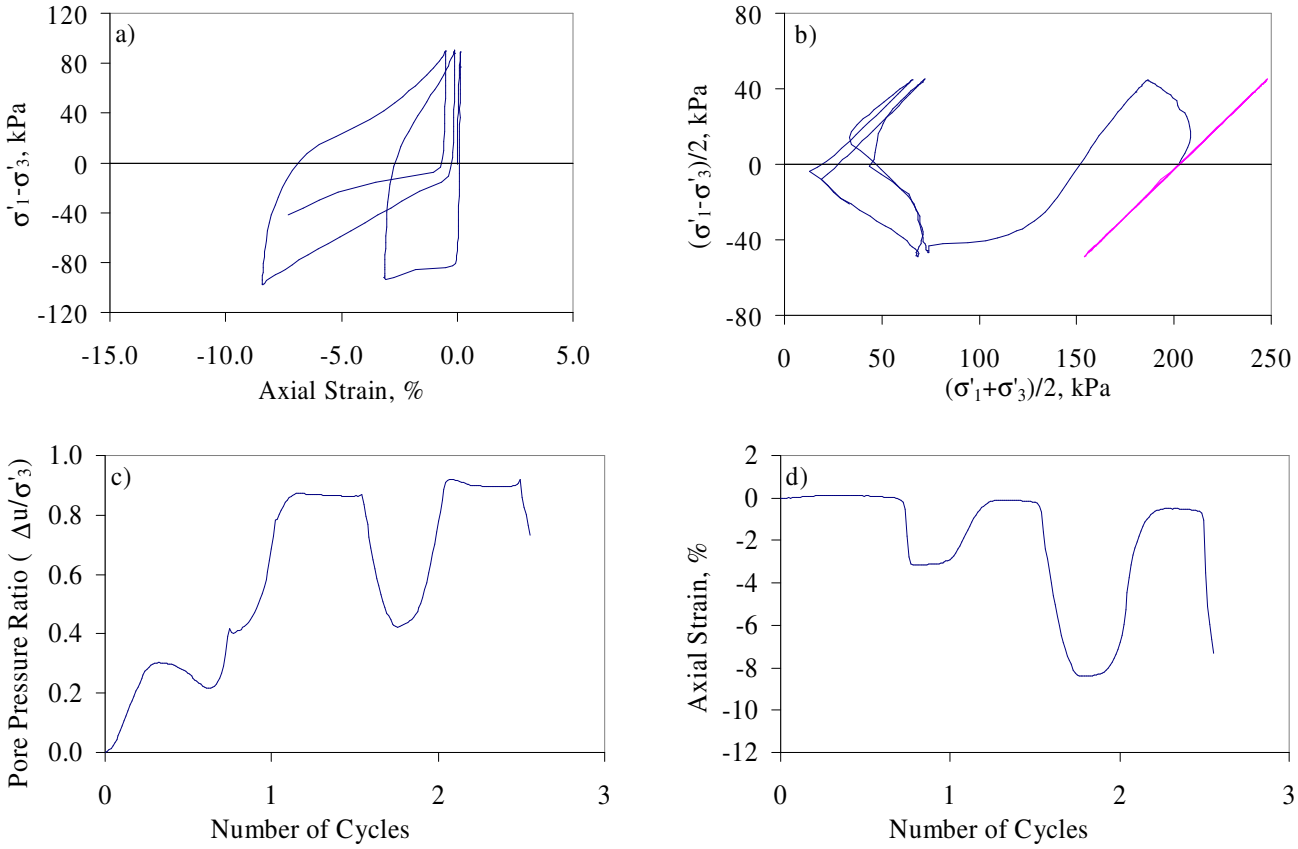


Figure A1.33. Cyclic shear response of rock-only material - Test RCT200-3; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 202.7 kPa; CSR= 0.227; e_{Rock} = 0.754).

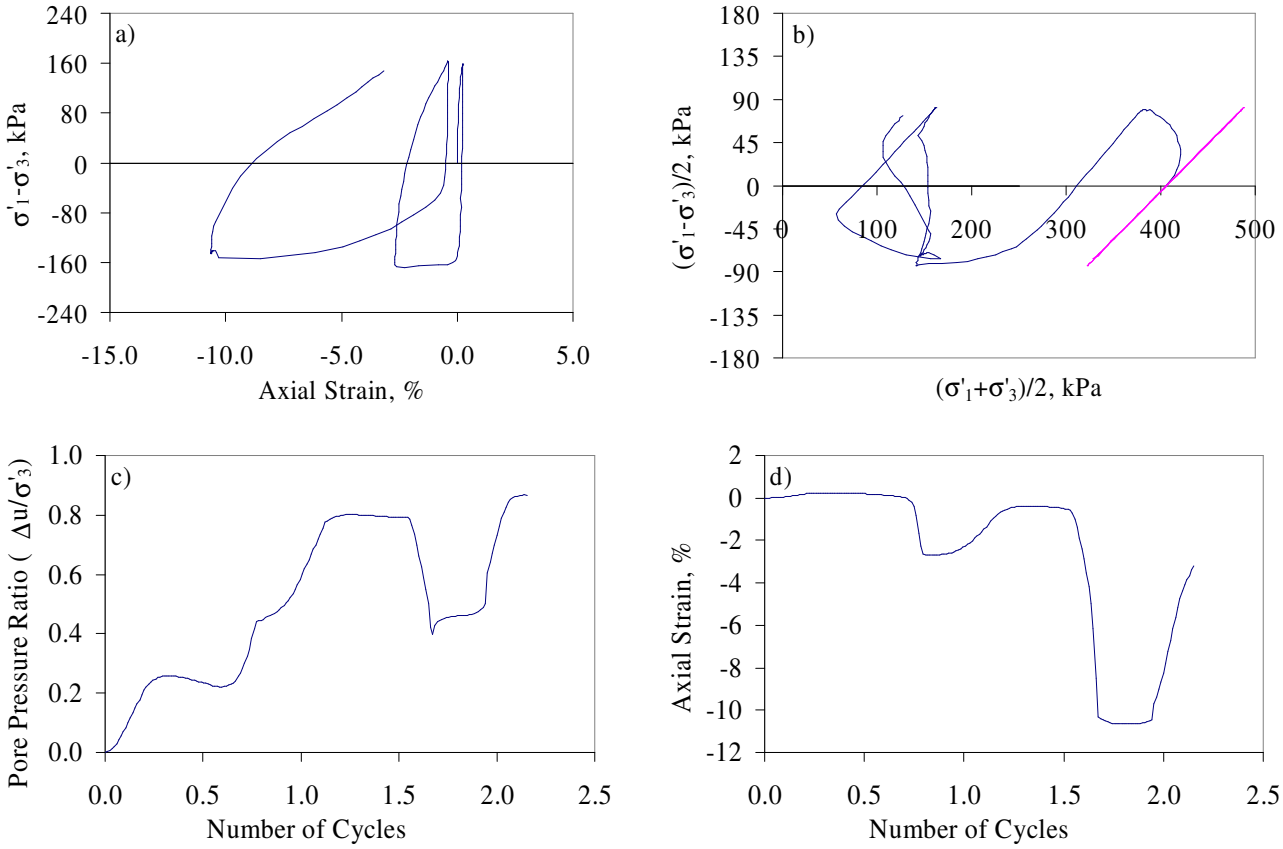


Figure A1.34. Cyclic shear response of rock-only material - Test RCT400-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 406.1 kPa; CSR= 0.199; e_{Rock} = 0.742).

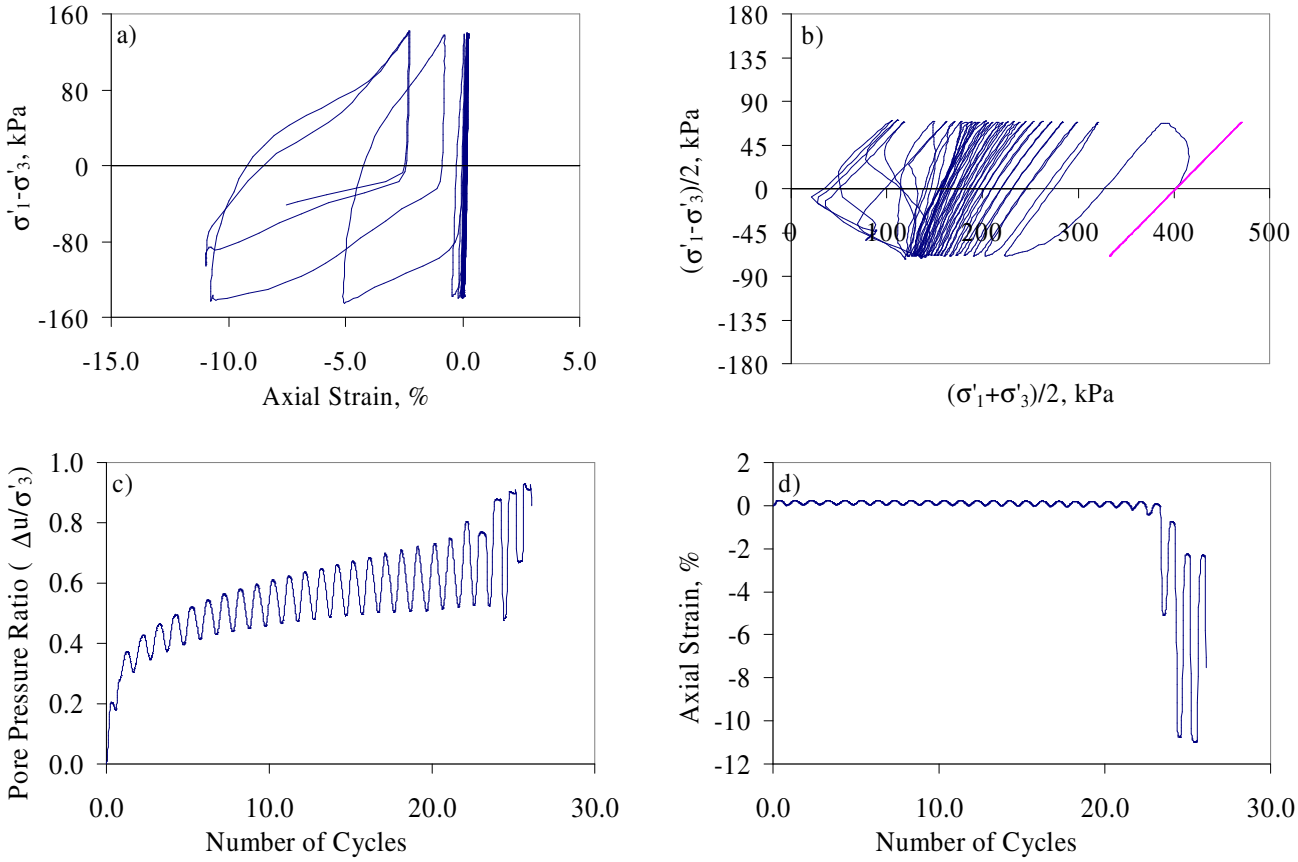


Figure A1.35. Cyclic shear response of rock-only material - Test RCT400-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 402.3 kPa; CSR= 0.172; e_{Rock} = 0.766).

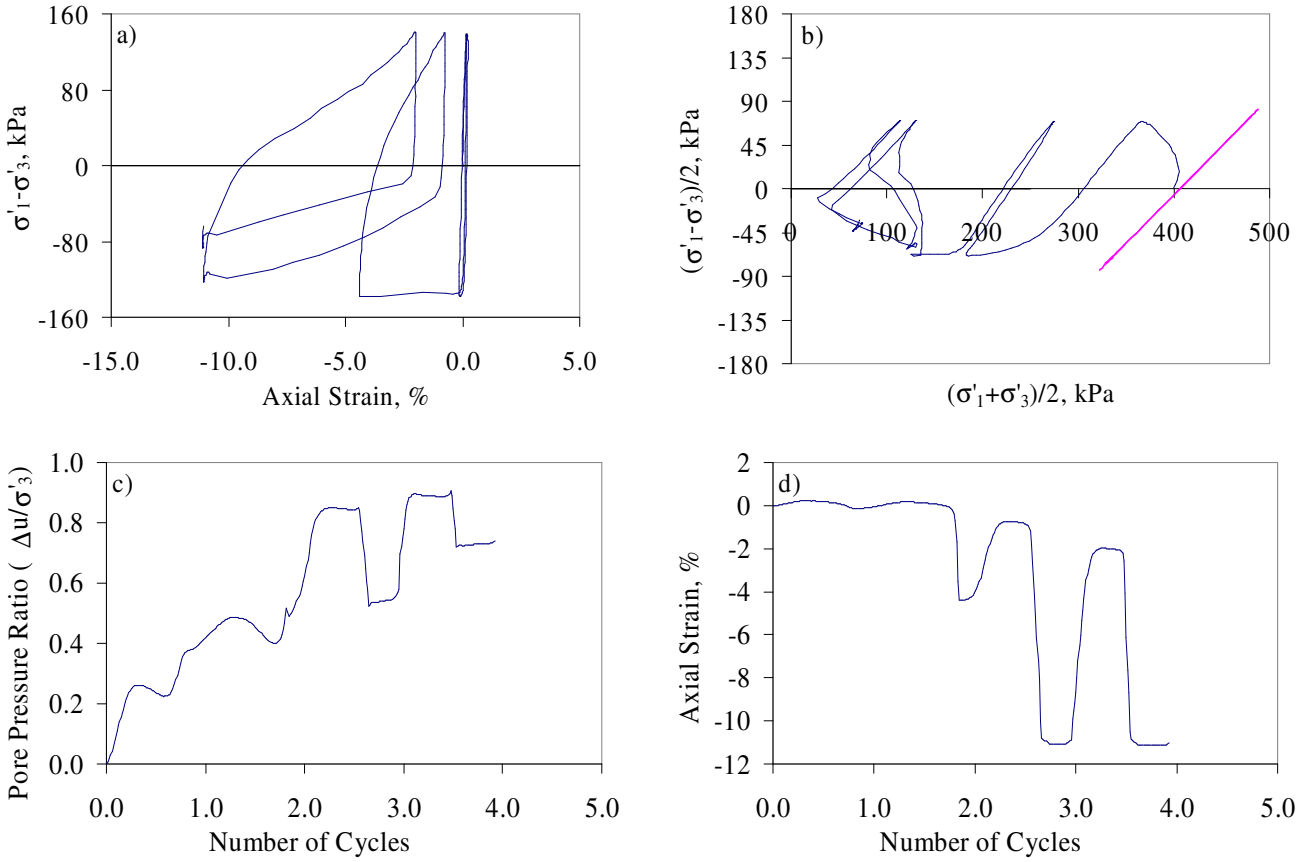


Figure A1.36. Cyclic shear response of rock-only material - Test RCT400-3; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 399.6 kPa; CSR= 0.173; e_{Rock} = 0.796).

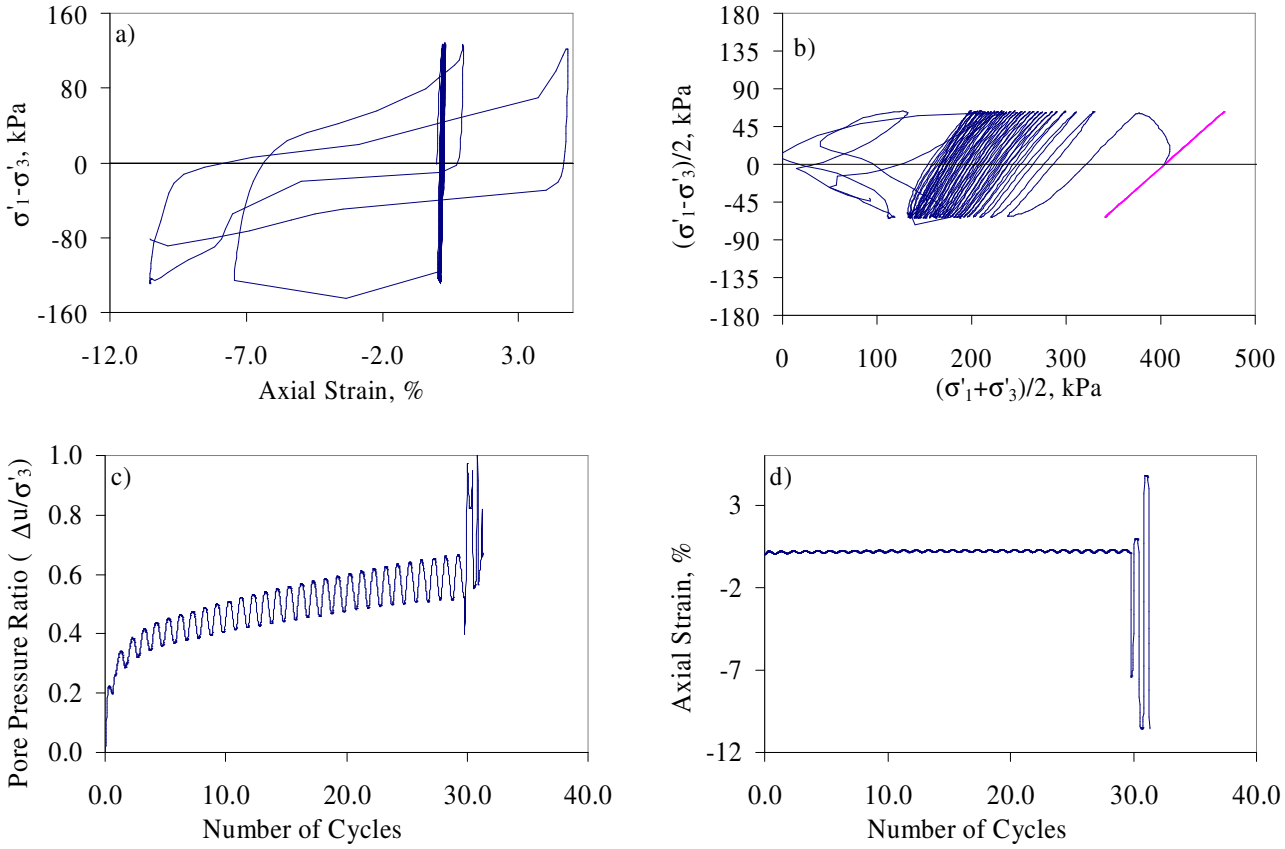


Figure A1.37. Cyclic shear response of rock-only material - Test RCT400-4; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 404.2 kPa; CSR= 0.156; e_{Rock} = 0.816).

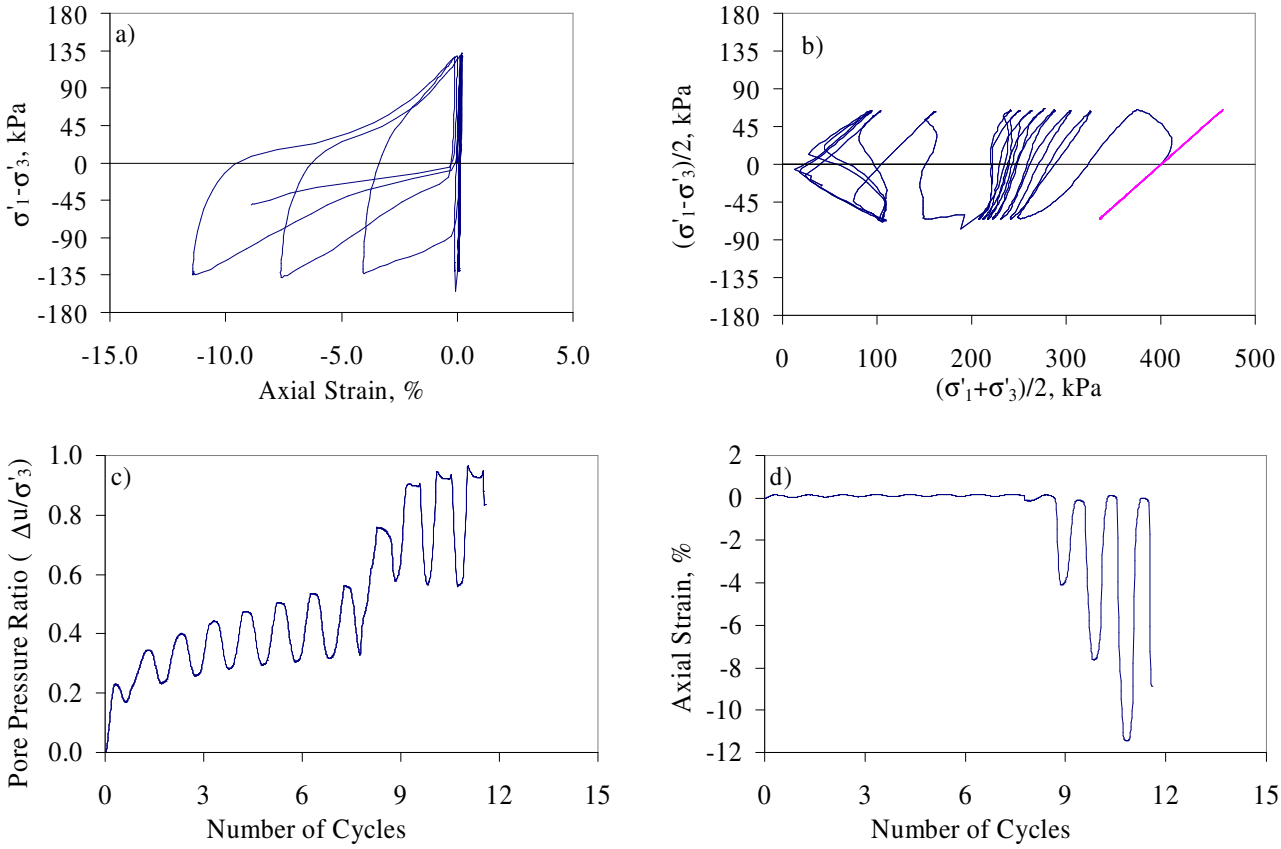


Figure A1.38. Cyclic shear response of rock-only material - Test RCT400-5; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 400.3 kPa; CSR= 0.162; e_{Rock} = 0.735).

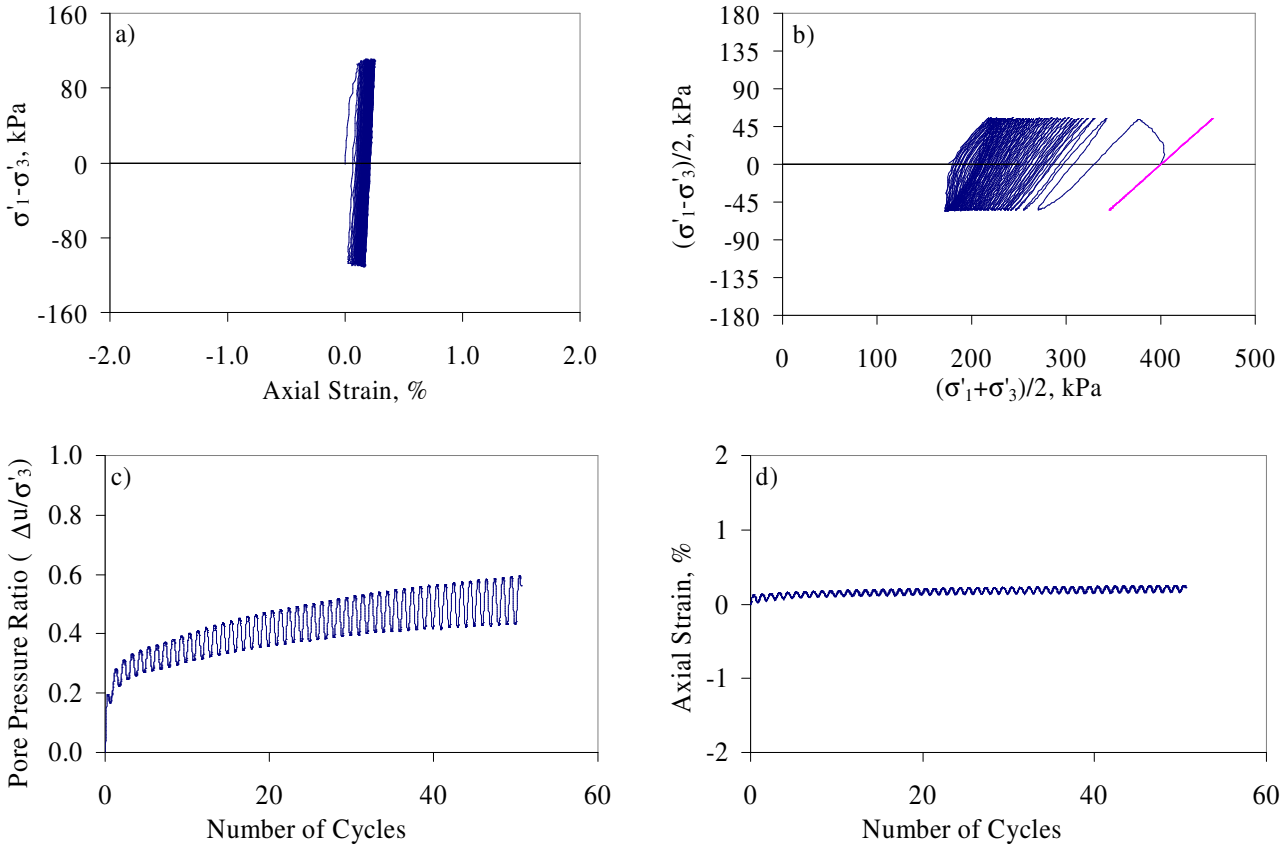


Figure A1.39. Cyclic shear response of rock-only material - Test RCT400-6; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 400.3 kPa; CSR= 0.135; e_{Rock} = 0.751).

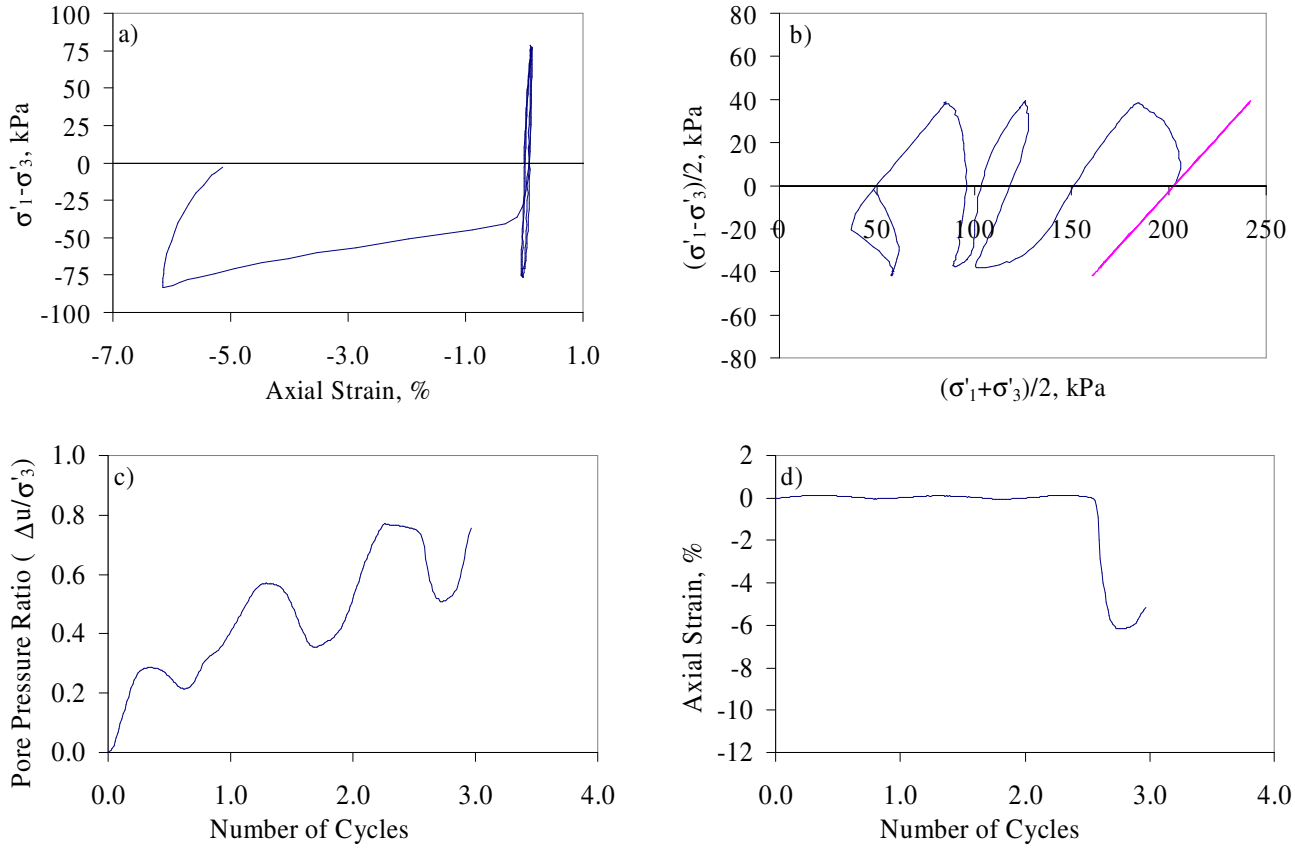


Figure A1.40. Cyclic shear response of rock-only material - Test RPCT200-1; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 202.4 kPa; CSR= 0.193; e_{Rock} = 0.779). (refer to Figure A1.41 for post cyclic shear part).

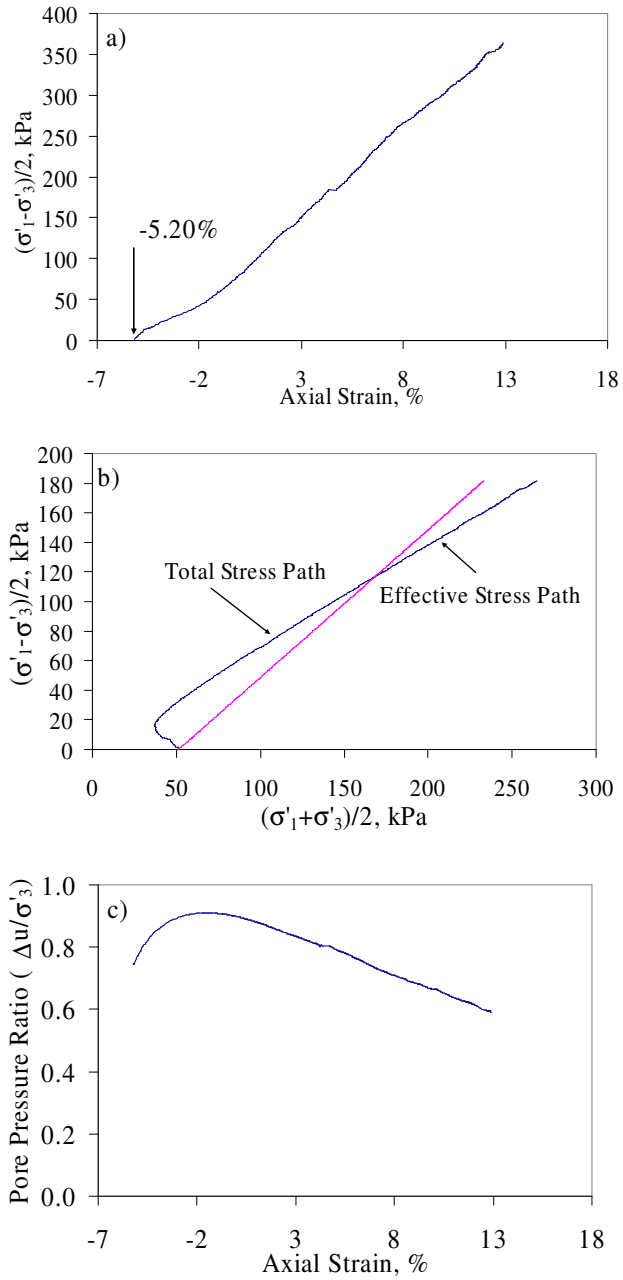


Figure A1.41. Post-cyclic shear test on rock-only specimen (RPCT200-1) after reaching 2.5% axial strain during cyclic shear – (a) Stress-strain response (b) Stress path (c) Excess Pore Water Pressure. (refer to Figure A1.40 for cyclic part).

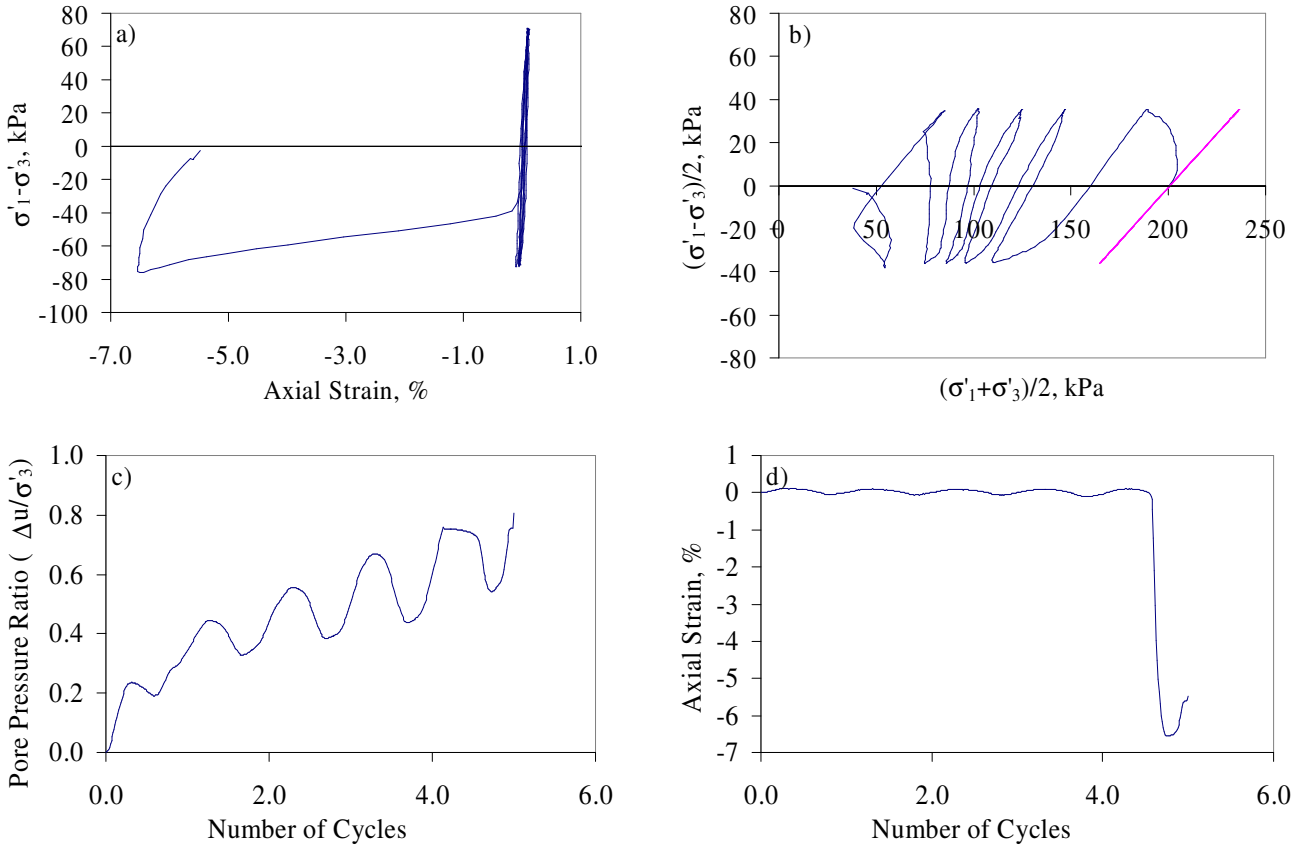


Figure A1.42. Cyclic shear response of rock-only material - Test RPCT200-2; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 200.8 kPa; CSR= 0.179; e_{Rock} = 0.807). (refer to Figure A1.43 for post cyclic shear part).

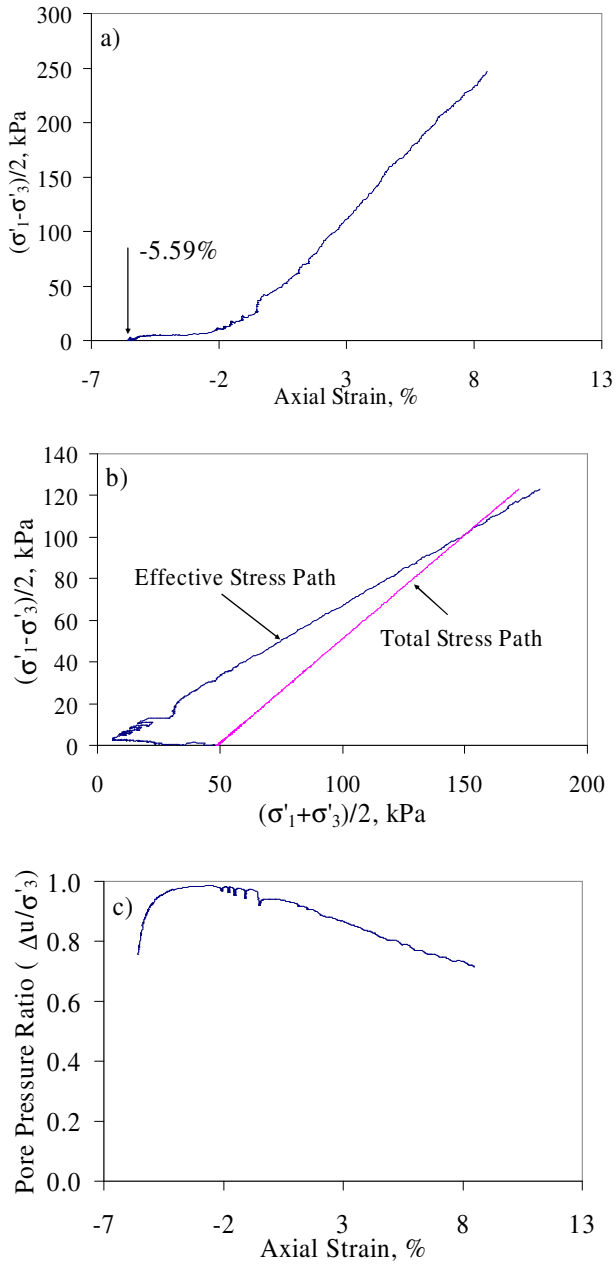


Figure A1.43. Post-cyclic shear test on rock-only specimen (RPCT200-2) after reaching 2.5% axial strain during cyclic shear – (a) Stress-strain response (b) Stress path (c) Excess Pore Water Pressure. (refer to Figure A1.42 for cyclic part).

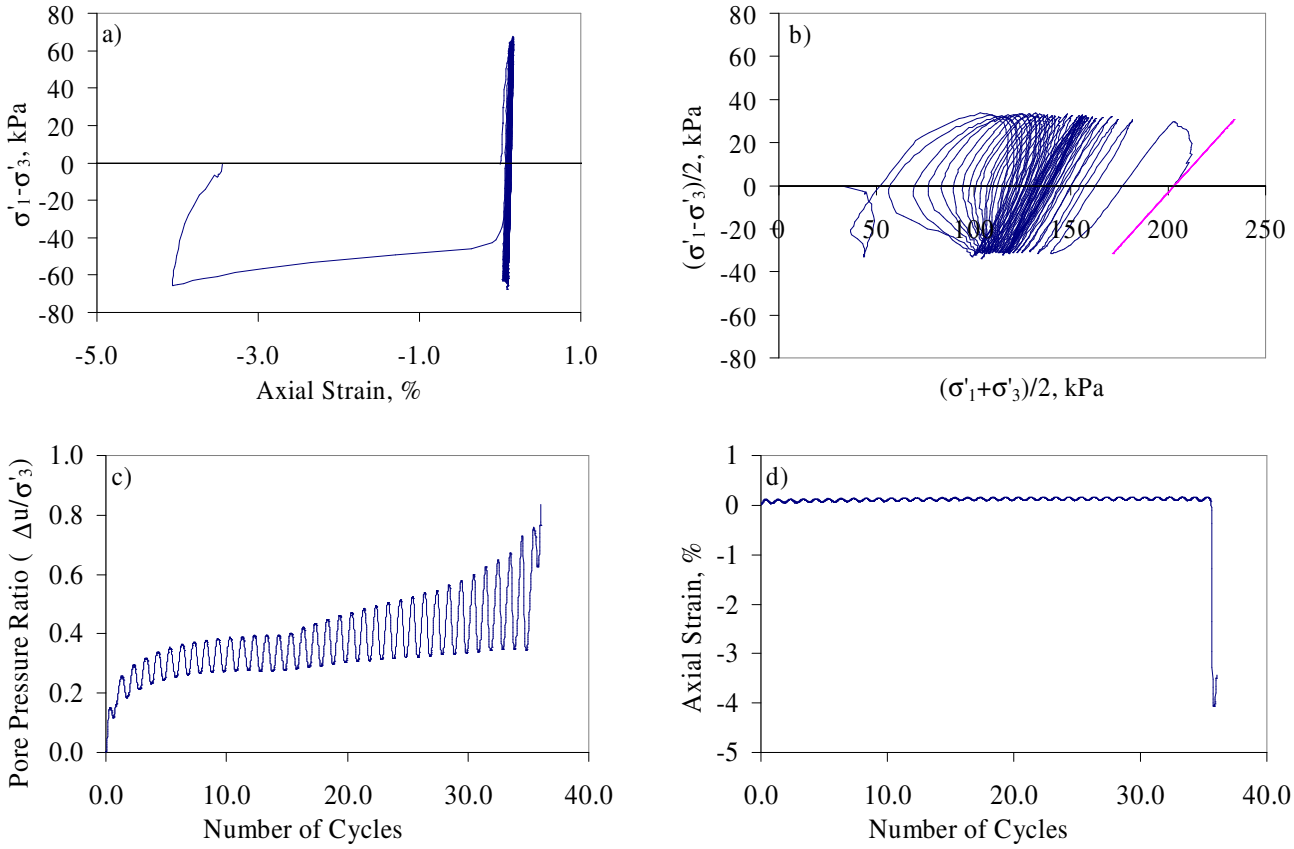


Figure A1.44. Cyclic shear response of rock-only material - Test RPCT200-3; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 203.1 kPa; CSR= 0.158; e_{Rock} = 0.799). (refer to Figure A1.45 for post cyclic shear part).

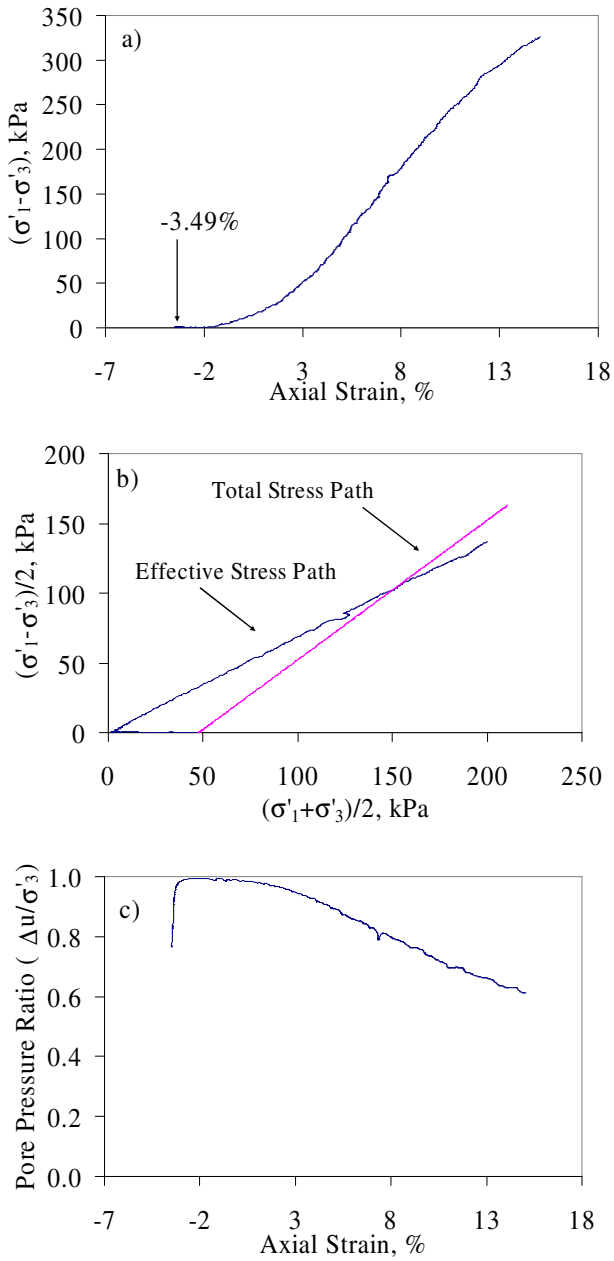


Figure A1.45. Post-cyclic shear test on rock-only specimen (RPCT200-3) after reaching 2.5% axial strain during cyclic shear – (a) Stress-strain response (b) Stress path (c) Excess Pore Water Pressure. (refer to Figure A1.44 for cyclic part).

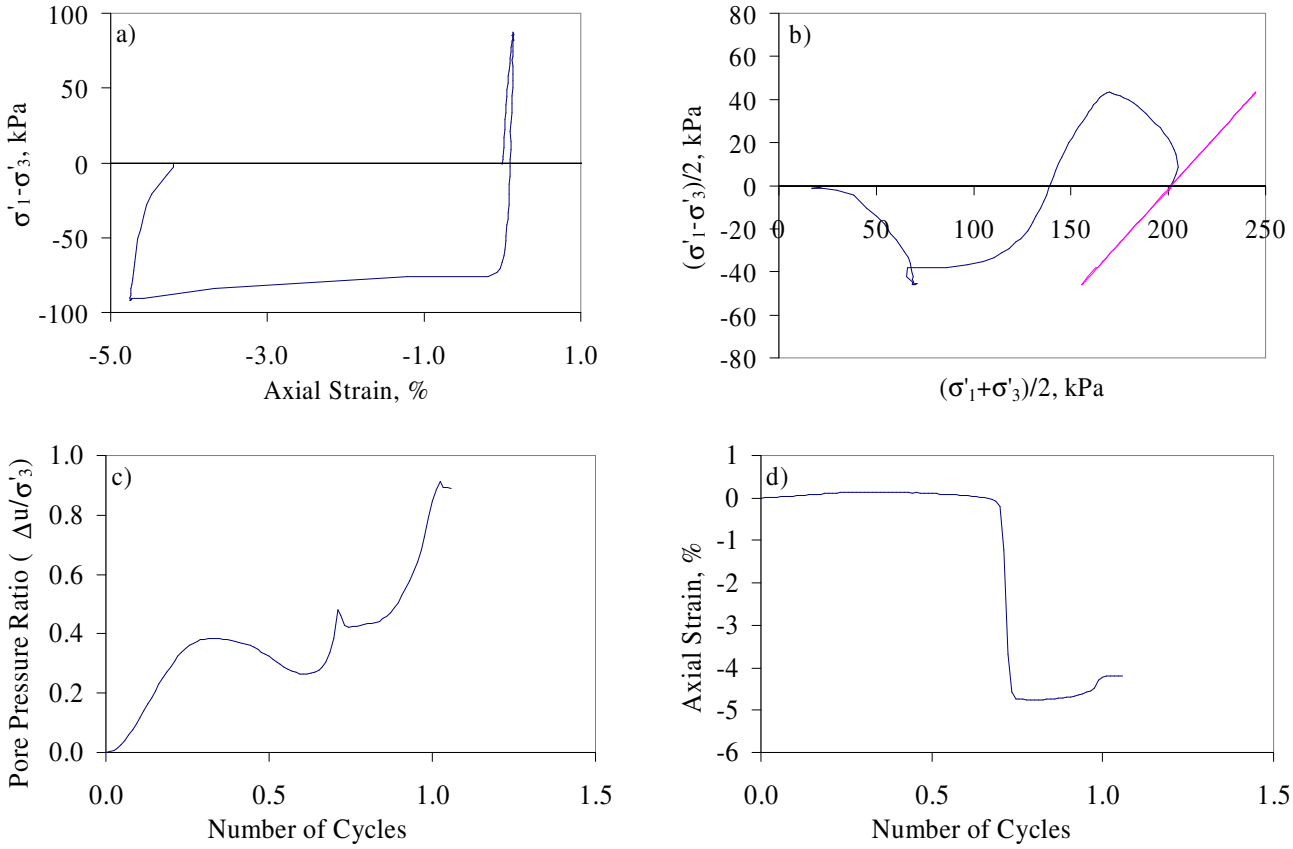


Figure A1.46. Cyclic shear response of rock-only material - Test RPCT200-4; (a) stress-strain [$(\sigma'_1-\sigma'_3)$ vs. ε_a]; (b) stress-path [$(\sigma'_1-\sigma'_3)/2$ vs. $(\sigma'_1+\sigma'_3)/2$]; (c) excess pore water pressure ratio [$r_u = (\Delta u/\sigma'_{3c})$] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (σ'_{3c} = 201.5 kPa; CSR= 0.223; e_{Rock} = 0.759). (refer to Figure A1.47 for post cyclic shear part).

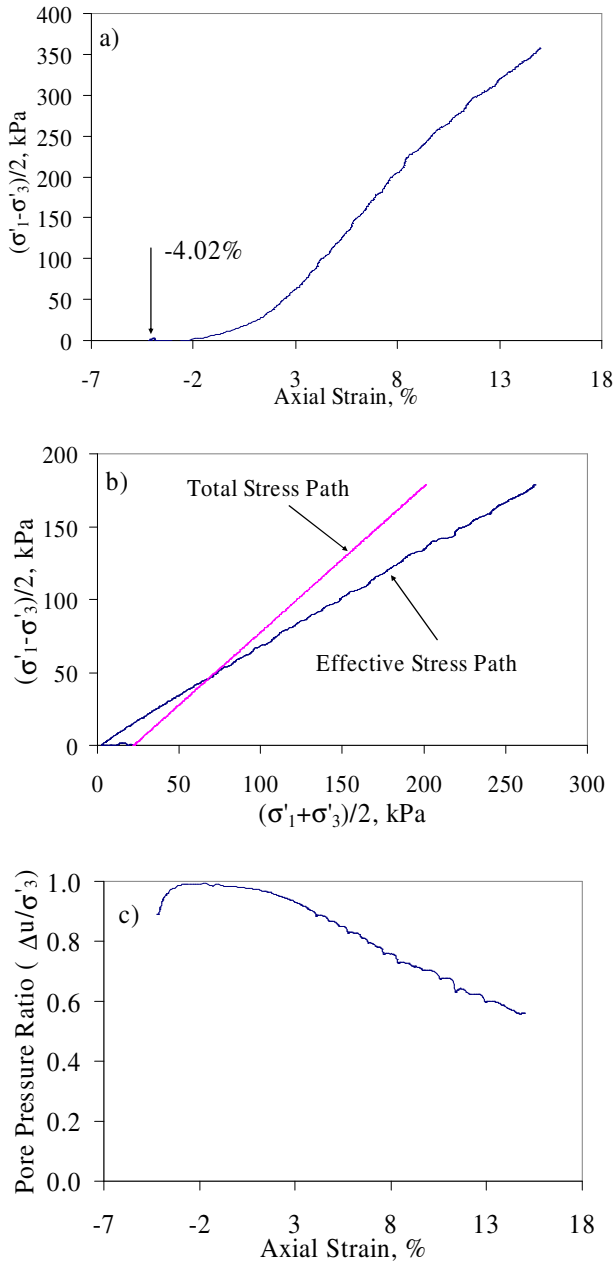


Figure A1.47. Typical post-cyclic shear test on rock-only specimen (RPCT200-4) after reaching 2.5% axial strain during cyclic shear – a) Stress-strain response b) Stress path c) Excess Pore Water Pressure. (refer to Figure A1.46 for cyclic part).

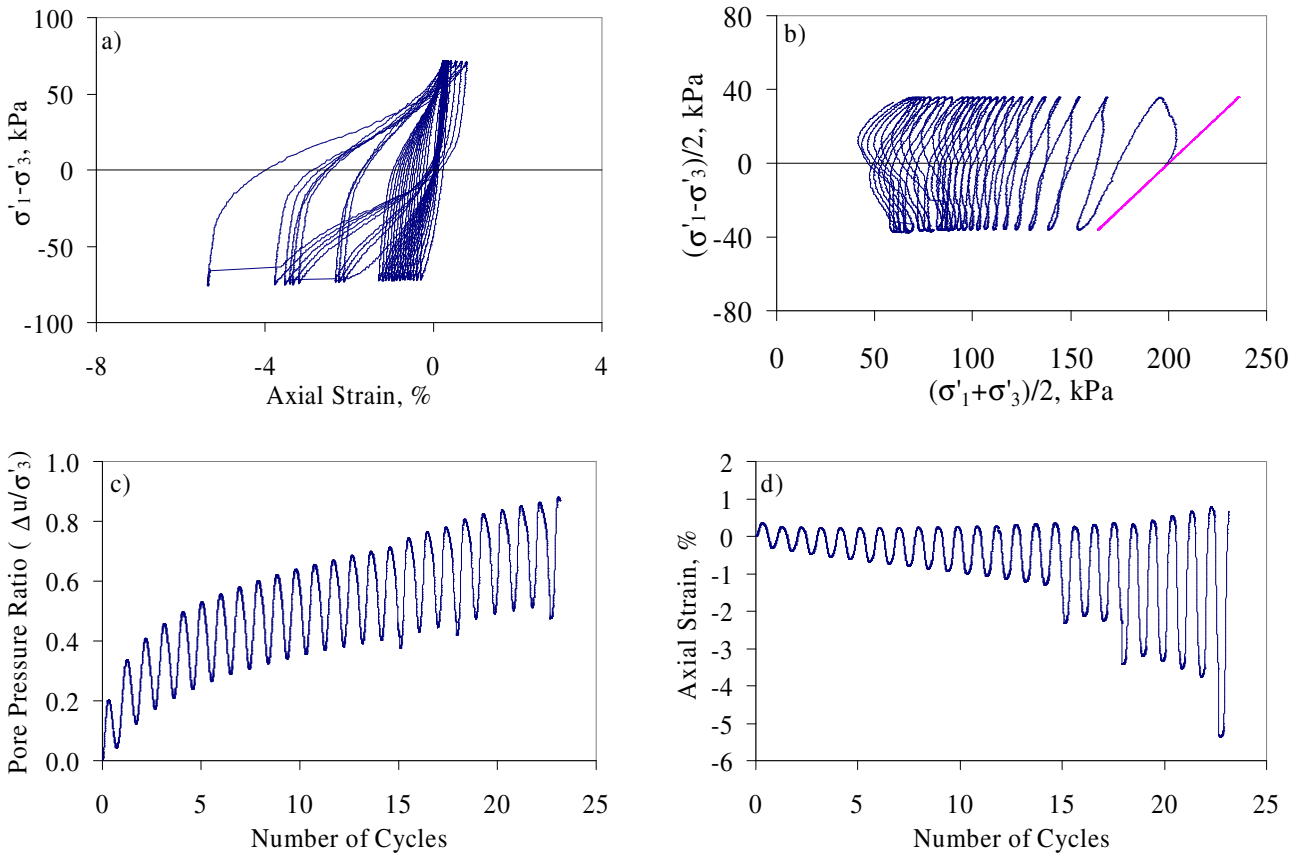


Figure A1.48. Cyclic shear response of tailings-only material - Test TCT200-1; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (CSR = 0.175; $e_{Tailings}$ = 1.071).

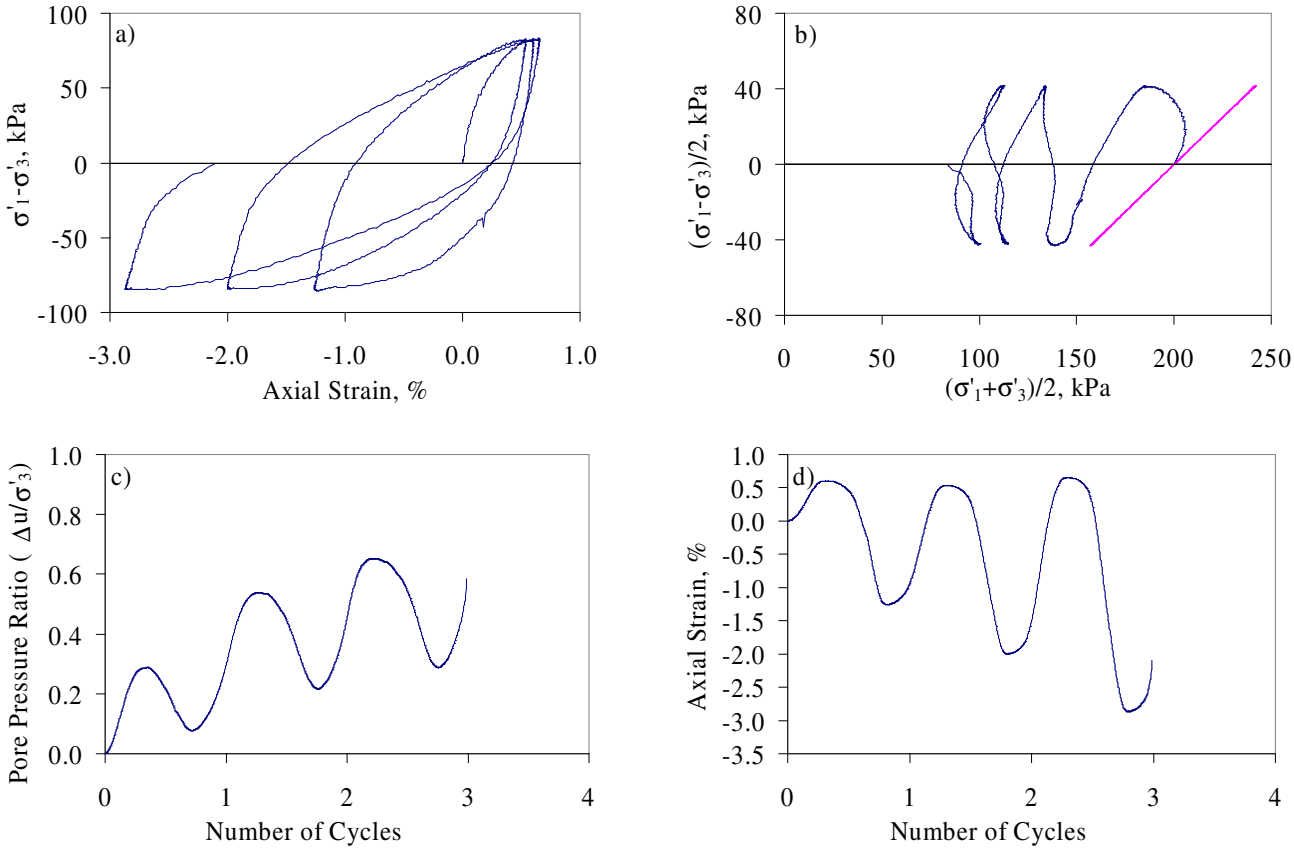


Figure A1.49. Cyclic shear response of tailings-only material - Test TCT200-2; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (CSR = 0.225; $e_{Tailings}$ = 1.091).

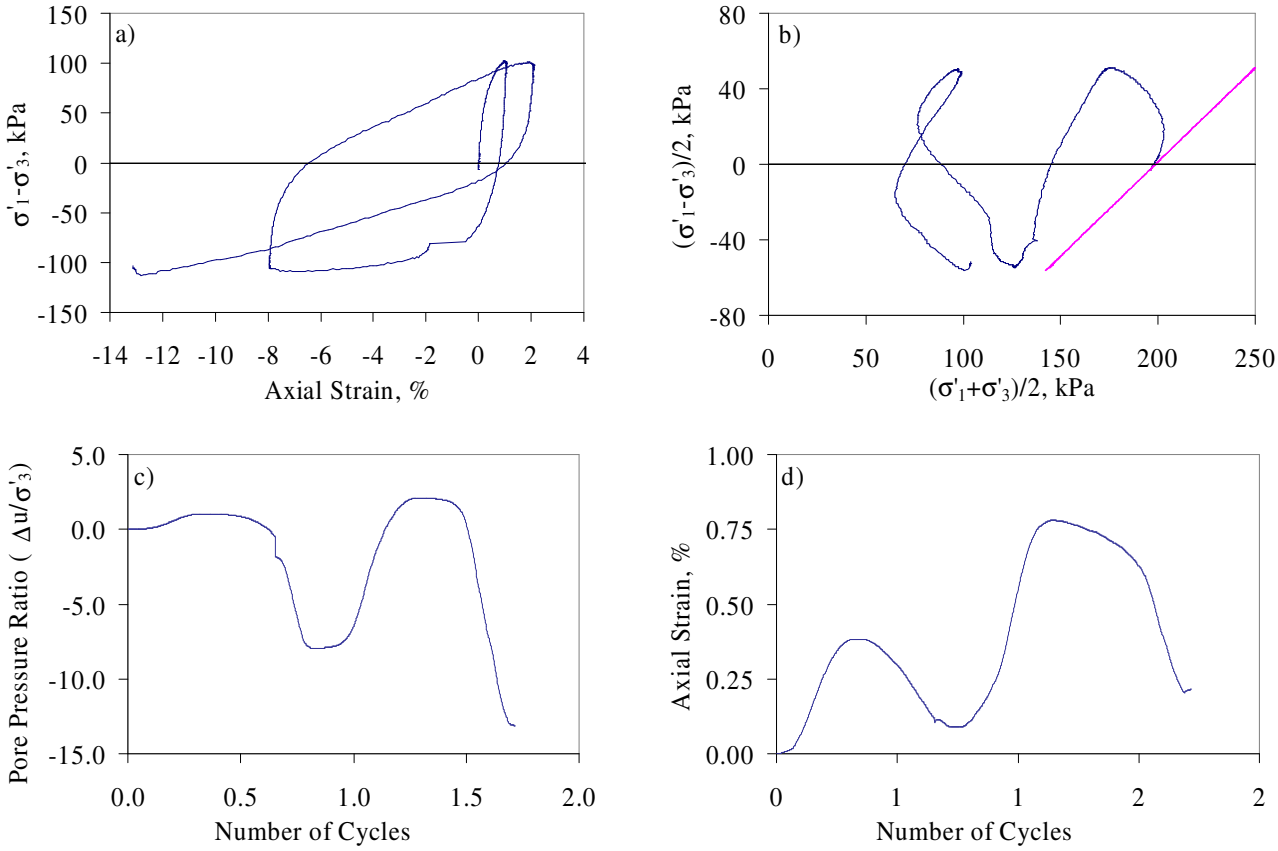


Figure A1.50. Cyclic shear response of tailings-only material - Test TCT200-3; (a) stress-strain [(σ'_1-σ'_3) vs. ε_a]; (b) stress-path [(σ'_1-σ'_3)/2 vs. (σ'_1 +σ'_3)/2]; (c) excess pore water pressure ratio [r_u = ($\Delta u/\sigma'_{3c}$)] vs. number of cycles; and (d) axial strain (ε_a) vs. number of cycles (CSR = 0.275; $e_{Tailings}$ = 1.107).