Investigating Cyclic Liquefaction in Transitional Tailings

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Thesis submitted to UCL for the Degree of
Doctor of Philosophy

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April 2022
Declaration

I, Abigail Cartwright, confirm that the work presented in this thesis is my own. Where information has been derived from other sources, I confirm that this has been indicated in the thesis.

Signed

Abigail Cartwright
Abstract

The Fundão and Brumadinho dam disasters have focussed attention on the stability of tailings dams, although in fact there have been around 250 tailings dam incidents in the last century. In recent years it has been identified that many natural soils of complex mineralogy and/or mixed grading have modes of behaviour that differ from conventional soil mechanics in that the effects of the initial void ratio at their deposition cannot be erased by loading at engineering stress and strain levels. The normal compression lines and critical state lines that they reach in terms of void ratio are therefore not unique but depend strongly on this initial state. From their origins as crushed rock, it may be expected that the “transitional” behaviour would occur in tailings, but it has often been overlooked as tests on samples of different initial void ratios required to identify this behaviour are rarely carried out.

This thesis describes laboratory triaxial and oedometer tests on tailings specifically designed to investigate transitional behaviour highlighting how the strength is dependent on the volume at deposition. The data show how the normal compression and critical state lines move in the void ratio – stress space depending on the initial density and the strong implications for cyclic liquefaction are discussed along with the implications for the method of placement for dam stability.

Undrained cyclic tests were carried out on a mineral sand tailings from Australia. Data for a more conventionally behaved tailings, an iron tailings from China, were also reanalysed. The mineral sand tailings was found to be highly transitional and cyclic tests concluded that the initial specific volume, \( v_i \), did not make a significant difference to the mechanical behaviour and \( v_i \) was found to be much less important than the cyclic stress ratio, CSR, and the initial mean effective stress, \( p_i' \).
Impact Statement

The analysis of laboratory test data of mineral sands tailings from near Perth, Australia has highlighted the transitional nature of the material. This behaviour differs from conventional soil mechanics in that the effects of the initial density at their deposition cannot be erased by loading at engineering stress and strain levels. The tests presented in this thesis show that the initial density does not make a significant difference to the mechanical behaviour in undrained cycling and the density was found to be much less important than the magnitude of the stress cycle and the initial mean effective stress.

A significant number of tailings are hydraulically placed in loose states. If a tailings is transitional, it might have been a concern that the initial placement density would have a significant impact on subsequent behaviour if compression under the overburden pressure does not cause it to reach a unique volume for a given stress level. This turns out not to be the case for the mineral sand tailings and the volumes in compression for different initial densities converge only slowly so the amount of compression experienced will be not so different for different initial densities. The difference in behaviour for different initial densities is then small in shearing and cyclic loading. It may therefore not be so necessary to be concerned about the placement density for transitional tailings. If the tailings had conventional behaviour the placement density would be very important, as was shown in the reanalysis of existing data.

This work was presented in various seminars and conferences by the Author including the 15th BGA Young Geotechnical Engineers Symposium in 2018 and the 20th International Conference on Soil Mechanics and Geotechnical Engineering, Sydney 2022. A paper titled ‘the implications of transitional behaviour for tailings’ was selected for the Sydney conference, and a 10-minute video presentation was
submitted. A paper for submission to Géotechnique is being prepared. Outputs were delayed and adjusted because of the effects of Covid.
Acknowledgements

This work would not have been possible without the help and support from the following people.

First of all, I would like to thank my supervisors Professor Matthew Coop and Dr. Beatrice Baudet for giving me the opportunity to carry out this research at UCL and for making this journey insightful, challenging and fulfilling. It has been such a fantastic experience and I leave with two very good friends.

Many thanks to Ben Boorman, Matt Wilkinson and Dr. Pedro Ferreira for their constant support in the soils laboratory and to Dr. Judith Zhou and Rachel Fairfax who helped me determine which tailings were safe to bring to UCL. To Dr. Li Wei, Professor Andy Fourie, Sara Töyrä and Roger Knutsson for supplying the tailings used in this study and to Professor Ian Wood and Jim Davy for assisting me in the Earth Sciences laboratory.

To my PhD colleagues for the hours spent helping each other in the laboratory and for the necessary coffee breaks. To UCLWFC, the Accies and TeamUCL for providing a much-needed stress outlet. I could not have asked for better teammates. To my best friends: Simone, Fareeha, Shannon, Lucy, Fiona, Helen, Hayley, Anna, Lucy and Neal. Thanks for the drinks and the laughs when things got too serious.

Thanks to my fantastic family. Special thanks to Jo and Andy for housing us during the multiple lockdowns and for being so generous and supportive, and to my parents, Karen and John, for pushing me to be the best I can be but for also loving me no matter what.

Finally, thanks to James for being a constant source of love and support and for funding my trips to Center Parcs over the last four years!
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List of Symbols and Abbreviations

\( A: \) Particle area

\( B\)-value: \ Skempton pore pressure coefficient

\( C_c: \) Compression index

\( C_s: \) Swelling index

\( C_u: \) Coefficient of uniformity

\( d_{50}: \) Median particle size

\( D_i: \) Initial diameter of the sample

\( e: \) Void ratio

\( e_{CS}: \) Critical state void ratio

\( e_i: \) Initial void ratio

\( e_{liq}: \) Void ratio for liquefaction

\( e:lnp': \) Void ratio-log stress plane

\( f_c: \) Fines content

\( G_s: \) Specific gravity

\( h_s: \) Granulate hardness

\( I_B: \) Brittleness index

\( I_P: \) State pressure index

\( K_0: \) Coefficient of earth pressure at rest

\( m: \) Gradient of oedometer compression paths

\( M: \) Extension modulus of rubber membrane per unit width

\( M: \) Gradient of CSL in \( q: p' \) space

\( n: \) Constant exponent in the Gudehus equation

\( N: \) The projected intercept of the isotropic NCL at 1 kPa

\( p': \) Mean effective stress

\( p'_{cs}: \) Equivalent pressure on the CSL

\( p'_{o}: \) Equivalent pressure on the NCL

\( p_i' \text{ or } p_0' : \) Initial \( p' \) at the start of cyclic loading

\( P: \) Gradient of \( \Gamma: v_{20} \) data

\( P_{real}: \) Particle perimeter

\( q: \) Deviator stress
\( q_{\text{max}} \): Maximum deviator stress
\( r \): Cyclic stress ratio
\( R \): Roundness
\( S \): Sphericity
\( u \): Pore pressure
\( v \): Specific volume
\( v_i \): Initial specific volume
\( v_{\text{liq}} \): Specific volume above which full static liquefaction occurred
\( v_{\text{max}} \): Maximum specific volume
\( v_{\text{min}} \): Minimum specific volume
\( v_n \): Normalised specific volume
\( v_n: \ln p' \): Normalised specific volume-log stress plane
\( v: \ln p' \): Specific volume-log stress plane
\( w_f \): Final water content
\( w_i \): Initial water content
\( w_L \): Liquid limit
\( w_p \): Plastic limit
\( w_0 \): Weight of oven-dry soil
\( w_A \): Weight of pycnometer filled with water
\( w_B \): Weight of pycnometer filled with water and soil
\( w_p \): Weight of empty pycnometer
\( w_{PS} \): Weight of pycnometer filled with dry soil
\( \gamma_{\text{bulk}, f} \): Final bulk unit weight
\( \gamma_{\text{bulk}, i} \): Initial bulk unit weight
\( \gamma_{\text{dt}} \): Initial dry unit weight
\( \gamma_w \): Unit weight of water
\( \varepsilon_a \): Axial strain
\( \varepsilon_r \): Radial strain
\( \varepsilon_v \): Volumetric strain
\( \sigma'_{\text{a}} \): Axial effective stress
\( \sigma'_{\text{c}} \): Initial effective confining stress
\( \sigma'_r \): Radial effective stress
\( \sigma'_{vmax} \): Maximum vertical effective stress
\( \phi'_{cs} \): Critical state angle of shearing resistance
\( \Delta \sigma_3 \): Change in cell pressure
\( \Delta \sigma_r \): Change in radial stress
\( \Delta q \): Amplitude of cyclic deviatoric stress
\( \Delta u \): Change in pore pressure
\( \infty \): Infinite cycles
\( \Gamma \): The projected intercept in the \( v:lnp' \) plane of the CSL at 1 kPa
\( \lambda \): The gradient in the \( v:lnp' \) plane of the CSLs or NCLs
\( \psi \): State parameter
\( \psi_m \): Modified state parameter
\( \beta \): Amplitude of cyclic loading ratio
\( \phi' \): Friction angle

1D-NCL: One-dimensional normal compression line
CE diameter: Circular equivalent diameter
CRR: Cyclic resistance ratio
CRSP: Constant rate strain pump
CSL: Critical state line
CSR: Cyclic stress ratio
CSSM: Critical state soil mechanics
Cyc: Cyclic test
DD: Dry deposition
IC NCL: Isotropic normal compression line
LEP: Lubricated end platen
LVDT: Linear variable differential transformer
MIP: Mercury intrusion porosimetry
MP: Moist placement
MT: Monotonic test
NC: Normally consolidated
NCL: Normal compression line
<table>
<thead>
<tr>
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<th>Description</th>
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<tr>
<td>OC</td>
<td>Overconsolidated</td>
</tr>
<tr>
<td>SEM</td>
<td>Scanning electron microscope</td>
</tr>
<tr>
<td>SOP</td>
<td>Standard operating procedure</td>
</tr>
<tr>
<td>TRIAX</td>
<td>Software for controlling triaxial testing</td>
</tr>
<tr>
<td>WS</td>
<td>Wet sedimentation</td>
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<td>XRD</td>
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1. Introduction

Tailings are the residual materials after extracting valuable minerals from the ores. The characteristics of tailings are highly variable depending on the composition of the ores and the extraction processes used. From their gradings and mineralogies, it might be expected that some or even many tailings would have “transitional behaviour”. There are an increasing number of soils that are found to have a mode of behaviour that is often referred to as transitional. While most soils may have well defined normal compression lines (NCLs) and critical state lines (CSLs) in the void ratio \( (e) \) or specific volume \( (v = 1 + e) \): log stress plane, for these materials the locations of these lines are strongly a function of the initial void ratio at the point they are deposited or created (Martins et al., 2001; Ferreira and Bica 2006). Unfortunately, researchers and engineers rarely look for this behaviour since it requires tests to be carried out from samples with a range of different starting void ratios at the point of setting the sample up, which is laborious, and researchers generally follow set procedures with a fixed initial void ratio. Li and Coop (2019) and Li et al., (2018) did look for transitional behaviour in tailings and while they did not find it in iron ore tailings of various gradings, they did find a weak transitional behaviour in a gold tailings. The purpose of this research was to identify the behaviour of more strongly transitional tailings, examining how this would influence its behaviour under both monotonic and undrained cyclic loading in, for example, a seismic event.

Three materials were tested in this study. These were an iron tailings from the Wanniangou tailings dam in Panzihua City, China, a mineral sand tailings excavated from a mine about 100 km South of Perth, Australia, and an iron tailings from Sweden.
The main aims of this research were:

- To identify the transitional nature of the three materials.
- To examine the cyclic behaviour of a soil that is transitional.
- To see how transitional behaviour affects the susceptibility to cyclic liquefaction.
- To identify whether or not placement density is important for a transitional soil during cyclic loading.
- To investigate fabric effects through different sample preparation methods.
- To see if layering has an effect on the cyclic response.
- To examine the effects of lubricated end platens on the cyclic response.

Following on from this introduction is the literature review (Chapter 2) which examines the dangers of tailings dam failures and highlights the importance of a study on the cyclic liquefaction of tailings, particularly focusing on the possible effects of transitional behaviour. Chapter 3 outlines the equipment and procedures used for the laboratory work conducted in this study, along with an introduction on the tailings selected for investigation. A reanalysis of previous research is presented in Chapter 4 focusing on the work carried out by Li (2017) on the iron tailings from the Wanniangou tailings dam in Panzihua City, China, and the work by Carrera (2008) on the fluorite tailings from the Stava, Italy collapse in 1985. Li carried out cyclic loading on a non-transitional soil and so these data were reanalysed for comparison. Li’s tests had been completed but had not yet been fully analysed. Chapter 5 investigates the mechanical behaviour of a mineral sand tailings from Australia through compression and shearing testing and the thesis ends with a conclusion and suggestions for future work in Chapter 6.
This work was significantly affected by covid and there were long periods of time where the soils laboratory at UCL was closed or access was restricted. During the course of this research the scope shifted from laboratory testing to the reanalysis of previous research because of this.
2. Literature Review

2.1 Introduction

This literature review is split into six main sections; the first section examines what exactly mine tailings are, including their characteristics, storage and demand. Section two looks at tailings dams and tailings deposition and identifies the ways in which tailings dams are constructed. A discussion on the liquefaction phenomenon is presented in section three, including tailings static liquefaction and cyclic liquefaction. An overview of some tailings dam failure case studies along with some ideas on preventative measures are given in the next section and then section five introduces the critical state soil mechanics (CSSM) framework. The literature review ends with a section on the sample preparation methods used to create samples for triaxial testing.

This literature review examines the dangers of tailings dam failures and highlights the importance of a study on the cyclic liquefaction of tailings particularly focusing on the possible effects of transitional behaviour. An overview of what has been discussed in the literature will be presented at the end to summarise key findings.

2.2 Tailings

2.2.1 Introduction

After extracting the valuable minerals from the rock milling process, a significant amount of residual waste is left behind. This waste product is known as tailings. It is estimated that the production rate of mine tailings ranges between five and seven billion tonnes per year worldwide (Qi and Fourie, 2019). Tailings are normally in the form of slurries, consisting of ground rock and process effluents, and often contain potentially hazardous contaminants. The solids in the slurry are unwanted minerals
such as silicates, oxides, hydroxides, carbonates, and sulphides (Lottermoser, 2014), however, the tailings will usually contain a small amount of the sought-after mineral. As the demand for minerals and metals rises, the challenges associated with the storage of tailings increases.

Tailings are an artificial material and the mechanical behaviour upon loading is different when compared with that of natural soils. It is essential to understand the mechanical behaviour of tailings materials so that tailings dams can be constructed and monitored safely.

2.2.2 Characteristics
Tailings are the waste material from very recent ore-crushing processes. This means that the particles are angular to sub-angular in shape. Tailings particles are therefore more angular than natural soil particles and this is likely to have an influence on the mechanical properties.

The crushing of rock to extract the metals is the process that determines the grading of the mining waste, therefore the grading can be very variable. The coarse and fine fractions are also separated to some degree by segregation of the tailings particles after slurry discharge into the tailings dam (Wijewickreme et al., 2005). Often, tailings materials are highly heterogeneous and may present anisotropic properties (Vanden Berghe et al., 2011). Tailings are generally characterised by a high specific gravity, $G_s$ (Carrera, 2008). $G_s$ is discussed in more detail in Section 3.7.1.

2.2.3 Storage
The cheapest and most common way to store the tailings slurry is behind dams known as tailings dams. The initial dyke of the dam is commonly made of the coarse part of the tailings as well as locally-derived soil and waste rock. The tailings are normally
stored under water to prevent the formation of surface dusts and of acid mine drainage (Kossoff et al., 2014). Acid mine drainage can be highly toxic and very damaging to the environment.

While most disasters involving hydraulic dams have been prevented some years ago, the same has not yet been observed with tailings dams (Soares, 2015). As the demand for mining products continues to rise, the storage capacity of tailings dams needs to increase. As already mentioned, the dams are usually raised using the tailings material itself, which can cause significant problems. One particular construction method, known as the upstream method, presents many stability problems, because the tailings themselves are used for construction. This method is outlined in Section 2.3.3 below. Liquefaction is also a major concern as tailings dams are not designed to support liquefied material. Failure case studies will be investigated in Section 2.5.

2.2.4 Demand

The demand for mining products is ever increasing and modern society could not function without the products of extractive mining industries. They are essential to many aspects of our daily lives. The mining products are vital components in, for example, metals, jewellery, paint, ships, aeroplanes, computers and construction materials (Kossoff et al., 2014). Advances in mining technology have made it possible to exploit lower grade deposits despite decreasing commodity prices, which means disposing of more tailings and putting more pressure on tailings facilities (Armstrong et al., 2019). There are at least 3,500 tailings dams worldwide (Kossoff et al., 2014) and this number is ever increasing.
2.3 Tailings Dams

2.3.1 Introduction

This section looks at the construction materials used to create the tailings dam, the construction methods used and the tailings deposition. Tailings dams mainly consist of a tailings pond and embankment and, after the extraction process, the tailings are pumped into the pond to allow for the separation of the solid particles from the water (Li, 2017). These tailings can be pumped from the mill as a slurry having a water content ranging from 50% to over 100% (Highter and Vallee, 1980). The construction type is selected based on a variety of factors including the geology of the area, topography and climate.

2.3.2 Construction Materials

Since tailings are a waste product of the mining industry, and their disposal adds to the cost of production, the majority of confining embankments are constructed of the coarse, sand-sized fraction of the tailings (Highter and Vallee, 1980). The coarsest particles used for the construction of the dam have the highest permeability of the tailings material so that the water in the pores can escape when there are adequate drainage outlets. Sometimes, but unfortunately not always, this sandy fraction is compacted using vibrating compactors to make it denser and, thus, more resistant to liquefaction (Carrera, 2008). A balance needs to be found between stability, environmental performance and cost. Cost is usually placed as most important, meaning that tailings dam failures continue to occur today. Ideally, a chemical and physical analysis of the tailings material should take place prior to construction to analyse potential dam failure or contaminant release, but this is not always the case.
2.3.3 Construction Methods

Generally, tailings dams are not constructed initially to completion. They are raised gradually or sequentially as and when the storage demand increases. The three types of raised embankments are; upstream (towards the tailings), downstream (away from the tailings), or in centreline (Lottermoser, 2014). All types start with a small embankment which may only be a few metres high.

The most common construction method is the upstream method (Fig. 2.1) and more than 50% of tailings dams worldwide were built using this method (Lottermoser, 2014). This is because less filler material is required during construction. This approach is the most cost-effective as it maximises the storage volume and minimises the volume of imported material. The method involves elevation of the embankment at different times to allow for more storage of tailings material as the mining process continues. The centreline of the retention embankment moves upstream as the height of the embankment increases (Harder and Stewart, 1996). There are, however, many disadvantages with this method. Dams using the upstream method represent up to 66% of the worldwide reported mine tailings dam failures (Carmo et al., 2017).

The dykes are sequentially constructed on the previously deposited tailings. The mechanical behaviours of these constructed layers depend on the method of deposition, process of sedimentation and rate of consolidation (Bhanbhro, 2014). If the previously deposited tailings are in a loose and saturated state, the dam could be highly susceptible to failure and erosion, and particularly vulnerable to liquefaction. Loose layers are likely to be stable under drained conditions but may fail in undrained conditions (Lade, 1992). Liquefaction can occur in loose layers if the raising of the dam is fast as the pore pressures increase and the effective stress reduces. Liquefaction is discussed in more detail in Section 2.4.
Good sedimentation and compaction are necessary with the upstream method as the construction material must be resistant enough to withstand the weight of the upper embankment (Carrera, 2008). The dam should be raised slowly to allow time for the deposited material to consolidate enough to support the next level of the dam. The tailings material itself is relied upon for stability and this is why this method should not be used in seismic areas (Vanden Berghe et al., 2011).

With the downstream method, the sand is deposited outside the embankment (Fig. 2.2). The crest moves progressively downstream as it is raised. The levees are built with imported material, generally selected for their good drainage properties. In this case, the tailings material does not contribute to the dam stability. This approach is the most robust but also the most expensive in terms of imported material (Vanden Berghe et al., 2011). This method also occupies more surface area than other methods and requires more filler material.

For a centreline tailings dam, the embankment is built vertically upwards (Fig. 2.3). The height of the structure is increased with imported material placed on top of the existing dam. This method is more stable than the upstream method, and requires less material to build than the downstream method. It is also common to find combinations of these different techniques. The most common is to build the lowest part of the dam through downstream or centreline construction and the last raisings using the upstream method (Vanden Berghe et al., 2011).

2.3.4 Tailings Deposition
In most cases, the tailings material is delivered hydraulically from the periphery of the dam (Fig. 2.4) (Vanden Berghe et al., 2011). The material is deposited through spray bars, spigots or hydro-cyclones (Fig. 2.5) (Soares, 2015). Among them the hydro-
cyclones are usually more suitable as they separate the sandy fraction from the fines and the water (Carrera, 2008).

The slurry material flows towards the centre of the pond and the coarser material settles first and forms the beach. The material in the pond is more fine-grained. An effective drainage system is essential to prevent the water table approaching the dam crest, and also to drain the tailings leachate. Some of the water can then be pumped away or drained to allow for consolidation. Consolidation is essential for the stabilisation of the dam. As discussed, the consistency of the tailings stored within the dam is initially that of a fluid slurry, meaning that the material is highly susceptible to liquefaction, although it consolidates as it is buried. Tailings dams are not designed to withstand fully liquefied tailings and this will be seen in the Aznalcóllar case study in Section 2.5.5.

2.3.5 Overview

The upstream method has been banned in some countries, however, there are old mining operations with unsafe tailings disposals that are difficult to identify as well as to control (Verdugo and González, 2015). Significant efforts must be made by governments to identify and prevent further tailings failures.

High levels of monitoring are required during the operation of the dam regardless of the method of construction. Monitoring needs to continue after the dam becomes inactive. However, this is not always done because of the costs involved. Over 70% of the larger mining operations have had tailings dam failures of some kind (Highter and Vallee, 1980). Tailings dam failures can occur due to multiple reasons and some of these are displayed in Figure 2.6. From this graph it can be seen that the number of incidents caused by seismic liquefaction is significant.
2.4 Liquefaction of Tailings

2.4.1 Introduction

Liquefaction is a phenomenon in which a soil becomes unstable and suddenly loses most of its strength and appears to behave as a liquid until a new stability condition is reached (e.g., Carrera, 2008). This occurs when the effective stress of a soil is reduced to essentially zero through an increase in pore water pressure, which results in a complete loss of shear strength. During an earthquake, the loading is repeated rapidly meaning that the water does not have time to flow out before the next load cycle is applied, thus increasing the water pressure, i.e., the soil is undrained or not fully drained. The liquefaction phenomenon majorly depends on the relationship between loading rate and permeability (Karakan et al., 2019). The soil begins to behave like a liquid, not showing resistance, which can cause devastating disasters (Ramos, 2021). It is one of the most dangerous phenomena in Geotechnical Engineering.

Liquefaction is generally only observed at shallow depths where the stresses are low and is the result of a disturbance. This can come from earthquakes, repeated loadings from engineering works or can be monotonic where there is loading under undrained conditions (Carrera, 2008). Monotonic loading can result in static liquefaction which can be triggered by; overtopping and erosion of the dam, a loss of containment in the dam due to dam instability, pore water pressure increase due to a rise in the dam height and heavy rainfall causing a rise in the phreatic surface (Williams, 2019).

Many of the basic concepts regarding the triggering, progress, effects, and mitigation of liquefaction remain unclear (Soares and Viana Da Fonseca, 2016). The liquefaction phenomenon has only recently been investigated and the fact that
liquefaction could be static, as well as cyclic, was recognised only after the Aberfan spoil heap failure in 1966.

2.4.2 Tailings Liquefaction

As stated by Carrera (2008), the liquefaction phenomenon only involves soils which are saturated, graded from sands to silts, in a loose state with a tendency to contract, and in conditions that do not allow drainage, even if just temporarily. Other factors that influence liquefaction susceptibility are the tailings composition, which includes particle size, shape and gradation. The composition of the soil has an impact on its volume change behaviour and consequently on the development of excess pore pressure required for liquefaction (Ramos, 2021).

Soils deposited in a loose state show high liquefaction susceptibility when saturated (Ramos, 2021). Sandy tailings are usually hydraulically placed onto the embankment and are in a very loose state. Compaction rarely occurs and this means that tailings dams are particularly susceptible to liquefaction.

Intense rainfall can result from the ongoing impacts of climate change. The rainfall can cause tailings dam reservoirs to liquefy if drainage is not adequate (Palmer, 2019). This can cause increased pressure on the dam walls eventually resulting in collapse.

Tests were conducted by Carrera (2008) on the fluorite tailings from Stava, with different silt and sand fractions, to investigate the effect of grading on the susceptibility to liquefaction. Generally, well graded soils are considered less susceptible to liquefaction because, in drained conditions, the smaller particles tend to fill the voids, and thus their compressibility will be lower, which produces lower increases in pore pressure in undrained conditions.
2.4.3 Cyclic Liquefaction

The release of seismic waves can cause increased shear stress on tailings embankments. The cyclic shear loading can then cause an accumulation of pore pressures in saturated tailings which can result in liquefaction. Cyclic liquefaction hazard is a repetitive phenomenon, not only due to the recurrent earthquake activity, but also due to the loosening of soil from previous cyclic loadings (Soares, 2015). The loading weakens the strength of the tailings, causing permanent deformations and destabilisation of the tailings dam (Lyu et al., 2019). The deformations largely stop once the cyclic loading ends.

Poulos et al., (1985), noted that ‘it has been customary to use the term ‘liquefaction’ to mean 5% strain in a cyclic load test’. This threshold was used when identifying cyclic liquefaction in the results in this study. The term cyclic mobility is usually applied to cases where zero effective stress is not approached and only limited deformations are produced (e.g., Gens, 2019), but there is no distinct point at which cyclic mobility ‘failure’ can be defined (Kramer, 1996). Figure 2.7 shows the cyclic stress path entering the ‘dilative’ region and forming repeated loops that touch the CSL which causes the sample to deform without any further changes to the mean effective stress, $p'$ (Hyde et al., 2006). This is cyclic mobility because $p'$ does not reach zero and so the strains are limited. In contrast to this, Figure 2.8 is an example of cyclic liquefaction from Ramos (2021) for a loose granular soil subjected to undrained cyclic simple shear loading. As the cyclic loading progresses, the stress-path approaches the origin and the strains are unable to keep up as the sample has little effective stress. Cyclic liquefaction occurs when the stress-path reaches the origin. For cyclic liquefaction in triaxial tests, the change in pore pressure would equal the initial $p'$ and so the current $p'$ would equal zero.
Seismic liquefaction is the second most common cause of tailings dam breakages in the world, with the exception of Europe, where the second most common cause is foundation failure (Antonaki et al., 2017). Unusual rainfall is the most common cause worldwide. In the field, the magnitude of the cyclic shear stresses from earthquakes will vary from cycle to cycle because of the irregular nature of the loading (Spence, 1993). Along with seismic waves, cyclic liquefaction can also be caused by vibrations from heavy equipment and nearby motions such as mine blasting. Despite being extensively studied by several researchers, soil liquefaction is far from being accurately predicted.

The analyses of various grain size distributions of soils which have liquefied or not during case history earthquakes was carried out by Tsuchida (1970). Coarse sands and gravels were found to be less susceptible to cyclic liquefaction as the large spaces between the particles allowed for the dissipation of pore water pressure. Janalizadeh Choobbasti et al., (2013) commented that as the particle size increases, the cyclic resistance increases as well. Tsuchida (1970) also found that soils which had a high content of fine particles were less susceptible to liquefy, due to the difficulty of particle rearrangement prevented by the plasticity of these soils. Tailings, however, mostly have small to no plasticity as the material is crushed rock.

2.4.4 Overview

The most dangerous and destructive failures of tailings dams are those in which the waste loses strength, becomes mobile, and flows as a viscous fluid in which the supporting fluid is usually water (Blight and Fourie, 2005), although the water often has a modified chemistry and can be very acidic. This is typical of the dams with part of their structure lying on the tailings deposits. It is therefore crucial to test and characterise tailings materials to prevent future failures.
2.5 Case Studies

2.5.1 Introduction

Tailings dam stability issues remain a major challenge in the mining industry. They are some of the largest structures designed by geotechnical engineers. Although tailings dam failures are often considered extremely rare events, they are far more common than is realised. In the 17-year period from 2000 to 2017, 36 such cases occurred (Armstrong et al., 2019). For a world inventory of 18,401 mines, the failure rates are estimated to be 1.2% over the last century or 2.2 per year (Soares, 2015), and the number of major tailings dam failures has doubled over the past 20 years (Armstrong et al., 2019). This is because there has been a reluctance to spend money on tailings deposition as it is a waste material and not a moneymaking aspect of the mining industry. Many of the failures remain under- or unreported, which has seriously hindered the development of appropriate safety regulations (Lyra, 2019). There also seems to be an insufficient understanding on the mechanisms associated with tailings dam failures (Lyu et al., 2019), which is why new and continued research in tailings is so important.

Below are a number of case studies where tailings dams have failed. It is important to examine the causes and impacts and identify the sites where liquefaction has occurred and where it can potentially happen again. The first liquefaction event ever recorded was the Calarevas dam failure, in Alameda County, USA, 1918 (Soares, 2015), however, tailings dams continue to liquefy and fail to this day.

2.5.2 Brazil - Bento Rodrigues

In Brazil the mining industry is very important to the country’s economy. In 2001, it was reported that there were 839 tailings dams in Brazil, representing 3.66% of dams
worldwide (ICOLD, 2001). Between 2000 and 2009, 140 dam incidents were reported in Brazil (Camarero et al., 2019).

On November 5th 2015, roughly 62 million m$^3$ of iron ore tailings were released because of the collapse of the Fundão tailings dam in Minas Gerais, Brazil (Fernandes et al., 2016). The increase in the load that was being deposited caused the dam collapse through static liquefaction. The slurry ran into the small district of Bento Rodrigues, displacing the entire population of 600 people and killing 19 (Fernandes et al., 2016). The tailings also caused irreversible environmental damage to hundreds of watercourses in the basin of the Doce River and associated ecosystems (Carmo et al., 2017). After reaching the Doce River Valley, the slurry travelled 650 kilometres until it reached the Atlantic coast 17 days later (Roche et al., 2017). The disaster devastated agricultural areas, and interrupted electricity generation and the water supply (Segura et al., 2016), and caused an almost complete extermination of the Doce river’s fauna (Camarero et al., 2019). The effects of the failure can be seen in Figures 2.9 and 2.10.

The dam was designed to contain 111.6 million m$^3$ in total of tailings material during its 25-year lifespan, but in the space of seven years, Fundão contained 56.4 million m$^3$ of iron ore tailings (Carmo et al., 2017). The upstream method was used to accommodate the increased volume. Two hours prior to the dam failure, two earthquakes were recorded in the mine area, and another small earthquake occurred during the time of the collapse. Although Brazil is not very seismically active, small earthquakes are a common occurrence (Agurto-Detzel et al., 2016). The earthquakes in the mine area all had a magnitude of 2.0-2.5. One theory for the collapse was that it was either triggered by the ground shaking from the earthquakes, or, the earthquakes caused the tailings to liquefy which resulted in dam failure (Agurto-Detzel et al., 2016). Soil liquefaction is the most common cause of tailings dam failure related to
earthquakes (ICOLD, 2001). However, the number and amplitude of seismic waves required to cause soil liquefaction is significant, and the earthquakes that occurred had a low magnitude and short duration (McRoberts and Sladen, 1992). Another cause could have been static liquefaction, where no seismic triggering is required.

After a complete risk assessment of the site, it was found that the main reason for the rupture of the structure was the excess of internal pore pressures. The dam was saturated and could not stand the high hydrostatic load, which triggered static liquefaction (Camarero et al., 2019). The earthquakes that occurred prior to the collapse exacerbated the structural weakness of the dam (Roche et al., 2017). The dam had also not been monitored or maintained properly.

Carmo et al., (2017) noted that at the time the collapse, Fundão was the seventh such case that had occurred in Minas Gerais alone since 1986. Even though this event highlighted the need to review tailings dams in Brazil, another tailings dam in Minas Gerais failed just over three years later killing hundreds.

2.5.3 Brazil – Brumadinho
A very recent failure was the 85-metre iron-ore tailings dam at the inactive Córrego de Feijão mine in Brazil. The collapse occurred on Friday 25th January 2019 and the dam released roughly 11.4 million m$^3$ of tailings. An image of the moment the Brumadinho dam collapsed is shown in Figure 2.11. At the time of writing, at least 260 people had died and a number of people were still declared as missing. This makes this event one of the deadliest tailings dam disasters in history (Palmer, 2019). The cause of the failure is still unknown and there was no warning from the monitoring system, but cracks appeared on the wall of the dam just before the collapse. The dam
collapsed within seconds and liquefied tailings material flowed rapidly (Palmer, 2019).

The dam was built in 1976 using the upstream method. The upstream method was also used to construct the Fundão dam and both dams are situated in South-Eastern Brazil. There are another 130 dams of this type in the country (de Sá, 2019) and because of this, it is evident that the main concern has been over the country’s economy rather than the potential effects on human life and the environment. The failure of the Fundão dam should have alerted the government to the need for tighter regulations.

2.5.4 Italy – Stava

These disasters are by no means restricted to developing economies, and on the 19th July 1985, the Prestavel fluorite tailings dam in Stava, Italy collapsed. The dam consisted of two basins, one on top of the other, and these were built on a steep slope where springs and water seepages occurred (Fig. 2.12). The inclination of the slope was 40 degrees, which was too steep (Van Niekerk and Viljoen, 2005). The upper dam was constructed when the first dam had reached capacity. The upstream method was used to construct the upper dam which should have been a concern as it should only be used when the tailings beneath are fully consolidated (Carrera, 2008). It is likely that this was one of the principal causes of the failure. The mine operators assumed that the tailings in both dams would consolidate soon after deposition and that the tailings stored in the lower dam could support the load of the upper dam (Roche et al., 2017). This assumption, however, was never verified.

The failure occurred when the up-slope basin collapsed, releasing liquefied tailings into the lower basin. This happened after days of heavy rain, which increased
the hydraulic gradient, which was already high due to the presence of springs in the foundations. The drainage pipes below the dams were incorrectly installed, which caused them to block easily (Van Niekerk and Viljoen, 2005), but a leak in one of the pipes finally caused the failure by static liquefaction. The inflow of the material from the up-slope basin caused overtopping of the lower basin and a subsequent collapse of the lower basin as well (Van Niekerk and Viljoen, 2005). Approximately 200,000 m$^3$ of tailings material liquefied and travelled at 90 km/h downslope, killing 268 people, destroying 63 buildings and demolishing eight bridges. The flow slide destroyed the whole village of Stava. The debris flow, in the first kilometres, had an average velocity of 30 km/h, which is a clear sign of the fact that the tailings underwent static liquefaction (Carrera, 2008).

At the inquest that followed the disaster, it became apparent that the tailings dam underwent no serious stability checks over a period lasting more than 20 years (Van Niekerk and Viljoen, 2005). As with all tailings dams, monitoring should have been regular.

2.5.5 Spain – Aznalcóllar

On the 27$^{th}$ April 1998, a 27 m high rockfill structure slid forward without previous warning. Progressive failure, acting during the long construction period of the dam, resulted in a reduction of the available clay strength in the foundations, eventually causing the failure (Alonso and Gens, 2006). It was the foundations that failed, not the dam itself. Once the dam slid as a rigid body, it caused the tailings behind to liquefy. The solid tailings material therefore turned into a heavy liquid with zero strength. This pushed the dam forward and exacerbated the failure. The central portion of the failed dam travelled 40-50 m until it came to rest and as a result, millions of cubic metres of
fluidised acid mine tailings were released (Fig. 2.13) (Alonso and Gens, 2006). Tailings dams are not designed to retain fully liquefied material.

The downstream construction method was used and the height and downstream extension of the embankment increased continually for 20 years (Alonso and Gens, 2006). This was to keep pace with the increased volume of tailings being decanted into the dam. The failure is an unusual case as it involved translational sliding of the entire dam. Figure 2.14 gives a model of the failure mechanism.

2.5.6 South Africa – Merriespruit

In South Africa, the mining sector is the single largest generator and accumulator of solid wastes (Van Niekerk and Viljoen, 2005). On the night of 22nd February 1994, the 31 m high northern wall of the number four tailings dam of the Harmony Gold mine collapsed (Fig. 2.15). More than 2.5 million tonnes of liquefied tailings swept through the village below. Eighty houses were swept away and 200 others were severely damaged. Seventeen people were killed (Van Niekerk and Viljoen, 2005). Figure 2.16 shows the flow path and damage caused by the liquefied tailings.

The trigger mechanism that caused the liquefaction was the excessive volume of water on top of the impoundment. Initially, a thunderstorm occurred, accompanied with 50 mm of rainfall. From a satellite image study, it was found that there was a considerable quantity of free water on the dam over a long period of time prior to the failure and the water level was estimated at 0.45 m below the crest of the dam on February 1st 1994 (Van Niekerk and Viljoen, 2005). This caused a breach to develop when a rainfall event occurred (Fourie, Blight and Papageorgiou, 2001). It should also be noted that the upstream method was used to create this dam.
On average two tailings dam failures occur every year in South Africa (Van Niekerk and Viljoen, 2005). Despite this knowledge, the Merriespruit disaster still occurred. The mining industry in South Africa now views tailings dams and their potential problems more seriously, and regular checks are now a requirement.

2.5.7 Chile – Las Palmas

The failure of the Las Palmas Dam is an example of cyclic liquefaction. Chile is one of the main copper producers in the world but is located in a geographical area where mega-earthquakes occur and, on the 27th February 2010, one of these earthquakes with a magnitude of 8.8 struck the central-south region of Chile. The earthquake rupture zone is presented in Figure 2.17. Cyclic loading from the earthquake caused the Las Palmas tailings dam to fail, and four people were killed. Satellite images taken before and after the failure are given in Figure 2.18.

The Las Palmas tailings deposit consisted of two dams, one lower and one upper, that were constructed in stages using the upstream method (Verdugo and González, 2015). During and immediately after the earthquake, liquefied tailings and the decant pond displayed an oscillatory movement, temporarily leaving the slope of the dam unconfined (Villavicencio et al., 2014). The upstream slope, in its weakened state, slumped towards the tailings basin, generating cracks or fissures parallel to the dam crest. The liquefied fine material moved through the cracks eroding the retaining dyke (Villavicencio et al., 2014). Tailings flowed downstream about 500 m, contaminating the surrounding area (Fig. 2.19).

Liquefaction sites of general soils covered an extension close to 1,000 km, which approximately represents twice the length of the rupture zone (Verdugo and González, 2015). In the area affected, there were seven major tailings deposits and more than 50
small to medium tailings dams, some of which had limited engineering design (Verdugo and González, 2015). The evidence of liquefaction in the tailings basin was shown by the eruption of sand volcanoes caused by the high water pressures at depth coming to the surface (Fig. 2.20) (Villavicencio et al., 2014).

2.5.8 USA – Tapo Canyon

A magnitude 6.7 earthquake hit Northridge, California on January 17th 1994. This resulted in a 24 m high tailings dam failure in Tapo Canyon, USA. The earthquake epicentre was located approximately 21 km from the tailings dam (Fig. 2.21) and the failure resulted from liquefaction of the stored tailings and also possibly from liquefaction of the embankment itself (Harder and Stewart, 1996). Two sediment boils in tailings were found nine days after the earthquake indicating liquefaction occurred because of the significant water pressures coming to the surface (Harder and Stewart, 1996).

The upstream method was used to construct the dam and substantial amounts of material were added in each stage (Harder and Stewart, 1996). Also, a lack of management after the closure of the tailings dam led to the accumulation of significant amounts of water, and the tailings were kept in a saturated state for a long period of time (Lyu et al., 2019).

Reductions in soil strength and stiffness caused by liquefaction of the tailings, and possibly portions of the dam embankment, caused large and relatively intact blocks of the dam to slide downstream over 60 m. This allowed 135,000 m³ of stored tailings to flow out through the breach and travel several hundred metres downstream (Harder and Stewart, 1996). Fortunately, there were no fatalities but there was considerable economic loss and environmental damage. It is likely that suitable
compaction of the embankments as the dam was raised would have prevented the failure.

2.5.9 Failure Prevention

It is expected that tailings dams should be stable enough to withstand the pressure of the stored material, but as the case studies have shown, this is not always the case, especially when they liquefy. Every tailings dam is unique and the construction depends on a variety of factors including the nature both of the land and the tailings themselves. The site and dam design needs to be selected carefully. The depositional site should be relatively flat without underlying topography. The Aznalcóllar case study indicated that foundations should be strong.

Most tailings dam failures involve water as one of the prime causes. Water levels in the dams are generally very high as the product is deposited in a slurried state. The water flow through the dam generally represents the most critical and most uncertain destabilising load (Vanden Berghe et al., 2011). Ground conditions and drainage facilities around the dam should be sufficient to handle flood episodes (Van Niekerk and Viljoen, 2005). The drainage system is an essential part of the design to prevent any pore pressure build-up close to the dam slope. Significant rainfall can increase water levels in the pond which can cause overtopping. The dams should therefore be created to handle any excessive rainfall.

Appropriate management and recording of any changes should be a continual process after the dam is built. Tailings dams need to be regularly monitored to analyse behaviour. This includes checking the chemical content of the tailings material so any undesired chemical reactions that could affect the behaviour of the dam can be prevented. Tailings are not natural and therefore may behave differently to natural
soils. They have a different chemical composition and have a different depositional process, and as a result, may develop properties that can potentially affect the performance of the dam (Vanden Berghe et al., 2011). For example, chemical reactions could affect the efficiency of the drainage system. An understanding of the mechanical properties and behaviour of tailings material is therefore essential.

From what is presented in the case studies above, it can be confirmed that the tailings dams built using the upstream method may not be safe, especially when cyclic loading occurs. The upstream method has now been banned in Chile but there are still dams that have previously been built using this method which are seismically unstable (Verdugo and González, 2015). Efforts must be made to identify these dams and prevent failures, but also during the construction phase of new dams, material should be compacted as it is deposited to increase stability. Also, because tailings dams are usually raised, using this method or others, the final height of the dam may exceed what was initially planned, which can cause stability issues. This needs to be taken into consideration before the dam is constructed. According to Van Niekerk and Viljoen (2005), the dam walls should not exceed an inclination of 36 degrees and should ideally be less than 26 degrees.

Laboratory tests should be carried out to understand the mechanical behaviour of the soil under various loading conditions to identify the susceptibility to liquefaction. Susceptible tailings can behave in an undrained, contractive, strain-softening manner, and can liquefy or flow (Williams, 2019). Mitigation methods should be applied if there is a chance failure could occur. Current studies have shown dry stacking to be a practical method to prevent the failure of future tailings dams in known zones of high seismic activity. The process involves the deposition of dewatered tailings into the containment areas. It is possible to recover valuable process water while dewatering
the tailings sludge, utilising modern separation technology (Klug and Engelke, 2018). Another preventative measure that could be used is paste thickener that decreases the water content in the slurried tailings. Using thickened tailings allows an extension on the lifetime of the dam as they have a lower void ratio and therefore occupy a smaller volume (Williams, 2019). Other contingency measures include building a toe berm, reducing the height of the dam, flattening a slope, lowering the dam pond, and slowing down the construction rate (Williams, 2019). The majority of these failure prevention methods cost a significant amount of money, but the economic and environmental loss would be much higher if a failure occurred.

2.6 Critical State Soil Mechanics Framework

2.6.1 Introduction

To analyse cyclic liquefaction by means of triaxial tests, the CSSM framework must be introduced. This is a framework that represents the behaviour of saturated soil. It is a means to identify those states of specific volume and confining stress under which a soil can be susceptible to static liquefaction (Carrera, 2008). By increasing the applied shear force, the soil will reach a point at which it flows as a frictional fluid, the critical state. Regardless of their initial state, soils will always tend to their critical state at large shear strains (Schofield and Wroth, 1968). After reaching this state, there are no more changes in mean effective stress, deviatoric stress or void ratio as the shear distortions continue (Schofield and Wroth, 1968).

2.6.2 Critical State Line

The identification of the critical state line (CSL) is important as it permits us to determine whether a soil in a particular state may be susceptible to liquefaction or not.
The liquefaction susceptibility of a soil is related to its density and stress state with respect to the CSL of the soil (e.g., Carrera, 2008). According to critical state theory, when subjected to a monotonic deviatoric stress, a soil sample tends to approach the CSL (Atkinson and Bransby, 1978).

The determination of the CSL is commonly achieved by performing a set of triaxial tests. After obtaining these critical state points, they can be represented in the $e:\ln p'$ plane as an often assumed straight line, named the CSL (Schofield and Wroth, 1968). Once this has been located, the susceptibility to liquefaction of the soil can be estimated by comparing its present state with the CSL. The CSL separates the samples on two sides; samples lying above the CSL will mainly show a contractive behaviour under shearing and are more prone to liquefaction, while those soils whose state lies below the CSL will tend to dilate, reducing the likelihood of liquefaction (Fig. 2.22) (Been and Jefferies, 1985).

The static liquefaction of the Stava fluorite tailings was studied by Carrera (2008). A reanalysis of this work has been carried out in Chapter 4. These results are compared with the results obtained in this project where a variety of samples were reconstituted and cyclically loaded, including loose samples subjected to high stresses, so more results could be seen above the CSL. Different void ratios and densities were examined. The density of a soil is fundamental to predictions of its behaviour, as dense soils are stronger and show a tendency to dilate, while loose soils are weaker and tend to compress. When sheared, denser soils show a volume increase, and loose soils exhibit a volume contraction. This tendency is designated by dilatancy – positive for dense soils and negative for loose soils (Atkinson and Bransby, 1978).
2.6.3 Normal Compression Line

The normal compression line (NCL) is parallel to the CSL, and is defined as the line to which all isotropic paths converge (Atkinson and Bransby, 1978). An example is shown in Figure 2.23 for a carbonate sand. The NCL is another important concept in the critical state framework. It is considered as a boundary line which separates possible and impossible states. Samples with initial states lying on the NCL are said to be normally consolidated, while those lying below are considered to be overconsolidated if they are clays or dense if they are sands. The NCL could be divided into two types based on the compression method used; the one-dimensional NCL (1D-NCL) and the isotropic NCL (IC NCL) (e.g., Li, 2017). Both of these were examined in this research.

2.6.4 State Parameter

The Fort Peak Dam in Montana suffered static liquefaction of its upstream slope during construction in 1938. Soares (2015) commented that the initial soil state plotted below the CSL. Casagrande (1975) attributed the fault to the inability of strain-controlled drained tests to replicate soil flow liquefaction. Wroth and Bassett (1965) proposed an appropriate physical parameter that combines the influence of void ratio and stress level, with reference to an ultimate or critical state to describe the soil behaviour. The state parameter is defined as:

\[ \Psi = e - e_{CS} \]

where \( e \) is the initial void ratio of the soil after consolidation and \( e_{CS} \) is the critical void ratio for the same mean stress.

A graphical representation of the state parameter is given in Figure 2.24. A negative \( \Psi \) means that a soil will have a dilative behaviour, while a positive \( \Psi \) is
related to a loose specimen susceptible to liquefaction. Soils that exhibit the same $\Psi$ are equidistant from the CSL, and consequently they should exhibit similar behaviours (Been and Jefferies, 1985). When subjected to cyclic shear stress any type of soil will contract, causing a progressive development of positive pore water pressure under undrained conditions regardless of the position of the state parameter (Soares, 2015), but that amount and rate of increase of pore pressure may well be related to the $\Psi$, as has been examined in this thesis.

2.6.5 Analysis of Previous Work

In order to analyse the work carried out in this research, an overview on what research has already been done needs to be made; the CSL is a straight line in many cases, but for the tests carried out in this research, the CSLs are likely to be curved, as it is for most sands and silts. For example, some curvature can be seen for low stress levels for Dog’s Bay carbonate sand in Figure 2.23. As the stresses increase it becomes more linear. Carrera et al., (2011) carried out tests to compare the void ratios at which a number of different soils reached liquefaction and concluded that clean sand was less easily liquefied than material containing a higher silt content. The CSLs of the fluorite tailings from Stava are presented in Figure 2.25. These move downwards initially as the silt content is increased, then they move back upwards, as indicated by the arrows on the graph.

The stress paths and the locus of the maximum deviator stress, $q_{max}$, from tests on clean sand are shown in Figure 2.26. For a curved CSL, a positive state parameter necessarily gives liquefaction at low stress levels, whereas the same positive parameter at high stresses will not generate liquefaction (Carrera et al., 2011). A schematic diagram and data showing the changing susceptibility to liquefaction of tailings with a curved CSL is displayed in the insert of Figure 2.27. Above the horizontal asymptote
\( p' \) goes to zero if the conditions are undrained. The ‘instability/liquefaction zone’ in Figure 2.27 corresponds with the low stress levels and the shear stress levels moving to \( p' = 0 \) in Figure 2.26. Stress levels are higher in the ‘compressive, strain softening zone’, and void ratios become lower. There is a high degree of strain softening as the distance to the CSL is significant, but movement stops at the CSL. Soils in this zone are referred to by many authors as undergoing ‘flow liquefaction’ (e.g., Lade and Yamamuro, 1997; Bedin et al., 2012) but this type of strain softening cannot explain the complete loss of strength seen at Stava, Fundão or Brumadinho where the tailings were liquid post failure. The highest stress levels are reached in the ‘compressive and generally strain hardening zone’ where only a small amount of strain softening occurs.

The maximum and minimum void ratios for a variety of tailings were examined by Li and Coop (2019). Figure 2.28 displays the horizontal asymptote of the CSLs. This is known as \( v_{\text{liq}} \) and is the specific volume above which full static liquefaction occurred on undrained loading. Other than one result from Bedin et al., (2012), the results are similar. The maximum and minimum attainable values of \( v, v_{\text{max}} \) and \( v_{\text{min}} \), reduce and increase together. The samples get denser as the fines content increases as the fines are filling the voids, but as the fines content continues to increase, the voids become full and the samples become looser. Also, the location of \( v_{\text{liq}} \) is always approximately in the middle of \( v_{\text{min}} \) and \( v_{\text{max}} \) so the tendency to liquefaction does not change much. Li and Coop (2019) examined the susceptibility to liquefaction for the pond, lower beach and upper beach of the Panzihua tailings dam (see Chapter 4). The tailings in the upper beach were most prone to liquefaction as the value of \( v_{\text{liq}} \) is lowest relative to \( v_{\text{min}} \) and \( v_{\text{max}} \), followed by the middle beach and then the pond material. The other data presented were too scattered to form a firm conclusion. This has been found for static liquefaction but not cyclic liquefaction.
López-Querol and Coop (2012) examined the results of drained cyclic triaxial tests performed on Dogs Bay sand (a carbonate sand). They carried out drained cyclic tests on loose samples, applying different amplitudes of cyclic loading for samples that generally had initial states on the wet side of the CSL. Eight sand samples, with different initial relative densities were tested. Figure 2.29 shows the values of change of state parameter against the initial state parameter for the eight tests and for the amplitude of cyclic loading ratio ($\beta = \Delta q / p_i'$, where $\Delta q$ is the amplitude of cyclic deviatoric stress) values of 0.1, 0.2 and 0.3. López-Querol and Coop (2012) concluded that the change of state parameter, which is related to the volumetric strain, depends more or less linearly on the initial state parameter and also on the amplitude of the cyclic loading. The initial state parameter value controlled the volume change and so the behaviour depended on the state parameter. But all were compressive whether or not the initial $\Psi$ was positive or negative. Less compression was seen when the $\Psi$ was negative, and more compression was seen when the $\Psi$ was positive.

The same carbonate sand was tested by Qadimi and Coop (2007). They carried out undrained cyclic tests. Samples were isotropically compressed and either reached line B, line A or the NCL, as seen in Figure 2.30. Lines A and B are parallel to the NCL and have different $\Psi$. When divided by $p_i'$ the same pore pressure responses are found for the initial states located on line A. The same can be said for line B and the NCL. This was done for a straight CSL. The outcomes are expected to change for a curved CSL, as two points could have the same state parameter value, but the behaviour could be different at different stress levels. A thorough analysis of the use of $\Psi$ for curved CSLs is presented in Chapter 4.
2.6.6 Cyclic Stress Ratio

The liquefaction resistance of a soil depends on how close the initial state is to the state corresponding to ‘failure’ and on the nature of the loading required to move it from the initial state to the failure state (Kramer, 1996). The analysis of liquefaction susceptibility is commonly made by using the cyclic stress ratio (CSR). Depending on the type of test, the CSR is defined differently. The CSR was proposed by Seed and Idriss (1971) and for cyclic loading the equation is as follows:

\[ CSR = \frac{q}{2\sigma'_c} \]  

(2.2)

where \( q \) is the difference between the minimum and maximum cyclic stresses, and \( \sigma'_c \) is the initial effective confining pressure. This research focused on three CSRs. How they were chosen is discussed in Chapter 5.

A soil’s resistance to liquefaction is generally represented by cyclic resistance curves which relate to the CSR with the number of cycles required for liquefaction to occur (e.g., Ramos, 2021). An example of the use of CSR is given in a study by Isihara (1985), where cyclic torsion tests were carried out. Figure 2.31 shows the number of cycles to reach a given shear strain. Tests were performed under the same initial state conditions at different CSRs. It can be seen that the number of cycles to reach failure increases as the CSR decreases.

Lei et al., (2017) examined the effects of the frequency and CSR of cyclic loading on the behaviour of a natural clay in China using triaxial tests. They found that with increases in CSR, soil failure occurs at gradually decreasing effective axial stresses. Figure 2.32 shows the triaxial test curves that were obtained at several CSRs, defined here as \( r \). The clay failed at different values of axial stress at different \( r \) values. With increasing \( r \), failure occurred at lower axial stresses. For example, when \( r \) was 0.133,
failure occurred at an axial stress of 550 kPa, and when \( r \) was 0.267, the soil failed at 350 kPa. With an increase in the \( r \) or CSR, the energy that is transmitted to the soil via the axial dynamic loading increases. A large CSR and a small effective axial stress could result in soil failure.

2.6.7  Cyclic Resistance Ratio

The capacity of a soil to resist liquefaction corresponds to the CSR required to trigger liquefaction and is represented by the cyclic resistance ratio (CRR) (Seed and Idriss, 1971). A CRR curve is used to identify where liquefaction occurred. Jefferies and Been (2016) presented a relationship between the CRR for failure in 15 cycles (CRR\(_{15}\)) and the state parameter, \( \Psi \), from tests on a variety of sands (Figure 2.33). There is some scatter in the data as the \( \Psi \) is not the only parameter that affects cyclic behaviour, but it can generally be seen that as \( \Psi \) increases, CRR\(_{15}\) decreases. The resistance to liquefaction generally decreases as the \( \Psi \) increases.

2.6.8  Transitional Behaviour

Many natural soils of complex mineralogy and/or mixed grading have modes of behaviour that differ from conventional soil mechanics in that the effects of the \( e_i \) at their deposition cannot be erased by loading at engineering stress and strain levels. While most soils may have well defined NCLs and CSLs in the \( e:lnp' \) or \( v:lnp' \) planes, for these materials the locations of these lines are strongly a function of the \( e_i \) at the point they are deposited or created (Martins et al., 2001; Ferreira and Bica, 2006). While such effects must be a function of the soil fabric, identifying where is the seat of that fabric and at what scale has proven elusive, although it is often unrelated to the means of sample preparation in laboratory prepared reconstituted samples (Shipton and Coop, 2015). The types of soils it can be found in are wide ranging, but typically it does not occur in poorly graded sands or the more plastic soils, but in soils that are
well and perhaps finer graded and often with complex mineralogy (Shipton and Coop, 2012). Particle breakage may or may not occur in soils with this behaviour. All soils must eventually reach a critical state with unique stresses, volumes and fabric if they are sheared far enough, so the absence of such unique states in these soils merely indicates that the strains applied by our laboratory tests (and around our engineering structures) are far too small to reach them (Todisco and Coop, 2019).

The characteristics of tailings are highly variable depending on the composition of the ores and the extraction processes used. From their gradings and mineralogies, it might be expected that some or even many tailings would have transitional behaviour. Unfortunately, researchers and engineers rarely look for this behaviour since it requires tests to be carried out from samples with a range of different starting void ratios at the point of setting the sample up, which is laborious, and researchers generally follow set procedures with a fixed $e_i$.

To quantify transitional behaviour oedometer and triaxial tests are carried out. Test results identify whether or not non-convergent compression behaviour exists. Ponzoni et al., (2014) identified that for soils with fully convergent paths the gradients of the data in Figure 2.34 (a) and (b), defined here as $m$, would be zero, and for soils with perfectly parallel compression paths $m$ would equal one. Ponzoni et al., (2014) found $m$ values of 0.40 and 0.38 from tests on silts and silty clays from the lagoon of Venice indicating moderate transitional behaviour.

Li and Coop (2019) and Li et al., (2018) did look for transitional behaviour in tailings and while they did not find it in iron ore tailings of various gradings, they did find a weak transitional behaviour in a gold tailings. The silt-sized gold tailings was found to have an $m$ value of 0.13 at 7 MPa from oedometer tests indicating slight
transitional behaviour (Fig 2.35). The tailings had a median particle size, $d_{50}$, of 0.011 mm and a coefficient of uniformity, $C_u$, of 7.3 so was fine and quite poorly graded. The oedometer curves only converged at very high stress levels, and convergence was slow, so their tailings was not strongly transitional, however, a unique NCL was not reached, even at high stresses (Fig. 2.36).

Shipton and Coop (2015) carried out oedometer and triaxial tests with a wide range of specific volumes for 75% sand and 25% kaolin mixed samples. Figure 2.37 displays the compression data from these tests. The oedometer data clearly shows that even if the higher specific volume samples have steeper compression paths, the convergence of the paths from different initial specific volumes is not rapid enough for there to be a unique NCL. There is also a lack of convergence seen in the triaxial data. Figure 2.38 shows the triaxial data for samples made with five different but similar initial specific volumes. The CSLs in triaxial compression for each sample are all separate and parallel, indicating transitional behaviour. This group of CSLs indicates that the initial specific volume controls the location of the current CSL.

Oedometer tests were carried out on a 75% quartz sand and 25% kaolin mix and a Botucatu residual sandstone from Brazil by Martins et al., (2001). The sand-kaolin mix was used to simulate the behaviour of reconstituted Botucatu residual sandstone samples. Figure 2.39 displays the lack of convergence identified in the tests. The compressibility of the sand-kaolin mix is smaller than that of the residual sandstone, but the curves do not converge, even at high loads. The sand-kaolin mix reflected the gap grading of the residual sandstone and Martins et al., (2001) concluded that the transitional behaviour did not result from the complex mineralogy arising from the weathered origin of the residual sandstone, but simply came from the grading.
2.7 Effects of Sample Preparation Methods

2.7.1 Introduction

The term ‘fabric’ defines the arrangement of particles of all size ranges, shapes and associated pores (Burland, 1990). Some authors agree that the initial fabric from sample preparation has a significant effect on the mechanical behaviour at small to medium strain levels, and this will be examined. A variety of sample preparation methods are investigated in the literature to see how this initial fabric affects mechanical behaviour. Some preparation methods have been replicated using iron tailings samples in preliminary tests. The samples were cyclically loaded to analyse how sample preparation affects the number of cycles to failure.

2.7.2 Sample Preparation Methods – An Introduction

Sampling of undisturbed tailings samples is difficult and expensive and as a result, laboratory testing is often carried out on reconstituted samples. A variety of preparation techniques for soils have been developed over the years including; the slurry deposition method, moist tamping, dry pluviation and wet pluviation. It is important to consider which methods can more closely simulate the structure and the actual stress-strain response of the soil being modelled. Zlatovic and Ishihara (1997) carried out undrained tests on loose silty sands, Nevada sands and Toyoura sands. They observed that the shear response was significantly affected by preparation methods at low strain levels. Figure 2.40 gives the $e:lnp$ graph after consolidation for Lagunillas sandy silt, prepared using the moist placement (MP), dry deposition (DD) and water sedimentation (WS) methods. The initial compression lines for the three fabrics are distinctly different. Based on the tests on the Stava tailings by Carrera at al., (2011), all shearing responses converged to a unique CSL independent of the
preparation method, but these tests were carried out at high strain levels and the effects of the initial fabric may therefore have been erased.

Sze and Yang (2014) conducted many laboratory tests and consistently found that two specimens of a sand prepared by different methods had different responses to applied monotonic loading under otherwise similar conditions. Different fabrics were created by the different methods. Samples constructed through different reconstitution techniques have proven to produce distinct undrained shear responses. This means that the in-situ soil strength might be over or under predicted when using reconstituted samples (Soares, 2015). A number of experimental studies have produced data showing that the liquefaction resistance of saturated sand specimens reconstituted by different compaction methods to the same density can be significantly different. Sze and Yang (2014) found that the method of specimen preparation or the fabric it forms plays an important role in the nature of sand response to cyclic loading. Figure 2.41 displays the relationship between the excess pore water pressure and the number of cycles. The dry deposition samples were more susceptible to liquefaction in cyclic loading than the moist tamping samples. This was the case for loose state samples and medium-dense state samples. Mulilis et al., (1977) conducted a series of cyclic triaxial tests and found that sand specimens prepared by moist tamping exhibited a much higher resistance to liquefaction, or CRR, than the samples created by air pluviation (Fig. 2.42). It can be seen in the figure that there is a significant difference in the CSR causing initial liquefaction for samples of Monterey sand formed by the different compaction procedures, even though the initial densities were the same.

Silver et al., (1980) compared cyclic triaxial and simple shear test results in specimens reconstituted by moist tamping and air pluviation (Figure 2.43). The figure shows that the sample preparation method did not affect the resistance of the simple
shear tests, however, samples subjected to triaxial testing prepared by moist tamping displayed higher resistance than air-pluviated prepared samples. Triaxial tests on the air-pluviated samples showed similar resistance to the simple shear samples.

2.7.3 Slurry Deposition Method

The slurry method shall be considered initially. When using a consolidometer to make a slurry sample various problems can occur; the samples can liquefy when being pushed out of the mould but even if they don’t, the sample can be disturbed as it needs to be carried to the triaxial cell and mounted by hand. Ideally, specimens should be moulded directly on the base of the cell, to avoid the need for transferring. Dr. Antonio Carraro, from Imperial College London, developed a new slurry-based preparation method (Carraro and Prezzi, 2008). A schematic representation of the method is given in Figure 2.44. The method includes a triaxial mould as part of the slurry deposition mixing tube, therefore avoiding transferring. A photograph of the apparatus Carraro developed is shown in Figure 2.45. The perspex mould sits on top of the split mould so dry tailings can be poured into distilled water or tailings water. The tailings are then shaken vigorously before being allowed to settle. Following this, the specimen is saturated, consolidated and sheared. The advantages of this method are that the preparation time is short, being only 2-3 hours for the tailings they tested, and also a uniform sample can be created consistently. There is also little segregation and the sample is already in an initial stage of saturation. Carraro and Prezzi (2008) found that the slurry deposition method avoids segregation of the fines, and produces the same type of fabric and stress-strain response of wet pluviation specimens. The slurry deposition and wet pluviation methods were examined in the preliminary tests presented in Chapter 4.
Laboratory tests were conducted by Chang et al., (2011) to investigate the effect of fabric on gold tailings using SEM (scanning electron microscopy). They prepared slurry samples on the triaxial pedestal and their method involved the deposition of the slurry in thin layers, stirring each layer after deposition to reduce segregation. They found that slurry deposition produced a uniform fabric which was similar to that of the undisturbed samples (Fig. 2.46). Murthy et al., (2007) also commented that the slurry deposition method ensures reasonably homogenous and reproducible specimens, and can recreate the in-situ fabric of most soils prone to liquefaction, such as tailings dams.

2.7.4 Moist Tamping

For the next three methods, a split mould is mounted on the triaxial pedestal. The membrane is placed over the greased base, and then stretched over the outside of the top of the split mould and a vacuum is applied to hold the membrane against the mould. At this stage the methods used to create the sample inside the mould can be different;

Moist tamping involves the compaction of moist material in a split mould to the required density or void ratio. The moist material is spooned into the split mould and levelled. Each layer is compacted using a hand-held tamper. The density of the specimen can be controlled by adjusting the weight and volume of the material placed in each layer (e.g., Frost and Park, 2003). Each layer is typically compacted to a lower density than the final desired value by a predetermined amount which is defined as under-compaction (Selig and Ladd, 1978). This ensures a uniform final void ratio. Among the sample reconstitution techniques, moist tamping is the most practiced for non-plastic soils.

Moist tamping was the most practical and preferred technique for Carrera (2008), but this technique can produce a potentially collapsible fabric which has often been
criticised (Vaid and Sivathayalan, 2000). There are problems with this technique as it enables specimen reconstitution at void ratios that may be too high and thus not possible for natural soil deposits formed underwater (Carraro and Prezzi, 2008). The method has also been shown to yield less uniform specimens (Frost and Park, 2003) and is responsible for creating a metastable honeycomb structure produced by capillarity, which is more prone to liquefaction (Soares, 2015).

Tests carried out by Chang et al., (2011) were found to liquefy using the moist tamping method. This was because the method produced a flocculated fabric, where bunches of finer material are found bridging the bulkier particles (Fig. 2.47). SEM images of the gold tailings taken from the pond, middle beach and upper beach were tested using the slurry and moist tamping reconstitution methods. These are summarised in Figure 2.46 along with undisturbed samples. It appears that the preparation method had little effect on the fabric of the pond samples because they mainly consist of small particles. The fabric of the middle and upper beach samples were, however, affected by the method. The images of both the undisturbed and slurry samples show a uniform fabric with platy particles dispersed around and in between the bulky particles. Chang et al., (2011) therefore concluded that the slurry deposition method was the most appropriate.

2.7.5 Dry Pluviation

For dry pluviation, also known as air pluviation, the sample is allowed to rain freely from a known height into the split mould. After the dry material has overfilled the mould, the excess is collected, and the weight of the material in the mould can be calculated. The void ratio of a sample prepared by dry pluviation depends on such variables as the opening size and the height from which the sand fell (e.g., De Gregorio, 1990).
Della et al., (2011) commented that the specimens prepared by the dry funnel pluviation method tend to be more resistant than those reconstituted by the moist tamping method. Figure 2.48 shows the variation of the undrained shear strength at peak with the effective confining stress using these two methods of deposition. It can be seen that the dry funnel pluviation method shows higher values of the deviator stress at peak, therefore a much higher resistance to liquefaction. The dry funnel pluviation method indicated a more stable response, while the moist tamping method exhibited more unstable behaviour because of the unstable fabric. Mulilis et al., (1977) tested this method and their results were in disagreement with this. From their study the samples prepared by wet tamping presented a liquefaction resistance higher than those prepared by dry pluviation.

2.7.6 Wet Pluviation

For wet pluviation, the inside of the membrane is filled with distilled water. The dry sample is submerged in distilled water and de-aired. This is then spooned into the mould carefully, ensuring the sample remains underwater at all times. This method is described in more detail in Section 3.2.3.

Tests were carried out by Carraro and Prezzi (2008) using this method and it was found that saturated specimens were produced that were easy to replicate and had fabric and behaviour similar to that of natural alluvial soils. However, Selig and Ladd (1978) commented that there are several problems with this method. The two most significant are the segregation of particles when using silty and relatively well graded sands, and the difficulty of readily preparing test specimens having a prescribed dry unit weight with uniform density. They concluded that a more precise means of preparing specimens is needed so that cyclic results will be consistent, repeatable, and less influenced by sample preparation.
2.7.7 Overview of Sample Preparation Methods

From looking at the various literature on sample preparation methods, no consistent conclusion can be obtained, and many conflicting results have been presented. However, most authors agree that the initial fabric from sample preparation has a significant effect on the mechanical behaviour at small to medium strains. There is still much debate over the preferred sample preparation method.

It has been demonstrated by Chang et al., (2011) that the slurry deposition method produces samples that better replicate the in-situ behaviour of tailings. It can be seen in Figure 2.49 that neither moist tamping nor slurry deposition fully replicates the behaviour of the undisturbed gold tailings sample, but the slurry method can be replicated easily and produces uniform specimens. However, it is very difficult to obtain slurry deposited specimens that are very loose. Moist tamping, on the other hand, can be used to obtain tailings specimens with large initial void ratios or with initial soil states lying above the CSL (Murthy et al., 2007). Slurry preparation generally replicates the fabric and behaviour of the tailings better than moist tamping, but neither fully replicates the undisturbed sample.

The pluviation methods are relatively easy to carry out but come with many problems. These include the segregation of particles and the difficulty in preparing specimens with a defined dry unit weight and density. The methods are not very precise and the dry method produces a fabric that can be highly contractive (Carraro and Prezzi, 2008).

Overall, for tailings materials, the literature suggests that the reconstitution techniques which may produce a same shear strength behaviour as the intact specimens were the slurry deposition methods. These methods simulate best the in-
situ fabric and stress-strain response of most liquefiable soil deposits of sands with or without fines.

Shipton and Coop (2015) carried out an extensive investigation on the effects of sample preparation for a transitional soil. Figure 2.50 gives the intercept of CSLs for 75% quartz sand and 25% kaolin mixtures for a range of sample preparation methods. The initial specific volume has been taken at 20 kPa, which was the start of isotropic compression in the triaxial tests, and the locations of the CSLs are quantified by $\Gamma$, the intercept at 1 kPa. The resulting relationship indicates a direct and linear influence of $v_{20}$ on $\Gamma$. Different preparation methods have been used, but the general trend remains the same. In this case transitional behaviour is therefore not related to the preparation method.

2.8 Literature Review Overview

A detailed literature review was carried out before formulating a laboratory test programme to achieve the objectives of this research which are discussed in Chapter 5. Carrera (2008) focused on the static liquefaction of one type of tailings. This research focused on the cyclic loading of tailings from different locations and comparisons were made.

With regards to sample preparation methods, it was discussed that the fabric is important in modelling the behaviour of tailings. In almost all soil experiments, stress-strain responses can be greatly influenced by different sample reconstitution methods, which generate different fabrics and structures in soil samples. Although there have been numerous studies on this aspect, many findings are contradictory so it is
important to examine this. One of the aims of this study was to therefore investigate fabric effects through a variety of sample preparation methods.

For tailings, advances and contradictions have developed simultaneously. The basic properties and strengths of mine tailings have been extensively measured in the laboratory, but in-situ static and dynamic behaviour have not adequately been determined by researchers. Investigation has been lacking into the fundamental aspects of cyclic behaviour, such as how the method of sample preparation affects the nature and characteristics of the cyclic response of the soil, including the deformation pattern, pore water pressure generation, effective stress path, and stress-strain relationship. More importantly, there is currently no overall framework for curved CSLs in the $v:\ln p'$ plane for undrained cyclic loading.

There is significant evidence of transitional behaviour in some soils in the literature, but how transitional behaviour affects susceptibility to cyclic liquefaction is missing. Examining this was therefore the main objective of this thesis, and results are discussed in Chapter 5.
Figure 2.1 - Raising a tailings dam using the upstream method (Earth Resources, n.d.).

Figure 2.2 - Raising a tailings dam using the upstream method (Earth Resources, n.d.).

Figure 2.3 - Raising a tailings dam using the centreline method (Earth Resources, n.d.).
Figure 2.4 - Example of a tailings dam profile (modified by the author from Vanden Berghe et al., 2011).

Figure 2.5 - Deposition methods of tailings (a) Hydro-cyclone, (b) Spray bar, (c) Spigot (Soares, 2015).
Figure 2.6 - Distribution of the number of incidents (%) according to cause in the world and in Europe from 147 historical tailings dam failure cases (Rico et al., 2008).

Figure 2.7 - Schematic diagram displaying contractive and dilative shear behaviour during undrained monotonic loading and cyclic mobility during cyclic loading (Hyde et al., 2006).
Figure 2.8 - An example of cyclic liquefaction in a loose granular soil subjected to undrained cyclic simple shear Loading (Ramos, 2021).

Figure 2.9 - The buildings affected by the Fundão tailings dam: (A) District of Bento Rodrigues, Mariana, (B) Urban area of the municipality of Barra Longa (Carmo et al., 2017).
Figure 2.10 - Impact of the Fundão tailings on the Doce River: (A) The river post-failure, (B) The effect the disaster had on marine life, (C) Dead fish at Governador Valadares, (D) Aerial photograph of the Doce River 25 days post-failure (Fernandes et al., 2016).

Figure 2.11 - The moment the Brumadinho dam collapsed (France 24, 2019).
Figure 2.12 - Aerial view of the Stava dam before failure (Chandler and Tosatti, 1995).
Figure 2.13 - Two aerial views of the breached Aznalcóllar dam a few hours after the failure: (a) view from the east; (b) detail of the breach (Alonso and Gens, 2006).
Figure 2.14 - Model for the motion of the Aznalcóllar dam slide (Alonso and Gens, 2006).

Figure 2.15 - Aerial view of the Merriespruit tailings dam failure showing the path of the destructive mudflow (Fourie et al., 2001).
Figure 2.16 - Flow path and damage caused by the liquefied tailings from the Merriespruit tailings dam (Van Niekerk and Viljoen, 2005).
Figure 2.17 - The rupture zone of the 2010 Chile Earthquake (Verdugo and González, 2015).

Figure 2.18 - Before and after the failure of the Las Palmas tailings dam (Verdugo and González, 2015).
Figure 2.19 - Tailings flowed 500m downstream after the Las Palmas dam failure (Villavicencio et al., 2014).

Figure 2.20 - Sand craters in the Las Palmas tailings basin (Villavicencio et al., 2014).
Figure 2.21 - Location of the Tapo Canyon tailings dam site (Harder and Stewart, 1996).

Figure 2.22 - Liquefaction susceptibility (Been and Jefferies, 1985).
Figure 2.23 - The NCL and CSL in \( v \cdot \ln p' \) space for Dog’s Bay sand (López-Querol and Coop, 2012).

Figure 2.24 - Definition of state parameter, \( \Psi \) (Been and Jefferies, 1985).
Figure 2.25 - Curved CSLs of the fluorite tailings from Stava with different percentages of silt and sand (Carrera et al., 2011).

Figure 2.26 - Stress paths and locus of $q_{\text{max}}$ from tests on clean sand that showed strain softening (Carrera et al., 2011).
Figure 2.27 - Schematic diagram showing the changing susceptibility to liquefaction of tailings with a curved CSL (Carrera et al., 2011).

Figure 2.28 - The relationship of the $v_{liq}$, $v_{max}$ and $v_{min}$ with fines content for a variety of tailings (modified by the author from Li and Coop, 2019).
Figure 2.29 - Change of state parameter after equilibrium ($\beta = 0.1$, 0.2 and 0.3) against initial state parameter for drained cyclic triaxial tests on carbonate sand (López-Querol and Cooper, 2012).
Figure 2.30 - Pore pressure changes in carbonate sands for states on lines parallel to the isotropic NCL for undrained cyclic tests: (a) initial states; (b) normalised pore pressure. $p_i'$ is the initial $p'$ at the start of cyclic loading (Qadimi and Coop, 2007).
Figure 2.31 - Examples of cyclic resistance curves (adapted by Ramos, 2001 from Ishihara, 1985).

Figure 2.32 - Triaxial tests under different CSRs (Lei et al., 2017).
Figure 2.33 - CRR for failure in 15 cycles as a function of the state parameter (Jefferies and Been, 2016).

Figure 2.34 - Quantification of the convergence of compression curves for intact specimens: (a) schematic compression curves; (b) calculation of $m$ (Ponzoni et al., 2014).
Figure 2.35 - Calculation of $m$ values for the tests with a maximum vertical stress of 7 MPa (Li et al., 2018).

Figure 2.36 - One-dimensional compression behaviour of a gold tailings (Li et al., 2018).
Figure 2.37 - Compression lines of 75% quartz sand and 25% kaolin mixture samples (Shipton and Coop, 2015).

Figure 2.38 - Triaxial data for the 75% sand and 25% kaolin samples displaying five groups of specific volume (Shipton and Coop, 2015).
Figure 2.39 - Oedometer data for Botucatu residual sandstone from Brazil and a sand-kaolin mixture (Martins et al., 2001).

Figure 2.40 - Initial compression lines of Lagunillas sandy silt, corresponding to three methods of sample preparation (Zlatovic and Ishihara, 1997).
Figure 2.41 - The impact of sample preparation on pore water pressure build-up during cyclic loading: (a and b) loose state; (c and d) medium-dense state (Sze and Yang, 2014).

Figure 2.42 - The CSR at ten cycles for the initial liquefaction versus initial effective confining pressure for samples created with three different compaction methods (Mulilis et al., 1977).
Figure 2.43 - Stress ratio plotted against the number of loading cycles to 15% double amplitude strain for wet tamped and air-pluviated Monterey sand (Silver et al., 1980).
Figure 2.44 - Schematic representation of Carraro's (2008) proposed method of reconstitution of specimens of sands containing fines.
Figure 2.45 - Slurry-based preparation method at Imperial College London (Carraro, 2018).

Figure 2.46 - A summary of undisturbed, slurry and moist tamped gold tailings fabrics (Chang et al., 2011).
Figure 2.47 - Gold tailings displaying a flocked fabric after moist tamping (Chang et al., 2011).

Figure 2.48 - Effect of the deposition method on the undrained shear strength at the peak (Della et al., 2011).
Figure 2.49 - Stress paths of middle beach undisturbed, slurry and moist tamped samples (Chang et al., 2011).

Figure 2.50 - Intercept of CSLs for 75% quartz sand and 25% kaolin mixture (Shipton and Coop, 2015).
3. Apparatus, Materials and Testing Procedures

3.1 Introduction

This chapter outlines the equipment and procedures used for the laboratory work conducted in this study, along with an introduction on the tailings selected for investigation. All tests were performed in the soils laboratory at University College London.

The chapter begins with a description of the equipment used, followed by the sample preparation methods and testing procedures. A series of experimental tests, mainly triaxial and oedometer tests, were carried out in this research. The chapter also covers the equations used for data analysis and details the processes required for correcting data where applicable. The Morphologi G3 particle characterisation system was used to create grading curves and analyse particle size and shape and this is discussed in Section 3.5. An overview of how the materials for this study were obtained and where they came from is given in the following section. Three materials were tested in total. These were an iron tailings from the Wanniangou tailings dam in Panzihua City, China, a mineral sand tailings excavated from a mine about 100 km South of Perth, Australia, and an iron tailings from Sweden. The chapter concludes with the procedures used for material characterisation.

3.2 Triaxial Apparatus

3.2.1 Introduction

The triaxial compression test is the most common and versatile test for examining the stress-strain and strength properties of soils. A Bishop and Wesley triaxial cell was used in this study to investigate the susceptibility to cyclic liquefaction and also to see
whether or not there were well defined normal compression lines (NCLs) and critical state lines (CSLs). Photographs of the apparatus are given in Figures 3.1 and 3.2 and a schematic diagram of a standard Bishop and Wesley triaxial apparatus is given in Figure 3.3.

The triaxial cell used in this research was connected to a loading system operated using air pressure. An analogue to digital (AD) converter was used to measure the stresses and strains from the various transducers through a computer program called TRIAX (Fig. 3.4). The apparatus controls stresses automatically by the computer through three air pressure controllers (cell, back and ram pressure), each fed with the standard laboratory compressed air supply. The maximum confining pressure that the apparatus could provide was 800 kPa. The ram and cell pressures were connected to air-water bladder cylinders, which converted air pressure into water pressure. All pressures were regulated by stepping motor driven valves, controlled by TRIAX. There was also a water supply system used to fill the chamber with water, after setting up a sample, and to control the water volume in the volume gauge. A linear variable differential transformer (LVDT) was attached to the 50 cm³ volume gauge, which measured the volume change during consolidation and drained shearing. The volume change was measured by water flow, so the sample needs to be fully saturated. The volume gauge was always left under pressure, even when there was no test taking place, to avoid it desaturating. Also, when there was no sample in the cell, the cell was left filled with water to ensure no air bubbles became trapped in the drainage lines.

The pedestal holding the sample moved up by increasing the ram pressure until the sample reached the load cell. The deviator stress could be measured once contact had been made which was measured by the load cell located above the sample. The cell was also connected to a constant rate of strain pump (CRSP) which regulated the
ram displacement for strain-controlled tests. This was composed of a stepping motor that drove a screw piston up and down. When the screw moved upwards, water was pumped into the ram and pushed the sample up at a constant rate. The axial strains were measured externally with an LVDT mounted outside the pressure chamber. The maximum range of this LVDT was 50 mm, which was enough to reach the required deformations. Two local axial LVDTs were mounted to the specimens axially, in the final year of testing, to attempt to enhance the quality of the test data. The LVDTs were set to their electrical zero by adjusting the transducer amplifier while there was no armature in the LVDT at the beginning of every test. After zeroing, they were attached directly onto the sample with mounts that were superglued on at two opposite sides (Fig. 3.5). This was a difficult task as the weight of the LVDTs caused the membrane to come away from the sample slightly. The initial distances between the middle of the two mounts (approximately 50 mm) were measured three times each with a caliper and averages were taken to calculate the gauge lengths. The calibrated range for these LVDTs was 12 mm.

All samples were cylindrical and the majority of the samples had a diameter of 38 mm and a height of ~76 mm. It was not quite possible to reach a height of 76 mm for the samples made on the platen as the split mould determined the height, and samples could not be made taller than the height of the split mould. These samples had a height of ~74-75 mm. The samples used for the tests carried out with lubricated end platens had diameters of 36 mm to be smaller than the 38 mm platens and so avoid overhang at large strains. More details on sample preparation methods are given below.

It is important to note that even though the soils laboratory was temperature-controlled, there were small temperature changes at about +/-1°C in the laboratory
depending on the time of day. These temperature fluctuations have had a small effect on the test data. It was found that the pore pressure typically increased in slight ‘waves’ over a 24-hour period during undrained cycling.

3.2.2 Testing Procedure for Samples Made in the Consolidometer

Higher water content slurries were first consolidated in a separate 38 mm consolidometer tube until they were stiff enough to be handled. Dense paste samples were also created using the consolidometer to see how the behaviour would compare with the loose samples. Different amounts of distilled water were added to ~300 g of dry tailings to correspond with the desired water content. A spatula was used to mix the tailings until the paste or slurry was homogeneous. This was then spooned into the consolidometer before weights were added in stages up to 5 kg. Weights were applied in approximately one-hour stages. A plastic ring was removed from the bottom section of the consolidometer after 24 hours to allow the sample to consolidate from the bottom as well as from the top. A dial gauge was added to the top of the consolidometer to identify when consolidation had ceased. This took approximately 3-4 days after the final load had been added. At this stage the sample was pushed out carefully from the consolidometer, and cut to the correct length using a wire saw.

A new slurry-based preparation method designed by Carraro and Prezzi (2008) was discussed in Section 2.7.3. The preparation time was approximately 2-3 hours for the tailings they tested. The preparation time using the tailings tested in this study would have been significantly longer, and so this method was not used. The samples would have taken much too long to consolidate on the platen.

Two preliminary tests were carried out on mixed slurry samples using the iron tailings from Panzihua City, China. The tailings were separated by sieving, and ~200
g of fine-grained material was mixed with ~200 g of coarse-grained material. The preparation method is then the same as above from the point the distilled water is added. Figure 3.6 shows one of the mixed samples being extruded from the consolidometer.

When carrying out a triaxial test using a consolidated slurry sample, the apparatus must first be de-aired. The cell pressure, back pressure and load cell readings were recorded at the end of a test when the cell was full of water. This was to check that the transducer readings had remained stable. Amendments were made to the previous test data if the readings were not zero. Half of the cell pressure, back pressure and load zero readings are either deducted or added to all of the readings exported in the data file. If the reading is negative then it was added, if it was positive then it was subtracted. The cell pressure, back pressure and load values were then re-zeroed for the next test whilst the cell was open to the atmosphere. The loading ram was moved all the way to the bottom by decreasing the ram pressure with the ram tap open. The cell was then drained and the cover removed. The drainage lines were de-aired by allowing water to flow through them for a few seconds until there were no more visible air bubbles. The top of the pedestal was left covered in water to prevent air entering the drainage lines. A porous stone was submerged in water in an open tin and de-aired using a vacuum pump. The base of the triaxial apparatus was cleaned and grease was applied to the O-ring around the base of the cell to ensure a tight seal and no leaks. Grease was also applied to the outside of the pedestal to avoid water infiltration under the membrane. The de-aired porous stone was placed on the base of the pedestal and a filter paper disc, cut to size, was placed on top.

The sample was gently pushed out of the consolidometer to the desired length and was then cut using a wire saw. The sample was placed directly on to the pedestal and
the top cap was added. The top cap was greased on its perimeter like the pedestal. Chinese lantern filter papers were added around all test samples to decrease the drainage path. Without them the consolidation time would be significantly longer. The thickness of the membrane was measured using a caliper and then it was stretched over the sample with a membrane stretcher. The membrane stretcher was also used to secure two O-rings around the membrane covering the pedestal and two O-rings around the membrane covering the top cap. The initial diameter and height of the sample were measured three times using a caliper. An average of these measurements was taken. Two times the width of the membrane was subtracted from the diameter of the sample.

The cell was bolted down and filled with water. The volume gauge was zeroed and cell pressure was manually adjusted to ~35 kPa and the back pressure to ~20 kPa. In order to keep the effective radial stress stable at the initial value of 15 kPa, the cell pressure and pore pressure were increased simultaneously, with a constant difference of 15 kPa. The saturation stage was started so that the cell pressure and pore pressure increased by 100 kPa over an hour and then stabilised. Specimens were saturated under low effective stress because once the sample was extruded from the consolidometer, the suction was largely lost and so a small effective stress was applied.

The pore pressure coefficient known as the B-value or Skempton B must be checked after this initial saturation stage. The ratio of the change in pore pressure, \( \Delta u \), and the change in cell pressure, \( \Delta \sigma_3 \), is calculated:

\[
B = \Delta u / \Delta \sigma_3
\]  

(3.1)

Ideally, the B-value should be 1, indicating a fully saturated sample, but a value of \( \geq 0.98 \) was used for the undrained cyclic testing (e.g., Hyde et al., 2006; Wichtmann et
al., 2010). If the value was found to be less than this, the cell and pore pressures were raised by another 100 kPa until a value of $\geq 0.98$ was achieved. From the literature, a B-value of $\geq 0.95$ was found to be acceptable for monotonic tests (e.g., Corrêa and Oliveira Filho, 2019; Li and Coop, 2019).

Once an appropriate B-value was reached, the effective stress was increased to 25 kPa. This was to give the sample more strength so that the sample did not liquefy when attaching the suction cap (Atkinson and Evans, 1985). A schematic diagram of the suction cap connection is given in Figure 3.7. A rubber suction cap was used to remove the initial tilting of the sample to improve the alignment of the sample and load cell. It was also required for cyclic loading as it allowed a negative deviatoric stress to be applied and also meant that the axial strain could be measured by an external LVDT. To connect the suction cap, a piston was attached to the line coming out of the top of the cell that is connected to the perspex adaptor attached to the load cell. This line is attached to a small tube that connects the suction cap to the outside which allows for the removal of water during connection. The ram pressure was increased manually so that it was equivalent to the cell pressure. The load cell reading was zeroed and then the load cell was wound down until there was about 1 N on the load value as the suction cap started to touch the adaptor. A dial gauge was then attached to the cell to monitor the axial displacement and the ram tap was opened. The axial ram pressure was increased manually by 10 steps of the motor at a time while the load was monitored on the computer. When the load reading was between 3 – 5 N, the wheel on the manual screw piston attached to the vent line was turned slowly in an anti-clockwise direction to remove water from between the suction cap and the extension piece on the load cell. The load cell reading was checked constantly to ensure there was no disturbance made to the sample. The suction cap was connected.
once the displacement dial on the loading ram stopped moving and this meant that the connection was at atmospheric pressure. Once the connection was made, the new sample height and diameter were calculated and entered into the TRIAX software. The volume gauge and internal LVDT transducers were then zeroed before isotropic compression commenced.

The next stage of the test was isotropic compression. The samples were isotropically consolidated to the desired effective stress by increasing the confining pressure by 3.5 kPa/hour whilst the back pressure remained constant and the deviatoric load was held at zero. This low rate was selected to avoid pore pressure build-up during compression and large amounts of creep after. According to López-Querol and Coop (2012), no cyclic loading should be applied until the volumetric strain due to creep is stable, with values smaller than 0.001%/h. All of the values here were less than or equal to 0.002%/h which was believed to be adequate in this case. Qadimi and Coop (2007) found that tests completed without reaching the required creep value would display larger excess pore pressures during undrained cyclic loading with the same number of cycles.

All samples were isotropically compressed for simplicity. It is important to note that soils in-situ are $K_0$ compressed, not isotropically, however, the overall aim of this study was to create a general framework and the stress-strain behaviour of the soils in any particular tailings dam or pond were not intended to be replicated.

At the end of isotropic compression, the sample height and diameter were once again recalculated and entered into the software. The volume gauge, LVDT transducers and computer timer were zeroed before starting the monotonic shearing or cyclic loading test. The monotonic shearing test process is described in more detail in
Section 3.2.6 and the cyclic loading test process in Section 3.2.7. At the end of the test, the suction cap was disconnected by gradually pushing water in between the adaptor and the suction cap using the manual screw piston, and by decreasing the ram pressure slowly. Once the ram was all the way at the bottom the ram tap was closed. The volumetric strain was then left to stabilise and a final volume gauge reading was taken. The pressures were decreased and the cell was drained. The local instrumentation was dismantled and the sample was carefully removed from the apparatus. The membrane was very carefully cut away from the sample. The sample was then oven-dried and weighed to determine the final water content. Any remaining sample residue left on the membrane was taken into account in the final water content.

3.2.3 Testing Procedure for Layered Samples

Two tests were carried out on reconstituted layered samples using the iron tailings from China. The deposition process behind tailings dams creates the layering (refer to Chapters 2 and 4). Tests were carried out to see what effect layering within the tailings structure might have on the behaviour, compared with similar homogeneous samples with the coarse and fine tailings mixed as might be created in laboratory reconstituted samples. The separated coarse material was not used in the consolidometer as it would be impossible to make a slurry sample using this method, so the wet pluviation method was used instead. The bottom half of the samples were constructed using the wet pluviation method, which is described in the paragraphs below. The consolidometer was used to make the top half of the sample using the finer-grained tailings. The consolidometer was also used to make the mixed samples. The mixed samples had the same constituents but were homogeneous.

When carrying out triaxial tests on the layered samples, the method is similar to the method above but there are some differences; a burette was attached to the drainage
line at the beginning of the experiment when the cell was filled with water. Water was allowed to flow through the tube attached to the burette and through the burette until there were no visible air bubbles left. The burette was used to apply suction so there was some strength in the coarse part of the sample before removing the split mould. Once the burette was attached and deaired, the water in the cell chamber was removed and the cell cover taken off. A membrane stretcher was used to stretch the membrane over the greased base of the pedestal and two base O-rings were applied. The split mould would not fit over the tube leading to the back pressure on the base of the triaxial pedestal (Fig 3.8), so this was altered. Once the alterations had been made, the joints of the split mould were greased along with the outside of the base of the membrane where the split mould sat. The split mould was placed around the membrane and the outside divide of the mould was greased and the membrane was folded over the top (Fig. 3.9). A jubilee clip was then tightened around it to stabilise and seal it. The membrane was filled with water and the height inside was measured with a caliper. This measurement was used to determine how far from the top of the split mould the pluviated sample needed to stop so that the total sample height was approximately 76 mm. A vacuum tube was attached to the split mould and the vacuum was turned on, giving about 10 kPa suction. This was enough to remove the air between the membrane and the mould. The membrane was then filled about 1 cm from the top with distilled water.

Approximately 100 g of dry, coarse tailings were poured into a bowl and covered with distilled water. The mixture was placed in a vacuum chamber and de-aired. The tailings were then carefully spooned into the membrane full of water, ensuring they were always kept underwater (Fig. 3.10). When the water level reached the top of the membrane, some of the water was siphoned out so the process could continue without
overflow. A caliper was used to ensure the pluviated sample height was approximately 38 mm. The tailings were allowed to settle and the rest of the excess water was carefully removed with a suction bottle. The bowl with the remaining tailings was placed in the oven for 24 hours and then weighed so that the weight of the sample in the apparatus could be calculated.

The consolidated sample was gently pushed out of the consolidometer and cut to a length of 38 mm with a wire saw. The sample was then trimmed (Fig. 3.11) as the diameter needed to be between 36-37 mm (Fig. 3.12) so it could slide into the membrane within the split mould. The trimmed sample was inserted into the membrane on top of the pluviated half and a slight pressure was applied to level it. The top cap was put on carefully and the membrane pulled over. O-rings were added to ensure a tight seal around the membrane, and the suction cap was positioned onto the top cap (Fig. 3.13). The vacuum and split mould could then be removed, and the sample could stand up by itself because a negative pore water pressure of -10 kPa was applied at the bottom drainage line with the burette. Figure 3.14 shows the layered sample after preparation. Dimensions were taken of both halves of the sample. The cell cover was then put on the apparatus, bolted down and filled with water, and the methodology followed from this point was as outlined in the test procedure above for slurry samples.

3.2.4 Test Procedure for Samples Made on the Platen

Dense pastes, made using the mineral sand tailings from Australia, could be placed directly into the membrane held on the platen inside a mould. The water content of these pastes was ~35%. The method for the samples made on the platen was the same for the procedure for layered samples up to the point where the membrane was folded over the top of the split mould. At this stage, the paste was spooned into the membrane
in the split mould and compacted gently in consistent layers using a flat piece of perspex rod. This was repeated until the paste reached the top of the split mould. This was then levelled with a ruler and the top cap was positioned on top. The four O-rings were then added using the membrane stretcher, and the greased suction cap was placed on the top cap.

3.2.5 Lubricated End Platens

Klotz (2000) and others have found that the main problem associated with the use of 2:1 (height to diameter ratio) samples and non-lubricated end platens is that shear planes are more likely to develop around the peak and so the volumetric strains are concentrated in a very small area of the sample. Non-lubricated ends also tend to give rise to a barrelled sample shape which is again a non-uniformity of strains during shearing. It was therefore decided to introduce lubricated end platens to see if these had any effect on the results.

Rowe and Barden (1964) were among the first to address this problem and introduced lubricated ends for their tests. They found that the 1:1 samples with lubricated end platens developed multiple shear bands and deformed much more uniformly. In contrast, the 2:1 samples with non-lubricated ends failed along a single shear band that developed following pronounced barrelling during the earlier stages of the test. Lubricated end platens were used to see if minimising the friction between the sample and end platens had any effect on the deformations and pore pressures generated during cycling and the number of cycles to failure.

The configuration chosen for lubricated end platens is essentially similar to that described by Head (1998). A new consolidometer was engineered so that 36 mm diameter samples could be made (Fig. 3.15). The method for making samples was the
same as that described in Section 3.2.2. The only difference was the weight applied because of the reduced diameter. Two polished steel plates were constructed with 38 mm diameters. Given the sample diameter of 36 mm, this configuration allowed for a radial strain of up to -5.6% before overhanging occurred. One steel plate was attached to the pedestal, and one to a newly crafted top cap. The steel plate attached to the pedestal initially had two small holes in it to allow for drainage.

A ~0.2 mm layer of grease was applied using a small paintbrush to each steel plate. This was then covered with a ~0.3 mm thick latex membrane disc. Each membrane had a diameter of 38 mm and eight short ~8 mm cuts were made in the radial direction from the edge of the membrane discs towards the centre to reduce the circumferential stiffness (Fig. 3.16). Two small holes were made in the disc that covered the polished steel plate that sat on the pedestal to allow for drainage.

The first lubricated end platen test was unsuccessful as the drainage holes in the polished steel plate on the platen became clogged with tailings. The absence of a porous stone meant that the drainage holes in the polished cap on the platen became blocked. This meant that the $p'$ value was not accurate because the pore pressure measurement was incorrect and therefore the effective stress was much lower than the required 200 kPa. Also, the volume gauge did not stabilise, even after a significant period of time because of the undissipated pore pressure. Because the effective stress was much lower than expected the test failed on the first cycle when cyclic testing commenced. The steel plate was therefore adapted to have a single small sintered bronze porous stone with a diameter of 6 mm placed in the middle of the plate (Fig. 3.17). This meant that water could flow through the porous stone to allow for drainage without it getting blocked with tailings. A small piece of filter paper was also added...
on top of the porous stone for each test. The membrane disc was also adapted to allow for drainage through this central porous stone (Fig. 3.18).

For lubricated end platen (LEP) tests, the samples were isotropically consolidated to the desired effective stress at 2 kPa/hour. This was slower than tests without lubricated end platens and the reason for this was to allow for time for drainage which was longer. Isotropic compression was also completed in stages and the volume gauge was tracked to ensure the final $p'$ value was accurate. One LEP cyclic test had a 10-minute cycle period which was twice the length of tests without lubricated end platens. This was to allow for the large pore pressure changes to be recorded accurately as the sample was expected to fail in <100 cycles. The second LEP cyclic test had a period of 5-minutes, as the test was predicted to not fail during the test period, and so the pore pressure changes would be small.

### 3.2.6 Monotonic Shearing

A number of drained and undrained monotonic tests were performed to evaluate the CSLs and NCLs of the tailings and to understand the behaviour when sheared. Once the required effective stress was achieved through isotropic compression, the shearing stage could begin. Prior to shearing, the volume gauge and LVDTs were zeroed and the new sample dimensions entered. The ram pressure tap was turned off and the CRSP turned on. The samples were then sheared using the CRSP at a rate of 0.1-0.2 mm/h until the ram had reached the end of its range.

### 3.2.7 Cyclic Testing

The resistance of tailings against cyclic liquefaction were evaluated in the laboratory by running undrained cyclic triaxial tests on reconstituted specimens. After isotropic compression, the drainage tap was closed prior to the start of cyclic testing to maintain
undrained conditions. A cyclic stress with a constant amplitude was then applied. Cycling continued in each test until the axial displacement reached +/-5 mm or the test reached ≥1700 cycles (approximately one week). ‘Failure’ was defined as the sample reaching +/-5% axial strain ($\varepsilon_a$). This was the point where the excess pore water pressure had reached, or was close to, the initial effective stress. It was unlikely that tests that completed ≥1700 cycles would ever reach failure.

The cyclic test functions were tested in the TRIAX software by carrying out a variety of tests on a consolidated sample to examine the wave shapes that could be created during cyclic testing. Fifteen tests were completed examining how the sine wave shapes were affected by different period lengths. The software did not work for triangular waves or square waves as the tests did not follow the shapes of these waves. Only sine waves were correctly followed. Sine waves were therefore selected for all tests. López-Querol and Coop (2012) commented that a three-minute total period was slow enough for their soils to ensure the stresses were homogeneous within the sample, with no undissipated pore pressures. The cycling was carried out slowly at one cycle every five minutes for the majority of the tests. From testing the software, the five-minute sine wave cycles were followed with very little error. The aim was not to replicate earthquake frequency, but to be able to track the pore pressure response accurately during the cycles. There was no evidence of undissipated or non-uniform pore pressures for this period and the pore pressure cycles followed the applied stress cycles closely. The load amplitude was selected according to the desired cyclic stress ratio (CSR). In total, 9 monotonic tests and 26 cyclic tests were successfully performed to establish a critical state soil mechanics (CSSM) framework.
3.2.8 Calibration

The LVDTs, load cell and volume gauge transducers were all calibrated to ensure all measured readings were accurate. The linear ranges of the transducers were determined by applying either known forces, displacements or pressures. To calibrate the LVDTs, a micrometer was used. The LVDTs were inserted into the micrometer and transducer readings were taken every 1 mm. The linear range for each LVDT was marked on each armature with a permanent marker. The calibrated range of the external LVDT was 50 mm, and the calibrated ranges of the internal local axial LVDTs was 12 mm. A calibrated manual screw piston was used for the volume gauge. Each full rotation of the piston equated to 1.10 ml. The load cell was calibrated by adding weights manually in increments and recording the load cell readings.

3.3 Data Analysis and Corrections

3.3.1 Void Ratio Calculations

Obtaining an accurate calculation of the initial specific volume, $v_i$, or initial void ratio, $e_i$, is fundamental in the CSSM framework, as it is a main factor that helps identify soil behaviour (e.g., Rocchi and Coop, 2014). It is imperative to distinguish whether any differences of void ratio are merely the result of errors in their measurement. For soil which fails at low or no shear stress, a soil can only liquify in monotonic loading if the void ratio, $e$, is equal to or greater than the void ratio for liquefaction, $e_{\text{liq}}$ (e.g., Carrera et al., 2011), which emphasises the need to measure $e$ accurately. There are seven formulae used by Rocchi and Coop (2014) to calculate the initial specific volume. Not all equations could be used as it could not be assumed that the samples were fully saturated at the start of the test and also because the tests were triaxial tests and not oedometer tests and so no final dimensions were calculated. Also, because of
the preparation methods adopted, some information used by Rocchi and Coop (2014) was not available. Each value was calculated in at least three different ways, using data from sample dimensions, wet weights, dry weights and water contents, before and after the test. This gives positive proof that each value of specific volume is accurate, and from these comparisons of methods it may be estimated that the typical error of specific volume in these tests was around +/-0.03, much less than the differences of void ratio recorded between tests. Others (e.g., Reid et al., 2021) have standardised end-of-test freezing, but methods from Rocchi and Coop (2014) were preferred here as these tell us the accuracy. The formulae used here are listed below:

\[
\begin{align*}
    v_i &= \frac{\gamma_w (1 + w_i) G_s}{\gamma_{bulk,i}} \quad (3.2) \\
    v_i &= \frac{\gamma_w G_s}{\gamma_{di}} \quad (3.3) \\
    v_i &= \frac{w_f G_s + 1}{(1 - \varepsilon_v)} \quad (3.4)
\end{align*}
\]

where \(\gamma_w\) is the unit weight of water, \(\gamma_{bulk,i}\) is the initial bulk unit weight, \(\gamma_{di}\) is the initial dry unit weight, \(w_i\) is the initial water content, \(w_f\) is the final water content, \(\varepsilon_v\) is the volumetric strain during the entire test.

When using the diameters of the samples in calculations, twice the width of the membrane was subtracted. The membrane width was measured for each test as it was found that the values were quite variable.

### 3.3.2 Membrane Corrections

The rubber membrane that surrounds the sample can affect the stresses, especially at the low stress levels reached in these tests. These must be corrected. The membrane rigidity may have a restraining effect when barrelling occurs and for low confining
pressures. This means that the radial stress that is truly applied to the sample is higher than the cell pressure measured with the transducer. Corrections were applied by adding to the radial stress, using the equation by Fukushima and Tatsuoka (1984):

\[
\Delta \sigma_r = -2 \left[ \frac{M \cdot \varepsilon_r}{D_i} \right]
\]  

(3.5)

where \( \Delta \sigma_r \) is the change in radial stress, \( M \) is the extension modulus of the rubber membrane per unit width, \( \varepsilon_r \) is the radial strain, \( D_i \) is the initial diameter of the sample.

The value of \( M \) equal to 0.36 kN/m has been experimentally measured by Carrera (2008) for identical membranes, and this value was used. Membrane corrections were made to monotonic test data. The membrane corrections made to the cyclic test data made no change to the results as the changes were so small, and where the maximum radial strain was -2.5% the correction was never significant.

With the membrane penetration effect, the membrane tends to penetrate the voids in between the grains at the interface if the sample is granular (ASTM, 2002) (Fig. 3.19). In an undrained cyclic test, the pore pressure increases and the water inside the sample moves to the lateral surface, refilling the voids. The \( \Delta u \) measured can therefore be underestimated. The membrane penetration effect affects the volume and not the stresses for a drained test, but for an undrained test the stresses are incorrect. Figure 3.20 is a photograph of one of the samples being tested under higher confining pressure. The penetration effect mainly depends on the particle size; according to Nicholson et al., (1993), this effect is important only if the grain dimension is higher than a threshold, which is considered equal to twice the membrane thickness. The tailings tested are mainly a fine-grained sandy soil, so are not significantly affected by the membrane penetration effects, as can be seen in the figure.
3.3.3 Compliance Test

An axial compliance test was carried out on a dummy sample. The sample was compressed to a load of 150 N and then unloaded to 0 N. The sample was then extended to a load of -80 N and then loaded to 0 N. Recordings of displacement (mm) and load (N) were taken every 10 seconds. Displacement against load was plotted on the same graph for both compression and extension and a trendline was added (Fig. 3.21). The trendline equation was then multiplied by the load data and deducted from displacement to make the external axial LVDT corrections. These corrections were made on all tests. They are only significant at small strain levels.

3.4 Oedometer Apparatus

3.4.1 Introduction

The oedometer test reproduces the conditions of one-dimensional compression in the laboratory. It is one-dimensional as only the vertical displacement is allowed to change, while the radial displacements are constrained by an oedometer ring, in which the sample is placed. Static loads are applied in stages and the strains and water flow are only allowed vertically. The compressibility of the soil is determined, as well as the consolidation and creep parameters, such as the compression index, $C_c$, and swelling index, $C_s$, if the sample is saturated. These tests were specifically carried out to identify whether non-convergent compression or “transitional” behaviour existed. Figure 3.22 shows a picture of the oedometer apparatus used in this study. Vertical loads were applied to the sample manually with a lever arm. The lever arm transmits the load to the sample and the ratio of the lever arm in this study was 10:1. The maximum load that could be applied to the oedometer apparatus used in this study was 160 kg.
3.4.2 Sample Preparation

Oedometer tests were carried out on the mineral sand tailings from Australia and the iron tailings from Sweden. Reconstituted samples were used since intact samples were impossible to obtain. Distilled water was gradually added into the dry tailings until the desired water content was reached. Lower water contents created a paste, and a slurry was created when higher water contents were used. The samples were stirred with a spatula to achieve a uniform consistency, after which, a vacuum was applied to the mixtures to remove trapped air. Prepared slurries and pastes were placed into the 50 mm diameter oedometer ring carefully with a small spatula to avoid trapping any air in the samples. The samples were then levelled off very carefully before the start of each test (Fig. 3.23). The surface of the sample was made as flat as possible so that there was good alignment with the top cap and no errors in the measurement of the sample height throughout the test. All tests performed were on saturated specimens.

3.4.3 Testing Procedure

All the screws on the lever arm of the apparatus were tightened before the start of each test. The lever arm was then balanced using the balancing weight. A dummy sample was used initially so the height of the sample could be calculated. Samples were created inside the oedometer apparatus so it was not possible to measure the initial height directly, and so a dummy sample was required. Two porous stones were used for all tests; one on top of the sample and one on the bottom. This was to allow the sample to drain when loads were applied. These were first de-aired using the vacuum pump. The larger porous stone was placed into the base of the cell. The metal ring that holds the oedometer ring in place was inserted, along with the oedometer ring. The dummy sample was then placed in the oedometer ring after the height had been measured using a caliper. The retaining ring was added on top and screwed down
evenly. The cell was then fixed to the top the oedometer apparatus and the top cap and yoke were attached. The yoke allows the transmission of the load from the lever arm carrying the weights to the top cap on top of the sample. A small load was added to the lever arm to keep the yoke in place. A dial gauge, used to measure vertical settlement, was adjusted so that it was on top of the ball of the yoke and a reading was taken. The dummy sample was removed and the sample preparation method above was used to add a tailings sample into the oedometer ring.

The mass of the tailings in the oedometer ring were recorded. Filter papers were positioned on the top and bottom of the sample, and the smaller de-aired porous stone was placed on top on the sample. The assembly of the metal ring, oedometer ring, retaining ring and top cap are all the same as above. The top cap centres the vertical load as it is applied to the sample. A water bath was fixed around the sample and the yoke was attached to the top platen along with a small weight on the lever arm to keep the yoke in place and to stop the sample from swelling. The dial gauge was moved round again so that it was on the top of the ball of the yoke and another dial gauge reading was recorded. The height of the sample could then be calculated by taking the dummy sample height and deducting the dial gauge reading with the dummy sample in and then adding the dial gauge reading with the sample in. The sample was then flooded by filling the water bath with distilled water. This water bath needed topping up throughout the test period, as the sample needed to be saturated throughout. At this stage, a new sample reading was taken so that the new sample height could be recorded. At time $t = 0$, the first load was added to the lever arm, and dial gauge readings were taken at predetermined intervals. After drainage was complete, the next vertical load was applied. The time between applying loads depended on the type of soil being tested. In order to determine the waiting time for each loading stage the
settlement readings were recorded at regular intervals. Figure 3.24 gives the settlement readings when 1.6 kg was added to a mineral sand tailings sample. It can be seen that the settlement stabilises after approximately one hour. The waiting time selected for each loading stage for the mineral sand tailings was approximately one and a half hours. When removing a weight to add a heavier one, the screw jack was wound up and the lever arm was held down so that there was no movement on the dial gauge. Weights were added up to a maximum of 160 kg and the resultant vertical stress was about 7 MPa.

After full consolidation was reached under the final load, the weights were removed in several stages to a low value close to zero, and the sample was allowed to swell. The dial gauge was moved to the side, and the water from the water bath was removed with a suction bottle. This preparation prior to the final unloading and the use of a small last load ensured that little water was absorbed from the porous stone by the sample while it was removed. Everything was dried with paper towels and the base was dismantled. The sample was then removed quickly and the sample height and water content were measured.

3.4.4 Void Ratio Calculations
Additional void ratio calculations to the ones mentioned in Section 3.3.1 could be used for the oedometer tests as these samples were fully saturated and final dimensions were taken. These additional formulae are listed below:

\[ v_i = \frac{y_w(1+w_f)G_S}{y_{bulk,f}(1-\varepsilon_v)} \]  
(3.6)

\[ v_i = w_i G_S + 1 \]  
(3.7)

\[ v_i = \frac{G_S - 1}{\left(\frac{y_{bulk,f}}{y_w} - 1\right)} \]  
(3.8)
\[ v_i = \frac{G_{S^{-1}}}{(y_{bulk,f} - 1)(1 - \varepsilon_v)} \]  \hspace{1cm} (3.9)

where \( y_{bulk,f} \) is the final bulk unit weight (Rocchi and Coop, 2014).

3.5 Morphologi G3 Particle Characterisation System

3.5.1 Introduction

The Morphologi G3 apparatus (Fig. 3.25) provides the ability to measure the size and shape of particles by image analysis and provides number-based statistics. A 5-megapixel digital camera takes individual greyscale images of each particle in 2D. There is a motorised objective revolver for automatic magnification changeover, and two light sources for reflected and transmitted illumination.

Samples can be dispersed in a number of ways: dry using the sample dispersion unit (SDU), dry by sprinkling, wet in a solvent which evaporates and wet in oil using a coverslip. The methods used are described in Section 3.5.2. After a sample was prepared, the sample was scanned and digital images were captured in real time providing visual verification of the data. The Morphologi software measured selected characteristics of each particle and provided reports that were exported for further analysis. Filters were set up to exclude contaminants (fibres) or touching particles. To exclude fibres, for example, that could have come from clothing, a filter was applied to exclude particles with an elongation higher than a certain value. Touching particles were excluded based on their convexity. A ‘scattergram tab’ sorts particles according to shape and a ‘comparison tab’ was available to identify key shape differences.

The particle size distributions of different tailings were obtained using two methods: sieving and Morphologi G3 particle analysis. The iron tailings from China
were washed through a 63 μm sieve to separate silt to sand sizes and then the separated materials were oven dried. A hydrometer test, following ASTM D 422 (ASTM, 2002), was attempted on the finer fraction (<63 μm) to obtain the grading curve, however a suspension could not be made when adding distilled water (Fig. 3.26). The tailings settled on the surface, so a deflocculant was used but this was still unsuccessful. The G3 Morphologi apparatus was therefore used to measure this section of the grading curve. This is discussed further in Section 3.6.4 below.

The Morphologi G3 apparatus was also used to determine the grading curve for the finer fraction of the mineral sand tailings from Australia because of the very small sample size of 0.5 g (Fig. 3.27). Multiple tests were carried out for separate samples, and repeated three times each, so it was concluded that using the Morphologi G3 apparatus would be quicker and more accurate than using the hydrometer.

3.5.2 Testing Procedure

A standard operating procedure (SOP) was created using the Morphologi software at the start of each test. This is a template that defines all of the measurement parameters and settings. Using an SOP ensures that measurements made on the same sample type are consistent. A sample name was required so that results could be easily identifiable. The number of slides or plates to be included in the analysis were selected under ‘measurement control’ and the ‘illumination’ page allows amendments to the light settings. The magnification was selected on the ‘optics selection’ pane. The ‘analysis settings’ page was displayed next and this is where filters could be applied to exclude certain parameters. Once the SOP was complete, the test could begin.

Four methods, as mentioned in Section 3.5.1, could be used to prepare the sample ready for image analysis. The methods used in this study were dry dispersion using
the SDU and wet dispersion with dry measurement. For the dry dispersion method, the dry sample was spooned into the inside of the SDU sample chamber. The SDU used the laboratory’s compressed air supply to disperse the dry sample onto a glass plate. The spoon size was selected depending on the grain size. The 5 mm$^3$ size was used because the iron tailings are fine-grained (<63 µm). The dry method was used here as the iron tailings settled on the surface when trying to make a suspension, as mentioned previously, so the wet method could not be used. The SDU sample chamber was then reinserted into the apparatus and the air pipe was pushed into the top of the chamber. The sample was then ready for dispersion.

The wet dispersion with dry measurement method was particularly useful for very fine particles, so this method was selected for the finer fraction of the mineral sand tailings from Australia. A suspension could be made with these particular tailings. Around 0.5 g of the sample was added to a vial along with 4 ml of a surfactant solution to prevent the agglomeration of particles. The sample was mixed so that it was in suspension and then a syringe was used to cover a clean glass slide with approximately 0.5 ml of the sample solution. The entire slide was covered and a clean slide was placed over the top to prevent pooling and contamination during evaporation. Once dry, the slide was ready to be measured. This process was repeated three times for each sample measured and an average was taken to complete the grading curve.

Once the sample was on the slide the test was started and the ‘microscope manager’ performed initial focusing and then the ‘focus on sample’ control dialogue appeared. This allowed the user to focus the microscope manually. Once this was set, the software scanned the plate and an estimated time for completion was given. Depending on the magnification and scanning area, tests took between one and 24 hours to complete.
3.5.3 Morphology

The Morphologi G3 particle characterisation system captures a 2D image of a 3D particle and calculates various size and shape parameters from this 2D image. The particle lies in its lowest energy position on the plate and so the largest area of the particle is presented to the camera. One of the principal diameters calculated, and the calculated diameter used in this study, was the circular equivalent (CE) diameter, which is the diameter of a circle with the same area as the 2D image of the particle. Because the particles are at rest, the minor diameter is omitted from all calculations, including aspect ratio and CE diameter. Greyscale images of a selection of individual mineral sand tailings particles taken using the Morphologi G3 particle analyser are given in Figure 3.28.

The Malvern Morphologi apparatus was also used to construct a cumulative frequency graph for aspect ratio, for the mineral sand tailings from Australia, which is given in Figure 3.29. Aspect ratio is used to describe shape and is defined as the particle width, measured along the minor axis, divided by particle length, measured along the major axis. The minor axis runs perpendicular to the major axis. The graph displays quite a range of aspect ratios arising from the nature of the material. These particular tailings underwent no crushing but there may have been some abrasion during the separation process, which involves washing the material through a sequence of spiral separators. Abrasion would also have occurred during the wind-blown transportation process from which the parent material originated.

The empirical chart proposed by Krumbein and Sloss (1963) was used to identify the sphericity and roundness of particles manually (Fig. 3.30). This method was used to identify a sphericity of 0.76 for the mineral sand tailings and a roundness \( R \) of
0.79. The sphericity ($S$) was also calculated by the Morphologi apparatus using Equation 3.10, which is equivalent to the circularity defined by Wadell (1933):

$$S = \frac{2\sqrt{\pi A}}{P_{\text{real}}},$$

(3.10)

where $P_{\text{real}}$ is the particle perimeter and $A$ is the particle area. Using this equation, the sphericity from the Malvern Morphologi apparatus was 0.73 for the mineral sand tailings. The meaning of these results are discussed in Chapter 5.

### 3.6 Materials

#### 3.6.1 Introduction

It took fifteen months to find suitable tailings for testing. This was because of the risks involved in handling tailings material as it can often contain hazardous chemicals. The original intended tailings to test were gold tailings from the Fazenda Brasileiro mine in Brazil and gold tailings from the Neves Corvo tailings dam in Portugal. Both tailings contained levels of arsenic that were too high to handle safely in the laboratory. The JECFA Provisional Tolerable Daily Intake (PTDI) for inorganic arsenic is 0.002 mg/kg bodyweight, equivalent to 0.12 mg/day for a 60 kg adult (European Food Safety Authority, 2010). The tailings from Brazil contained 1,587 ppm of arsenic and the tailings from Portugal contained 5,095 ppm of arsenic (Table 3.1). If these tailings were to be tested, it would not be safe to pour any contaminated water down the sink, as arsenic is water soluble. A disposal team would need to safely remove this. Any equipment that the tailings touched would have had to have been labelled separately and treated or removed after use and this would have been expensive. A separate ventilated lab with a fume cupboard would have needed to be set up, which is not currently available at UCL. The tailings could have been flushed a number of times.
first to remove some of the arsenic, but again, the contaminated water would have
eeded to have been removed. Dr. Judith Zhou, the department chemical safety officer,
and Rachel Fairfax, a safety advisor at UCL, concluded that these materials were
unsafe and should not be tested at UCL. It was decided that gold tailings should not
be shipped to UCL due to health and safety issues, and a more innocuous material
should be used instead.

3.6.2 Mineral Sand Tailings from Australia

A number of leads were contacted that could potentially send tailings material to UCL,
including contacts in Australia, China, India and Sweden. Professor Andy Fourie, of
Civil and Resource Engineering at the University of Western Australia, shipped
mineral sand tailings excavated from a mine about 100 km South of Perth, Australia.
This was chosen because the separation process for the heavy minerals (generally
rutile and zircon) is a gravimetric one, so obviating the need for excessive laboratory
safety measures, because no chemicals had been used in the process. Rutile is a rich
source of titanium dioxide and is usually found as part of mineral sand deposits. It is
also used for the production of titanium metal. To accentuate the transitional
behaviour, the medium sand part of the fraction was removed by sieving; such grading
separation often occurs when tailings are deposited hydraulically.

3.6.3 Iron Tailings from Sweden

Sara Töyrä from Luossavaara-Kiirunavaara AB (LKAB), who works in dam safety,
and Roger Knutsson, from Tailing Consultants Scandinavia, Sweden, sent over iron
tailings. The mineralogy of the tailings is safe, including the arsenic content which
does not exceed 2.3 ppm (Table 3.2). The upstream method in combination with
spigotting was used and so the tailings material is segregated close to the dam. Around
the year 2000 the construction of the embankments changed to the upstream method,
so there are now layers of fine tailings and slime included in the embankment’s structural zone. There was a drilling unit at site that could have been used for undisturbed sampling, but after discussion it was decided that this would be inappropriate as shipping the material would disturb and densify the samples. Disturbed samples were sent over, and the grading curve is given in Figure 3.31.

3.6.4 Iron Tailings from China

The final tailings included in this study is an iron tailings from the Wanniangou tailings dam in Panzihua City, China. This tailings had already been tested in the laboratory at UCL and so the risk assessments had already been completed confirming that it was safe to handle in the laboratory. Latex gloves were worn when handling tailings material to avoid irritation, and fume masks were worn during sieving, to avoid the inhalation of any small particles.

Preliminary work was carried out on the iron tailings from the Wanniangou tailings dam. The iron tailings received were not uniformly graded. The particle size distribution was obtained by wet sieving, to minimise the creation of dust (Fig. 3.32). A range of sieves were used to achieve precise grading: 63 µm, 150 µm, 212 µm, 300 µm, 425 µm, 600 µm and 1 mm. After sieving, all soil samples were oven-dried separately to obtain their individual dry mass, allowing the percentage of each size to be calculated. Figure 3.33 shows the gradings curves. It can be seen that both the coarse and fine tailings are quite well graded and have exactly the same fines content. The materials are quite similar. Once the grading curves had been produced, it was decided that all material would be passed through the 63 µm sieve to make a better distinction between fine-grained and coarse-grained material, for an additional two triaxial tests. An analysis of this data is given in Chapter 4.
### 3.7 Material Characterisation Procedures

#### 3.7.1 Specific Gravity, $G_s$

The specific gravity, $G_s$, of a material is defined as the ratio of the weight of a volume of the material to the weight of an equal volume of distilled water. The specific gravities of tailings examined in this study were measured following the procedures recommended in the ASTM D 854 (ASTM, 2002). The method outlined is described below:

1. Determine the weight of the empty, clean and dry pycnometer, $W_p$.
2. Pass 15 g of a dry soil sample into the pycnometer and record the weight, $W_{ps}$.
3. Add distilled water to ¾ full and de-air.
4. Fill the pycnometer with distilled water, dry the exterior surface, then weigh, $W_B$.
5. Empty the pycnometer and clean. Fill with distilled water, dry the exterior surface, then weigh, $W_A$.

A set of three parallel tests were conducted to obtain three specific gravity values for each tailings. Tailings are crushed rock so there is no reason to believe that coarse and fine tailings should have different $G_s$ values. The three values were calculated using the equation below:

$$G_s = \frac{W_0}{(W_A-W_p)-(W_B-W_{ps})}$$  \hspace{1cm} (3.11)

where $W_0$ is the weight of the sample of oven-dry soil.

#### 3.7.2 Index Tests – Atterberg Limits

Index tests were conducted to measure the liquid limits and plastic limits to better understand the basic influence of water on tailings behaviour, based on the guidelines from Head (2004). The term plasticity is the ability of a soil to undergo irrecoverable
deformation without cracking or crumbling. Plasticity is due to the presence of a significant content of fines in the soil. The upper and lower limits of the range of water content over which the soil exhibits plastic behaviour are defined as the liquid limit, \( w_L \), and the plastic limit, \( w_P \). Above the liquid limit, the soil flows like a liquid, below the plastic limit, the soil is brittle and crumbles.

The analysis of the liquid and plastic limits of the finer part of the mineral sand tailings from Australia were carried out. For the liquid limit, approximately 300 g of dried tailings were mixed with distilled water to form a homogenous paste. This was then placed into a metal cup with a spatula and levelled so that the surface was smooth. A cone penetrometer was positioned above the sample, and the cone was lowered so that it just touched the surface of the sample. The cone was then locked at this level and the displacement reading was zeroed. The lock was then released for five seconds, allowing the cone to penetrate the soil sample. This penetration depth was measured. The sample surface was levelled and the test was repeated three times for accuracy. The entire process was repeated at least four times using the same soil sample, but increasing the water content each time. The penetration values should cover the range of approximately 15-25 mm. Cone penetration was plotted against water content, and the best straight line fitting the plotted points was used. The liquid limit is defined as the percentage water content corresponding to a cone penetration of 20 mm. The liquid limit for the fines of the mineral sand tailings from Australia was 59.6%. Loose samples made in the consolidometer were made at a water content determined by multiplying the liquid limit by 1.5 (89.3%).

For the plastic limit, the soil was mixed with distilled water until it became sufficiently plastic to be moulded into a ball. Approximately 2.5 g of tailings fines were formed into a thread. The thread was rolled out onto a glass plate using the
fingertips until the diameter was approximately 3 mm. The thread was then remoulded and the procedure was repeated until the thread of soil sheared both longitudinally and transversely when it had been rolled to a diameter of 3 mm. The procedure was repeated using three more parts of the sample, and the water content of the crumbled soil was determined as a whole. This water content is defined as the plastic limit as it is the lowest water content at which the sample is plastic. The entire test was repeated using four other sub-samples, and the average was taken. The plastic limit for the finer part of the mineral sand tailings was 30.1%. Index tests could not be completed for iron tailings from China and Sweden as the materials had very low plasticity.

3.7.3 X-ray Diffraction Analysis
The mineralogy of the Australian tailings was investigated through X-ray diffraction analysis (XRD). Professor Ian Wood from UCL Earth Sciences assisted with the tests and analysis. The sample was ground under ethanol using an agate pestle and mortar and then X-rayed in two different ways; firstly, a mount was made by side-filling the dry material into a holder against a slightly roughened surface. The powder diffraction pattern was then measured with radiation with a wavelength of about 1.79 Angstroms (Å). Steps of 0.017 degrees were scanned from two degrees to 120 degrees with the X-ray tube running at 40 kilovolts (kV) and 20 milliamperes (mA). The resulting data are given in Figure 3.34. The vertical scale on this plot is the square-root of the number of counts to reveal the weaker parts in the diffraction pattern. The quartz peak at about 31 degrees is so strong that if a linear scale is used the other peaks would be too small to see easily. The sample clearly contains large amounts of quartz due to the sharp peaks with red reflection markers. The broader blue peaks with blue reflection markers indicate the presence of kaolin. There are many weak peaks in the sample that are difficult to identify. They could be due to a mixture of several phases, such as rutile.
and anatase (both TiO2) and perhaps some iron oxides. As previously mentioned in Section 3.6.2, it is likely that rutile is present in these mineral sand deposits. It is also likely that the minerals present are from an alkali feldspar which is indicated from the presence of the green reflection markers.

Secondly, in order to have a better look at the silicate clay minerals, an orientated sample was made by shaking a small quantity of the material with deionised water and spreading the slurry onto a glass plate which was then air-dried. The resulting data are given in Figure 3.35. The graph shows that if there are any layer silicates other than kaolin present, they are only in extremely small amounts. Mica minerals would give a peak at ~10 degrees and chlorites, smectites and vermiculites etc., would give a peak in the range of 4-10 degrees, and there is no sign of this. It looks like kaolin is the only layer silicate present.

The search-match program that was used to identify the minerals present was called ‘HighScore+’. On the basis that the sample contains quartz, kaolin and an alkali feldspar, the suggested relative proportions of the minerals in terms of weight percent are: quartz 60%, kaolin 22% and feldspar 18%.

3.7.4 Scanning Electron Microscope

A scanning electron microscope (SEM) was used on the mineral sand tailings from Australia to look at the orientation of particles in a loose sample and a dense sample. An SEM uses electrons to form largely magnified images. Laboratory technician Jim Davy from UCL Earth Sciences assisted with the tests.

The samples were one-dimensionally consolidated in an oedometer up to a value of 6.4 kg and then extruded from the oedometer ring carefully. 6.4 kg was selected as it equates to 319 kPa vertical stress which is similar to the value used in the triaxial
apparatus. Approximately 1 cm³ was cut out of each sample and these cubes were left to air dry for 24-hours. These were then snapped in half to expose a fresh surface. It was important to identify the top and bottom parts of the samples throughout the testing process to be able to analyse the orientation of the particles correctly after the images were produced. The snapped samples were superglued to metal mounts carefully (Fig. 3.36). Once the glue was dry, silver was painted onto the dry glue and onto the edges of the sample that touched the glue. The samples were then put into an ‘agar sputter coater’ to coat the samples in gold (Fig. 3.37). These were to give the samples conductive properties. Without the coating, there can be a build-up of electrons on the sample which can change the SEM images which can give a false impression of how the sample looks. Once coated in gold, the samples are added to the base that will go inside the SEM chamber (Fig. 3.38). A photograph of the SEM used in this study is presented in Figure 3.39. Once the samples are in the chamber the images are produced instantaneously. Different magnifications can be selected in the SEM software. The magnifications chosen for these samples was x300, x1500, x5000 and x10000. The software was also used to adjust the focus and brightness to generate clear images (Fig. 3.40). The SEM images are presented and analysed in Chapter 5.
Table 3.1 - The principal minerals found in the Neves Corvo tailings (Mafalda Oliveira, personal communication, 2018).

<table>
<thead>
<tr>
<th>ELEMENTS</th>
<th>CONCENTRATION</th>
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<tbody>
<tr>
<td>Copper</td>
<td>0.66 %</td>
</tr>
<tr>
<td>Lead</td>
<td>0.25 %</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.74 %</td>
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<tr>
<td>Tin</td>
<td>0.16 %</td>
</tr>
<tr>
<td>Sulphur</td>
<td>23.5 %</td>
</tr>
<tr>
<td>Iron</td>
<td>27.6 %</td>
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<tr>
<td>Arsenic</td>
<td>5095 ppm</td>
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<tr>
<td>Antimonium</td>
<td>590 ppm</td>
</tr>
<tr>
<td>Bismuth</td>
<td>67 ppm</td>
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<td>Parameter</td>
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<tr>
<td>V</td>
<td>ppm</td>
</tr>
<tr>
<td>Zn</td>
<td>ppm</td>
</tr>
</tbody>
</table>

Table 3.2 - The principal minerals found in the iron tailings sent from Sweden (Sara Töyrä, personal communication, 2019).
Figure 3.1 - Bishop and Wesley triaxial apparatus used in this study.

Figure 3.2 - Triaxial apparatus used at UCL, including an iron tailings sample for testing.
Figure 3.3 - Schematic representation of Bishop and Wesley triaxial apparatus (modified by the author from Todisco, 2016).
Figure 3.4 - The TRIAX software running a cyclic test using sine wave loading.

Figure 3.5 - Axial LVDTs mounted on a mineral sand tailings sample.
Figure 3.6 - A consolidated iron tailings slurry sample from China being extruded from the consolidometer.

Figure 3.7 - Schematic representation of a rubber suction cap connection (Todisco, 2016).
Figure 3.8 - Before the split mould was altered it did not sit properly on the base.

Figure 3.9 - After the split mould was altered and attached, the membrane was folded over and grease was applied to the splits down the mould.
Figure 3.10 - The submerged iron tailings from China being spooned into the membrane filled with distilled water.

Figure 3.11 - A half sample of the iron tailings from China from the consolidometer being trimmed to size.
Figure 3.12 - The trimmed half sample of the iron tailings from China with a diameter between 36-37 mm.

Figure 3.13 - Two O-rings were added and the greased suction cap was fitted on the top cap.
Figure 3.14 - A layered sample of the iron tailings from China. The upper half was made using the consolidometer and is fine-grained and the bottom half is coarse-grained and has been prepared using the wet pluviation method.

Figure 3.15 - The consolidometer constructed for 36 mm diameter samples.
Figure 3.16 - The membrane discs with radial cuts for the lubricated end platens.

Figure 3.17 - The adapted polished steel end platen with porous stone in the centre. The two outer drainage holes have been filled with superglue.
Figure 3.18 - The membrane disc with one hole in the middle to allow for drainage through the porous stone which has been covered with filter paper.

Figure 3.19 - Membrane penetration effect in granular soils (Head, 2004).
Figure 3.20 - Membrane penetration under high confining pressure on an iron tailings sample from China.

Figure 3.21 - A compliance test graph displaying displacement against load for both compression and extension on a dummy sample.
Figure 3.22 - The oedometer apparatus used in this study.

Figure 3.23 - A levelled off sample of the mineral sand tailings ready for an oedometer test.
Figure 3.24 - Sample settlements with time for an oedometer test stage on the mineral sand tailings from Australia.

Figure 3.25 - The Morphologi G3 apparatus and computer software.
Figure 3.26 - The iron tailings from China settling on the surface of a hydrometer test.

Figure 3.27 - Grading curve of finer fraction of mineral sand tailings from Perth, Australia.
Figure 3.28 - Morphologi greyscale images of individual particles from the mineral sand tailings from Australia. CE = circular equivalent diameter, and AR = aspect ratio.

Figure 3.29 - Cumulative frequency graph for aspect ratio for the mineral sand tailings from Australia.
Figure 3.30 - Particle shape characterisation chart (Krumbein and Sloss, 1963).

Figure 3.31 - Grading curve for the iron tailings from Sweden.
Figure 3.32 - Wet sieving the iron tailings from the Wanniangou tailing dam.

Figure 3.33 - Grading curves of coarse-grained and fine-grained iron tailings from the Wanniangou iron tailings dam.
Figure 3.34 - X-ray diffraction analysis of the mineral sand tailings from Australia indicating a strong presence of quartz.

Figure 3.35 - X-ray diffraction analysis of silicate clay minerals in the mineral sand tailings from Australia.
Figure 3.36 - The loose (left) and dense (right) samples mounted for SEM testing.

Figure 3.37 - The agar sputter coater coating one of the samples with gold.
Figure 3.38 - The gold coated samples on the base ready for the SEM.

Figure 3.39 - The scanning electron microscope at UCL.
Figure 3.40 - The SEM software use to select magnification and adjust the focus and brightness of the images.
4. **Reanalysis of Previous Research**

4.1 **Introduction**

Iron tailings from the Wanniangou tailings dam in Panzihua City, China were used in this part of the study. Figure 4.1 shows an aerial view of the dam and surrounding area. It is one of the largest tailings dams in Asia and was constructed using the upstream method (Li and Coop, 2019). The dam is located near residential areas and transportation infrastructure so it was important that the material was tested to analyse the liquefaction potential. The dam is also 210 m high which is significantly high for a tailings dam, and so stability is a major concern (Li and Coop, 2019). The material was excavated from three different locations: upper beach (UB), middle beach (MB), and pond (PO) by Li (2017) (Fig. 4.2). This was to ensure a complete investigation of the behaviour of the tailings, as the particle size differed at each location. The particle size distributions are shown in Figure 4.3. Due to the sorting of the depositional process, moving away from the outfall at the dam, the sandy material formed the upper beach and the particle size changed gradually to a silty material in the pond (Li, 2017).

The specific gravity, \( G_s \), values were measured by Li as being equal to 3.37, 3.14 and 3.11 for UB, MB and PO respectively (Table 4.1). These values are relatively high because of the presence of metal minerals. Li investigated the mineralogy of the tailings through X-ray diffraction analysis (XRD) (Table 4.2). After extracting the valuable metal mineral, the remaining part mainly consisted of pyroxene (diopside), feldspar (labradorite), hornblende, chlorite, and a small quantity of metal minerals like ilmenite and iron oxide. The minerals identified for the three localities are quite similar, but with slightly changing quantities. The UB tailings, which is the coarsest, was found to have more metal minerals, and the PO tailings, the finest one, contains more chlorite, which belongs to clay-sized minerals.
Li used a scanning electron microscopy (SEM) to analyse the morphology of the tailings. It can be seen in Figure 4.4 that the UB and MB materials are similar, apart from the particle size. They both consist of bulky, angular and sub-angular particles. For the PO material, the bulky particles are also angular to sub-angular, but many more small particles exist, which are flatter and more elongated.

The initial specific volumes for all tests before isotropic consolidation for the cyclic tests are given in Table 4.3. These will be discussed later on in the chapter.

4.2 Reanalysis of the Work by Li (2017)

The compression paths for isotropic compression for the upper beach (UB), middle beach (MB) and pond (PO) material are displayed in Figure 4.5(a)-4.5(c). The one-dimensional normal compression lines (1D-NCL) came from the oedometer tests carried out by Li (2017). In plotting the current mean effective stress, $p'$, Li had to estimate the coefficient of earth pressure at rest, $K_0$, from Equation 4.1 (Jaky, 1948):

$$K_0 = 1 - \sin \phi'_{cs} \tag{4.1}$$

where $\phi'_{cs}$ is the critical state angle of shearing resistance.

The isotropic normal compression lines (IC NCL) are estimates by Li (2017), as his tests did not reach these high stress levels. Two CSLs have been added to each graph; one using digitizer software, where the CSL was copied from Li (2017), and one using the Gudehus equation (Gudehus, 1996). Li did a series of monotonic triaxial tests that gave him the CSL. This CSL was taken directly from the data and was therefore more accurate than the CSL from the Gudehus equation, which does not necessarily have to fit well. The Gudehus equation is given below:
\[ e_i = e_{i0} \exp \left[- \left( \frac{3p'}{h_s} \right)^n \right] \]  

where \( e_i \) is the current void ratio, \( e_{i0} \) is the maximum void ratio, \( p' \) is the current mean effective stress, \( h_s \) is the granulate hardness, and \( n \) is the constant exponent.

The compression paths all start at states below the CSL. The UB samples are not compressing much because the samples are dense and coarse-grained and the stress levels are low (Fig. 4.5(a)). The compression paths for the MB samples are slightly inclined showing little compression (Fig. 4.5(b)). The looser sample (MB1) compresses more than the denser sample (MB2) as expected. The UB and MB materials are sandy and silty and therefore do not compress very much at low stress levels. The PO graph displays very different behaviour when compared with the UB and MB samples. The PO samples are much looser and more compressible, and there was evidence of a yield where the volume change starts to accelerate (Fig 4.5(c)). The PO material was more clay-like and more plastic than the UB and MB materials. The CSL from Li for the PO material is a straight line, even at low \( p' \) values. The CSL from the Gudehus equation has not been added as fitting a curved CSL is unnecessary. The UB and MB materials are more sandy and less plastic than the PO material. The UB and MB CSLs are therefore curved. This curvature generally indicates the onset of particle breakage (Coop, 1990).

Li (2017) carried out undrained cyclic loading tests on UB, MB and PO samples. The number of cycles to failure for each test are given in Figure 4.6(a) – 4.6(c). Two of the cyclic tests for the UB material (UB3 and UB4) have cyclic amplitudes of 120 kPa. These tests also have \( p_i' \) of 200 kPa (Fig. 4.6(a)). All other tests have amplitudes of 60 kPa and \( p_i' \) of 100 kPa. The number of cycles are given next to the data points.
in each graph. As to be expected, the denser samples took more cycles to reach cyclic mobility than the looser samples.

The axial strains (0.125% and 0.5%) during undrained cyclic loading for UB, MB and PO are given in Figures 4.7(a) – 4.7(c). For the UB material (Fig. 4.7(a)), similar axial strains are reached at similar $p'$ values when comparing the two 120 kPa amplitude tests. For example, the $p'$ value for 0.125% axial strain is similar for UB3 and UB4. This is also true for the two 60 kPa amplitude tests. For example, the $p'$ value for 0.5% axial strain is similar for UB1 and UB2. Trendlines have been added and the strain percentages are slightly inclined. The denser samples reach a slightly lower $p'$ value at each axial strain percentage. This is to be expected as at a given $p'$ value the strains are larger for looser samples as the looser samples are not as stiff as the denser ones, but it is perhaps surprising that density does not have a greater effect. Yield points for where the stiffnesses rapidly decline, along with contours, have also been added to the plot. The definition of ‘yield point’ is discussed in more detail below when Figure 4.11 is introduced. These contours align with the axial strain contours. MB1 and MB2 have similar void ratios, and like the UB material, similar axial strains are reached at similar $p'$ values (Fig. 4.7(b)). Sample PO2 was denser than PO1, and again it can be seen that similar axial strains are reached at similar $p'$ values (Fig. 4.7(c)). This is interesting as the PO material was more plastic than the UB and MB materials and yet the pattern was still the same. The main thing that is different is the higher specific volumes that are indicative of how the grading is changing. It is difficult to evaluate the axial strains because of the limited range of $p'$ and $v$ seen in all three of the plots, but within this limited range the axial strains are almost the same. The differences of $v$ are only relatively small and do not have a large effect.
The combined \( v:lnp' \) graph displaying the number of cycles to failure and estimated contours for UB, MB and PO samples are shown in Figure 4.8. Contours have been added for two cycles to failure and ten cycles to failure for the 60 kPa amplitude tests (black contours) and the 120 kPa amplitude tests (red contours). All contours indicate that at a given \( p' \) value the number of cycles to failure are greater for the denser samples. This might be expected since the number of cycles to failure from the initial state would be expected to be greater for a denser sample, so the \( p' \) at a given number of cycles to failure will always be greater than for a loose sample as the test proceeds. However, it is interesting that unique contours can be drawn and that these are similar for all three materials. Perhaps these contours work well because the three CSLs are not so different. This similarity means that the mechanical behaviour of the three materials might be alike. The 120 kPa amplitude tests reach ten and two cycles to failure at higher \( p' \) values than the 60 kPa amplitude tests, and so the 120 kPa amplitude tests are reaching failure more quickly at a higher \( p' \) value as expected.

The combined normalised \( vn:lnp' \) graph is shown in Figure 4.9 and displays the number of cycles to failure with contours for UB, MB and PO samples. The stress paths of the three tailings have been normalised using the normalised specific volume equation below:

\[
v_n = \frac{(v_i - \Gamma)}{\lambda}\]  

(4.3)

where \( v_i \) is the current specific volume and \( \Gamma \) and \( \lambda \) are the projected intercept and the gradient in the \( v:lnp' \) plane of the CSLs at 1 kPa.

The normalisation cannot work well for the curved part of the CSL for UB and MB, so only the straighter parts have been normalised. Normalisation brings together the CSLs, but it does not make too much difference in the graph as the CSLs were
already quite similar before normalisation, as shown in Figure 4.8. It can be seen that the amplitude makes surprisingly little difference on the number of cycles to failure. For example, doubling the amplitude does not have double the effect. The normalised graph also shows that the initial states for the PO samples cannot exist for the UB and MB materials. UB and MB samples could not have the $\nu n$ values of the PO samples, and so the contours on the normalisation may be to some extent artificial.

The combined normalised $\nu n: \ln p'$ graph displaying axial strain % for all samples is shown in Figure 4.10. Contours have been added for 0.125% and 0.5% axial strain for the 60 kPa amplitude tests (black contours) and 120 kPa amplitude tests (red contours). Contours have also been added for the yield points. When normalised, there are significant differences in $\nu n$ than in $\nu$ originally. There is a small range in the $\nu n$ values and the contours are different for different $p'_i$ values. The contours for the 60 kPa amplitude tests are close to being vertical, and so the location (UB, MB, PO) is not important. The factors that are most important are the $p'$ and the amplitude because the specific volume only has a small effect. The state parameter, $\Psi$, (see Chapter 2) is also less important and hence grading which affects the initial specific volume which seems to have little effect, and this was unexpected. The contours for the 120 kPa amplitude tests are more inclined meaning that both $p'$ and $\nu$ affected the axial strain. The contrast of the strongly inclined contours on Figure 4.9 and more vertical ones for the 60 kPa amplitude tests on Figure 4.10 indicates that at a given number of cycles to failure, denser samples have accumulated more strain than looser ones, so current strain will not be a good indicator of proximity to failure.

The decay of stiffness during undrained cyclic loading for all samples are given in Figure 4.11. The stiffness is calculated as a secant between the peak and trough of each cycle. The thicker lines represent the 120 kPa amplitude tests (UB3 and UB4),
whilst the thinner lines represent the 60 kPa amplitude tests. The variation in specific volume is indicated by the dashed type of the lines. The stiffnesses should be expected to depend on the density of the samples, but these results show that there is a weak trend with void ratio. Also, it does not look as though the different localities (upper beach, middle beach, pond) show any real significance in terms of the decay of stiffness. All of the plots display a point where the stiffnesses rapidly decline. These points are a good way to define cyclic mobility and are referred to here as ‘yield points’. The stress paths have been plotted for Li’s second test on the pond tailings (PO2) (Fig. 4.12) and for the first layered test (L1) (Fig. 4.13). The yield points have been plotted in orange. In both graphs, the yield point does not correspond to anything noticeable on the stress path and there is no change in shape. However, the stiffnesses plummet after reaching these yield points. The normalised version of Figure 4.11 is given in Figure 4.14. Normalisation for the critical stress level $p_i'$ brings the stiffness plots closer together and the stiffnesses on the ‘knees’ have become quite consistent. Again, there is a weak trend with void ratio, and normalisation has not improved this.

Figure 4.15 is a combined normalised $vn$:$lnp'$ graph displaying stiffnesses, yield points and contours for UB, MB and PO samples. There is a vertical contour going through the yield points for the 60 kPa amplitude tests (black contour). This was added using the six yield points from the test data. There are only two yield points for the 120 kPa amplitude tests and so it has been assumed that the contour going through these points (red contour) is also vertical. Contours have been added through the 20 MPa and 50 MPa stiffness points for the 60 kPa amplitude tests (black dashed contours). They have also been added through the 20 MPa, 50 MPa, 100 MPa and 150 MPa stiffness points for the 120 kPa amplitude tests (red dashed contours). All of these contours are parallel to one another and are close to being vertical. As already
mentioned, vertical contours indicate that the controlling parameters are $p'$ and the amplitude and volume are unimportant. When $\sim$20 MPa stiffness is reached, cyclic mobility is triggered for the 60 kPa amplitude tests (black contour), and when $\sim$50 MPa stiffness is reached, cyclic mobility is triggered for the 120 kPa amplitude tests (red contour). The higher amplitude tests reach the yield points at a higher $p'$ than the lower amplitude tests.

4.3 Analysis with New State Parameters

The state parameter was introduced in the literature review in Chapter 2. For reference, the state parameter is defined as:

$$\Psi = e - e_{CS}$$  \hspace{1cm} (4.4)

where $e$ is the initial void ratio of the soil after consolidation and $e_{CS}$ is the critical void ratio for the same mean stress. A negative $\Psi$ means that a soil will have a dilative behaviour, while a positive $\Psi$ is related to a loose specimen susceptible to liquefaction. Soils that exhibit the same $\Psi$ are equidistant from the CSL, and consequently they should exhibit similar behaviours (e.g., Been and Jefferies, 1985).

Figure 4.16 is a schematic diagram displaying a curved CSL in the $v$:$\ln p'$ plane. Curved CSLs were introduced in the literature review in Figure 2.27. Points A, B and C all have the same positive $\Psi$ in Figure 4.16. From Carrera et al., (2011), for monotonic undrained shearing, points A and B would not have the same behaviour even if they have the same $\Psi$. Point A will clearly liquefy as $p'$ tends to zero while point B will reach a stable critical state. Point C is also above the curved part of the CSL, but the $\Psi$ is high enough so that the starting point is above the asymptote and so
it will still liquefy. This is a function of having a curved CSL. It is therefore not expected that the state parameter will work for static liquefaction with a curved CSL.

Figure 4.17 is a schematic diagram displaying points above and below a curved CSL. After plotting a curved CSL it would be interesting also to see how point C would behave for undrained cyclic loading, having a negative $\Psi$, and how this would compare with points A and B. All points will move horizontally when undrained cyclic loading is applied, but they will not stop at the CSL. This issue has not been addressed in the literature and the papers discussed mostly had soils that displayed straight CSLs.

Carrera (2008) examined the static liquefaction of tailings taken from the Stava, Italy collapse in 1985. Tests were conducted by Carrera (2008) on the fluorite tailings from Stava, with different silt and sand fractions, to investigate the effect of grading on the susceptibility to liquefaction. The Stava case study was described in Chapter 2. Figure 4.18 gives the grain size distributions of the Stava tailings Carrera used; a fine sand and a silt. The specific gravity, $G_s$, of the sand is 2.721 and that of the silt is 2.828 (Carrera et al., 2011). Carrera carried out a number of monotonic triaxial tests on the sand and silt mixed at different percentages. The critical state data for all the tests are presented in Figure 4.19. The sand (green) has the highest void ratio, followed by the silt (blue), then the 70% sand and 30% fines mix (red) and finally the 50% sand and 50% fines mix (pink), so as fines are added the CSL moves downwards to lower void ratios and then back up, as indicated by the arrows on the graph. This graph also displays the initial points before shearing for each test that has been reanalysed in this chapter. These were plotted by the Author so that the $\Psi$ values could be calculated. Not all tests were selected as not all of the information was available. The modified state parameter, $\Psi_m$, brittleness index, $I_B$, and state pressure index, $I_P$, were also calculated for this data set. The data for these parameters are presented in Table 4.4.
Bobei et al., (2009) proposed a modified state parameter, $\Psi_m$, for characterising the static liquefaction of sands with fines. The $\Psi_m$ is defined as:

$$\Psi_m = \Psi |\frac{\Delta p'}{p'}| e$$  \hspace{1cm} (4.5)$$

where $\Delta p'$ is the difference in mean effective stress between the state at the start of undrained shearing and that on the CSL. A factor $|\frac{\Delta p'}{p'}|$ is applied that gives a higher value of $\Psi_m$ with the increase in $p'$ when the value of $\Psi$ remains constant. This was intended to improve the prediction of liquefaction tendency because the $\Delta p'$ is increasing by an equal amount to $p'$.

Bobei et al., (2009) noticed that for their tests the $\Psi$ values only reduced slightly for higher $p'$ values and concluded that this did not give an adequate explanation for the change in behaviour, and so $\Psi_m$ was proposed. Figure 4.20 gives a schematic diagram of points above and below the CSL. Point B shows a negative $\Psi$ value and a negative $\Delta p'$. The modulus is therefore required for $\frac{\Delta p'}{p'}$ as otherwise there would be a positive $\Psi_m$ whether the state were above or below the CSL and this cannot be correct.

Point A is above the asymptote. For points above the asymptote, $|\frac{\Delta p'}{p'}|$ would equal one as $p'$ at the critical state is at zero. This would mean that the $\Psi_m$ equation for points above the asymptote would be $\Psi \times e$. Carrera’s (2008) data have been analysed in terms of $\Psi_m$.

Bobei et al., (2009) noted that the factor $|\frac{\Delta p'}{p'}|$ in the definition of $\Psi_m$ can also be related to the state pressure index, $I_p$, which is defined as the ratio of the current mean pressure to the mean pressure at the critical state that corresponds to the current void ratio (Wang et al., 2002):
\[ I_p = \frac{p'}{p'_{cs}} \]  \hspace{1cm} (4.6)

The \( I_p \) is an alternative way to account for the dependency of soil behaviour on the CSL, and the dilatancy and strain softening of a soil are identified using this parameter. Soils on the NCL have a fixed value of \( \frac{p'}{p'_{cs}} \) typically between 2-4, as long as it is parallel to the CSL (Fig. 4.21 – points B, C and D). All points above the CSL asymptote have an \( I_p \) of infinity (point A).

Bishop (1967) introduced the brittleness index, \( I_B \):

\[ I_B = \frac{(q_{max} - q_{cs})}{q_{max}} \]  \hspace{1cm} (4.7)

Figure 4.22(a) is a schematic diagram of a \( e:\ln p' \) graph displaying three points. The same points have been plotted on the \( q' \) against \( p' \) graph in Figure 4.22(b). An undrained test at point A is above the asymptote and strain softens all the way to \( p' = 0 \). Another test at point B is to the right of the CSL and strain softens consistently as it is on the curved part of the CSL and so \( p' \) reduces significantly. A test at point C strain softens less as it is on the straight part of the CSL so \( p' \) reduces less. A final test at point D is to the left of the CSL and displays strain hardening because \( p' \) is increasing. The reduction or increase of \( p' \) is therefore directly related to the reduction in \( q' \) and the existence of strain softening, and so \( I_p \) is not particularly useful. The \( I_B \) is also not particularly useful if the CSL is curved as it can already be determined what will liquefy, i.e., all states above the horizontal asymptote.

Carrera’s (2008) tests were reanalysed in terms of brittleness index, \( I_B \), state pressure index, \( I_p \), state parameter, \( \Psi \), and modified state parameter, \( \Psi_m \). The data were plotted from 24 of Carrera’s (2008) tests. Figure 4.23 displays the \( I_B \) against \( I_p \) values. The circular symbols indicate the tests that did not liquefy, and the triangular
symbols indicate the tests that did liquefy. Liquefied tests had a $I_B = 1$. A $I_B$ of zero indicated no strain softening at higher pressures, and those tests with a $I_B \sim 0.5$ were near the curved part of the CSL in the middle. The tests with initial states located under the CSL have a $I_p \leq 1$ and are not very brittle at all. The tests that lie above the CSL have a $I_p > 1$, and as the $I_p$ increases, so does the $I_B$. There are not enough data points to see a clear trend for fines tests (yellow) and the 50% sand, 50% fines tests (red), so, it is not possible to see clear differences between the different mixes. Nevertheless, the relationship between $I_p$ and $I_B$ is clear but self-evident from Figure 4.23 and so is of little use in predicting static liquefaction since the triaxial test needed to give $I_p$ would also identify the CSL and which controls whether samples liquefy.

Figure 4.24 shows the $I_B$ values against $\Psi'$. There is no clear trend visible. The graph shows that samples can have the same $\Psi'$ but display very different behaviour. For example, for a $\Psi' \sim 0.05$, there is a clean sand sample that has a $I_B \sim 0.4$, but also a 50% sand and 50% fines sample that has liquefied. There are also three clean sand samples with very similar $\Psi'$ values (0.07-0.08) that display a wide range of $I_B$ from 0.05 to 0.95. There is a significant difference between the soils, and inconsistencies even for one soil, and so this graph does not work very well, as expected from Figure 4.16.

According to Bobei et al., (2009), samples with higher $\Psi_m$ values should be more brittle, and lower $\Psi_m$ values should give less brittle behaviour. The $\Psi_m$ values have been plotted in Figure 4.25. A trendline has been put through the data points for the clean sand tests (blue). Trendlines and estimated trendlines have been added for the 70% sand and 30% fines tests (green), 50% sand and 50% fines tests (red) and the fines tests (yellow) as there are limited data points for each set of tests. There is much
better agreement in this graph than in the $\Psi$ graph. The trendlines do show more brittle behaviour with an increasing $\Psi_m$, while the scatter is much reduced for each individual soil, but there is no agreement between the different soils. There is therefore little use in using the $\Psi_m$ as a prediction tool when all the plots are different for different soils. 

The CSL is also required in order to calculate the $\Psi_m$ and so it is a cumbersome method.

Li’s (2017) tests were reanalysed in terms of state pressure index, $I_p$, state parameter, $\Psi$, and modified state parameter, $\Psi_m$. The data were plotted from eight of Li’s (2017) tests, but unlike Carrera’s (2008) tests above, these were on cyclic loading. All tests were on the iron ore tailings from Panzihua and had a CSR of 0.3. Figure 4.26 displays the number of cycles to failure against the $I_p$. There were eight tests plotted in total; four tests for the tailings from the upper beach (blue), two tests for the tailings from the middle beach (red) and two tests the tailings from the pond (yellow). There is no real trend in the data, as tests can have the same $I_p$ value but very different numbers of cycles to failure. For example, two upper beach tests and one middle beach test all had a $I_p$ of 0.07, but the number of cycles to failure ranged from 14-149.

Figure 4.27 displays the number of cycles to failure against the $\Psi$ for the same tests mentioned above. An interpreted trendline has been plotted as the data do seem to curve slightly. There is a slight trend displaying that as the $\Psi$ increases, the number of cycles to failure decreases. This, however, is not the case for all tests. For example, for a $\Psi$ of -0.31, there is an upper beach sample that fails at 43 cycles, but another upper beach sample with a higher $\Psi$ value of -0.29 fails at cycle number 149. There is a significant difference between the tailings found at the different locations, and
inconsistencies even for the tailings at one location. This is similar to what was identified for Carrera’s tailings in Figure 4.24.

Li’s data were then plotted to display the number of cycles to failure against the $\Psi_m$ in Figure 4.28. There is more spread in the data when comparing them with Figure 4.27, especially for the upper beach material. There is no trend visible, for example, the $\Psi_m$ values for the two pond material tests are very similar (0.00 and 0.02) and the number of cycles to failure are 16 and 6 respectively. There is less agreement in this graph than in the $\Psi$ graph, which is the opposite to what was seen for Carrera’s data. The scatter has increased for each individual soil location and there is no agreement between different soil locations.

Overall, it would seem that the $\Psi_m$ is not a useful tool in predicting static liquefaction or cyclic liquefaction behaviour when the plots are different for different soils. The CSL is also required to calculate the $\Psi_m$, as previously mentioned, which makes calculating the $\Psi_m$ laborious.

### 4.4 Preliminary Tests on Panzihua Tailings

Tests were carried out on the Panzihua tailings to replicate simplistically the layering in tailings dams. It is difficult to obtain intact samples so most work is done on reconstituted samples (see Chapters 2 and 3). The layering is lost in reconstituted samples, so it is important to analyse the behaviour of this simple form of structure. The preparation of these samples was described in section 3.2.3.

A set of three parallel tests (two with coarse tailings, one with fine) were conducted to obtain three $G_s$ values. The tailings are crushed rock so there is no reason to believe that the coarse and fine tailings should have different $G_s$ values. The three
values were calculated to be 3.184, 2.950 and 3.045, and so the average was calculated to be 3.063 (Table 4.5). The results are not within the suggested +/-0.03 of the average value (ASTM, 2002). This is because when applying a vacuum to the material in the pycnometers, a small amount of the material was ejected and lost. These tests have since been repeated. The tests on the fines were unsuccessful and would not work due to the material not mixing with water, while the coarse material confirmed the previous result of 3.063 after completing three tests. The $G_s$ values calculated by Li (2017) were equal to 3.37, 3.14 and 3.11 for UB, MB and PO respectively (Table 4.1). It is unclear why there is a difference between the $G_s$ values calculated by Li and those calculated by the Author, but it could be to do with the ejected material lost during deairing. The value of 3.063 was therefore used for the specific volume calculations. The value indicates that the material has a high density. It is larger than the typical values for sands and clays and this is due to the presence of metal minerals with high specific gravities.

Four successful cyclic tests on different samples were carried out in total; two on mixed slurry samples and two on layered samples. The mixed and layered samples having the same overall particle size distribution. It was also intended that they had the same overall void ratio but there were some small differences. In each case the samples had a $p_i'$ of 100 kPa and were subjected to a cyclic $q$ of +/- 25 kPa or a CSR of 0.25. From the results for layered sample 1, it can be seen that an S-shaped pore pressure response curve was generated that accelerated when failure was approached (Fig. 4.29(a)). The curve is steep with a rapid generation of pore water pressure at the start, followed by a straight central part in which a gradual increase in pore pressure continues for a relatively long time. As the specimen approaches cyclic mobility, there was a more rapid increase in pore pressure. The sample took approximately 1400
cycles to fail. It can also be seen in this graph that the change in pore pressure did quite not reach 100 kPa which was the initial $p'$. The zero readings were checked before each test and the transducers did not drift. This would be because complete liquefaction was not reached but the sample failed through cyclic mobility. The pore pressure increased with cycling and the stress path migrated to a lower $p'$. There was a slight loss of control shown in the graph at very low $p'$ values (Fig. 4.30(a)). This was because the strains were very large and so the apparatus could not keep up with the required stresses. Cyclic mobility was being reached and during each cycle the control systems were no longer able to apply 25 kPa. The apex of the stress path was also not at the $p'$ axis as would normally be expected to be, but there was no offset on the load cell after the test, so the reason for this is uncertain. The strongly dilative stress paths result from the sample having been unloaded to a very low $p'$, and because the stress path cannot cross the peak failure envelope, so on each cycle it simply climbs up the envelope until the maximum applied stress and the sample is failing at the top of each cycle, reaching cyclic mobility, so there is no reason for it to reach $p' = 0$. Figure 4.31(a) gives the axial strain graph for this test showing very large strains when cyclic mobility is reached.

The same CSR parameters were used to test mixed sample 1 and the results can be seen in Figures 4.29(b) – 4.31(b). From the stress path it can be seen that $q = 25$ kPa was again inaccurate for each cycle towards the end of the test. This was again down to the strains being very large, and the apparatus not being able to keep up. The apex of the stress path is also not at the $p'$ axis, but again the reason is unknown. The response of the S-shaped pore pressure response curve and axial strain graph was similar to that for layered sample 1.
Layered sample 2 did not reach cyclic mobility even after being subjected to ten days of cyclic loading. In Figure 4.29(c) it can be seen that the sample was not close to cyclic mobility, as the curve levels out, and the test was therefore stopped. The stress path is given in Figure 4.30(c). There was a small lack of control shown when the sample was under compression even when cyclic mobility was not reached and the strains were small. Figure 4.31(c) gives the axial strain graph. It can be seen that strains are very small and the sample was not near failure.

The final specimen was mixed sample 2. The compressors that supply the lab with air failed meaning that all pressure was lost which could have affected the sample prior to testing. However, the specific volume was accurate because the value that was based on the final water content (Eq 4.5 in Table 4.6) agreed with the other values. So, it is not believed that the compressor failure had an adverse effect on the sample. Again, it can be seen from Figure 4.29(d) that the sample was not close to cyclic mobility after six days of cyclic loading and so the test was stopped. The stress path is given in Figure 4.30(d), and again it can be seen that there was a small lack of control when the sample was under compression. The strains in the axial strain graph in Figure 4.31(d) are very small and the sample was not near failure.

The pore pressure responses from each test have been plotted on the same graph in Figure 4.32. The difference between the first set of tests (layered sample 1 and mixed sample 1) and the second set of tests (layered sample 2 and mixed sample 2) was the grading. From Figure 4.3 it can be seen that the two gradings for the coarse and fine layers for the first set of tests are actually quite similar and they have the same fines content. These coarse and fine fractions had been taken from different samples supplied by Li (2017) from China. A better distinction was therefore made for the second set of tests. Everything that passed through a 63 µm sieve was then classed as
fine-grained (purple curve), and everything that did not was classed as coarse-grained (green curve) (ASTM, 2002). There is a significant difference between these two grading curves.

After sieving, the two samples in the second set of tests both had the same particle distribution size contribution. The fines content of 50% <63µm in these samples was much higher than the fines content in mixed Sample 1 and layered Sample 1, and this is why there is a significant difference in the results.

The specific volumes can be seen in Table 4.6. The results from Eq. 4.5 are mostly low values. This was because it was difficult to retain all of the water from the liquefied samples at the end of the tests. Even if the samples did not liquefy, water was lost because the material is a coarse-grained soil. Removing the membrane also tended to lose some of the water. Therefore, these values were ignored.

The specific volume values for mixed sample 1 and mixed sample 2 are similar, but the pore pressure response graphs, axial strain graphs and stress paths are significantly different. This is because they have different gradings. The second set of samples have a much higher fines content than the first set of samples. Layered sample 1 has a similar specific volume to the mixed sample and quite a similar pore pressure response to mixed sample 1. However, as discussed above, the grading in the two layers were too similar to conclude that layering has no effect. Layered sample 2 has the highest specific volume. It was expected that Layered sample 2 would have a higher specific volume than the mixed samples as the sample consisted of two uniform halves which both have high specific volumes and for which the gradings are very different. Despite having a high specific volume, the layering did not have a big effect on the pore pressure response. It would have been expected that with such a high
overall specific volume, the pore pressure build-up would have been much more rapid than for mixed sample 2 and that failure would have been reached quickly. However, the sample does not fail but reaches a stable state in a similar manner to mixed sample 2. This means that the layering may not have a significant effect on the response. Further testing is required on denser layered samples, with a similar specific volume to mixed sample 2, to confirm whether or not layering does have an effect on the cyclic response. However, a layered sample would be expected to have a higher specific volume than a mixed one for a given preparation procedure or depositional process, as was the case here, because each part of the sample was poorly graded and will therefore have a looser particle packing than the poorly graded mixed sample. The similarity of cyclic behaviour of a looser layered sample and a denser mixed one is consistent with what has previously been observed in the monotonic testing of layered samples, where the main effect of layering was simply an increase of the specific volume with little change of the behaviour (Cotecchia et al., 1999).

4.5 Test Comparisons

Table 4.7 has been combined with the data from the layered and mixed sample tests to create a summary table. Tests UB3 and UB4 had a $p_i'$ of 200 kPa. All other tests had a $p_i'$ of 100 kPa. All of Li’s (2017) tests had cycle periods of two minutes, whilst the mixed and layered sample tests had cycle periods of three minutes. UB3 and UB4 had cyclic amplitudes of 120 kPa whilst the rest of Li’s tests had cyclic amplitudes of 60 kPa. All mixed and layered sample tests had cyclic amplitudes of 50 kPa. At the time the cyclic amplitude was selected for these tests the values used by Li were unknown. The CSR was 0.3 for all of Li’s tests and 0.25 for all of the Author’s tests.
The decay of stiffness during undrained cyclic loading for UB, MB, PO and mixed and layered sample tests are shown in Figure 4.33. The graph is the same as Figure 4.11 but now includes the mixed and layered sample tests. The addition of these tests confirms the weak trend with specific volume, as previously discussed. The stiffesses for the mixed and layered sample tests are quite similar to the stiffesses from the UB, MB and PO tests. Mixed sample 2 (M2) and layered sample 2 (L2) both show a slight decline in stiffness as the number of cycles increases, but both tests are not close to reaching cyclic mobility. These are the only tests that do not reach cyclic mobility. The stiffesses plummet rapidly for tests M1 and L1 after the yield point was reached. It can be seen that for stiffness it does not matter if the sample is layered or mixed, as both M1 and L2 reach the yield point at around similar stiffesses (~80-90 MPa). The version of Figure 4.33 normalised for \( p_i' \) is given in Figure 4.34. Again, it can be seen that the stiffesses on the ‘knees’ are quite consistent, with values around ~0.5-0.8, and that normalisation has not improved the weak trend with specific volume.

As discussed previously, the fines content in mixed sample 2 and layered sample 2 is higher than that in mixed sample 1 and layered sample 1. Mixed sample 2 and layered sample 2 will have a grading closer to MB, and so a comparison will be made between these sets of tests. Although the grading is closer to MB it is important to note that it is not exactly like MB. Mixed sample 1 and layered sample 1 have a grading closer to UB, and so a comparison will be made between these sets of tests. Again, the gradings are not exactly the same, but the UB grading is much more similar to the mixed sample 1 and layered sample 1 gradings than the MB grading is to the mixed sample 2 and layered sample 2 gradings.
The compression paths for the isotropic compression data for tests carried out on layered sample 1, mixed sample 1 and upper beach samples are given in Figure 4.35. The UB samples are not compressing much because the samples are dense and coarse-grained and the stress levels are low. The initial specific volumes for the UB samples are similar to those for mixed sample 1 and layered sample 1, with values ranging between 1.454 and 1.584 (Table 4.7). The $v_i$ for mixed sample 1 was 1.489 and the $v_i$ for layered sample 1 was 1.518 which are also fairly alike. The compression paths for mixed sample 1 and layered sample 1 are therefore similar to those for the UB samples.

The pore pressure response curves for layered sample 1, mixed sample 1 and UB samples are shown in Figure 4.36. The UB samples took considerably fewer cycles to reach failure than mixed sample 1 and layered sample 1. UB1 and UB3 were the loosest and took 20 and 32 cycles to fail, respectively. UB2 and UB4 were denser and took 43 and 149 cycles to fail. Mixed sample 1 had a similar initial specific volume to UB2, and the same $p_i'$, but took 908 cycles to reach failure. This was because the mixed sample test had a lower cyclic amplitude of 50 kPa and a slightly different grading. It should also be noted that the cycle period was not exactly the same. The work carried out by Boulanger et al., (1998) indicates that such small differences in cycle period should not make a significant difference and will not have a noticeable effect on the results. The layered sample took 1381 cycles to reach failure. Despite having a higher specific volume, the layering did not have a big effect on the pore pressure response, as previously discussed, although the layered sample builds up pore pressures slightly less rapidly than the mixed sample, even though it has a higher specific volume. In situ samples will be layered because of the way they are deposited. Retrieving intact samples is very difficult and so these tests show that testing reconstituted samples should be conservative. Layered tests therefore liquefy less
easily than mixed samples and so the effects of layering should not be a concern. The number of cycles to failure can also be seen in Figure 4.37. This graph also takes into account the cyclic amplitude of the tests and, as to be expected, the 50 kPa amplitude tests (grey cycle paths) took the largest number of cycles to reach failure.

Figure 4.38 is a combined $\nu:\ln\rho'$ graph displaying stiffnesses and yield points for layered sample 1, mixed sample 1 and upper beach samples. The yield points have been plotted from Figures 4.33 and 4.34. Contours (although there are only two data points for each amplitude) have been added to this graph and these are displayed in Figure 4.39. There is a black contour going through the yield points for the 60 kPa amplitude tests, a grey contour going through the yield points for the 50 kPa amplitude tests and a red contour going through the yield points for the 120 kPa amplitude tests. For the 50 kPa and 60 kPa amplitude tests, cyclic mobility was triggered at about 20-30 MPa stiffness. Cyclic mobility was reached at ~50 MPa for the 120 kPa amplitude tests. However, there was not a sufficient difference between the gradings of the two halves of the layered sample and so it was ensured that there was a significant difference between the layered sample gradings.

The compression paths for isotropic compression data for tests carried out on layered sample 2 and middle beach samples are given in Figure 4.40. There is no isotropic compression path for mixed sample 2 because the compressors that supply the laboratory with air failed meaning that all pressure was lost. The sample was recovered but no data were recorded during this period. The compression paths for the MB samples are quite straight and show little compression. The MB material is sandy and silty and does not compress very much at these low stresses. The compression path for Layered sample 2 follows almost the same trajectory as the compression path
for the denser MB sample (MB2). The initial specific volumes are very similar, with Layered sample 2 having a $v_i$ of 1.688 and MB2 having a $v_i$ of 1.656.

The pore pressure response curves for layered sample 2, mixed sample 2 and middle beach samples are displayed in Figure 4.41. The curves for layered sample 2 and mixed sample 2 follow a similar trend. These curves are levelling off, indicating that cyclic mobility was not close, even after >2000 cycles. MB1 and MB2 display similar pore pressure response curves and reach cyclic mobility quickly. All tests had a $p_i'$ of 100 kPa, similar $v_i$ values and only slightly different cycle periods. The reasons for the differences in the pore pressure responses are the variations between the cyclic amplitudes and, in particular, the difference between the gradings. Figure 4.42 gives the number of cycles to failure during undrained loading for layered sample 2, mixed sample 2 and MB samples. MB1 took 9 cycles to reach failure, whilst MB2 took 18 cycles. Both mixed sample 2 and layered sample 2 were not close to reaching cyclic mobility, as already shown with the pore pressure response curves.

Figure 4.43 is the combined $v$:ln$p'$ graph displaying stiffnesses and yield points for layered sample 2, mixed sample 2 and middle beach samples. No contours have been added as there are only two data points. When ~20 MPa stiffness was reached, cyclic mobility was again triggered for MB1 and MB2. This was similar to the upper beach samples that had the same cyclic amplitude (60 kPa) as MB1 and MB2 where cyclic mobility was triggered at ~20-30 MPa. The stiffnesses recorded for layered sample 2 and mixed sample 2 are much higher than those for MB1 and MB2. The layered sample has a higher specific volume than the mixed sample and this can be seen in these results. Even though there was this difference, the behaviour tends to be quite similar. This was also seen in the work carried out by Coop and Cotecchia (1997)
where the main effect of layering was an increase in the specific volume but little change in the behaviour.

4.6 Summary

Carrera’s (2008) monotonic test data were reanalysed in terms of $I_B$ against $\Psi$ and no clear trend was visible. Samples had the same $\Psi$ but displayed very different behaviour. There were significant differences between the soils and inconsistencies even for one soil. $I_B$ was plotted against $\Psi_m$ and much better agreement was seen in this graph than in the $\Psi$ one, but there was still no agreement between the different soils. Li’s (2017) cyclic test data were reanalysed and there was a slight trend displayed in the data showing that as the $\Psi$ increased, the number of cycles to failure decreased. This was, however, not the case for all of the tests. There was again a significant difference between the tailings found at the different locations, and inconsistencies even for the tailings at one location. The $\Psi_m$ was examined in terms of number of cycles to failure and no visible trend was observed. There was less agreement in this graph than in the $\Psi$ graph. Overall, there seemed to be little use in using the $\Psi_m$ as a prediction tool for both static and cyclic liquefaction when all the plots are different for different soils.

The contours found in the $\nu:\ln p'$ and $\nu\eta:\ln p'$ planes for the number of cycles to failure were strongly inclined. This meant that both $p'$ and $\nu$ affected the current number of cycles to failure. The contours found in the $\nu\eta:\ln p'$ plane for axial strain and stiffness were slightly inclined. The factors that were most important here were the $p'$ and the amplitude because the specific volume only had a small effect. This will
be examined for the mineral sand data in Chapter 5 to see whether the contours are inclined or vertical.

There was a vertical contour going through the stiffness yield points for the 60 kPa amplitude tests at ~20 MPa and another vertical contour going through the stiffness yield points for the 120 kPa amplitude tests at ~50 MPa on the $vn:lnp'$ graph. When ~20 MPa stiffness was reached, cyclic mobility was triggered for the 60 kPa amplitude tests, and when ~50 MPa stiffness was reached, cyclic mobility is triggered for the 120 kPa amplitude tests. The stiffnesses plummeted after reaching these yield points, however, the yield points do not correspond to anything noticeable on the stress paths.

The mechanical behaviour of the Panzihua iron tailings seemed to be similar across the three localities. The location, and hence grading, was not found to be important and neither was the volume. Overall, the parameters that seemed to be the most important were $p'$ and cyclic amplitude and therefore CSR.

Layered tests liquefied less easily than the mixed samples and so the effects of layering should not be a concern. Li (2017) found that the specific volume and the grading did not have much effect on the cyclic response of tailings and so it is therefore not surprising that the layering also had little effect on the cyclic response.
Table 4.1 - Summary table characterising Panzihua iron tailings (Li, 2017).

<table>
<thead>
<tr>
<th>Tailings type</th>
<th>Iron (UB)</th>
<th>Iron (MB)</th>
<th>Iron(PO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{50}$ (mm)</td>
<td>0.22</td>
<td>0.035</td>
<td>0.023</td>
</tr>
<tr>
<td>$C_u$</td>
<td>10.4</td>
<td>10</td>
<td>6.7</td>
</tr>
<tr>
<td>$C_c$</td>
<td>2.6</td>
<td>1.1</td>
<td>2.2</td>
</tr>
<tr>
<td>$G_s$</td>
<td>3.37</td>
<td>3.14</td>
<td>3.11</td>
</tr>
</tbody>
</table>

Table 4.2 - Mineralogy of Panzihua iron tailings (Li, 2017).

<table>
<thead>
<tr>
<th>Mineral</th>
<th>PO</th>
<th>MB</th>
<th>UB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diopside</td>
<td>28.3%</td>
<td>46.1%</td>
<td>30.2%</td>
</tr>
<tr>
<td>Labradorite</td>
<td>24.1%</td>
<td>40.2%</td>
<td>32.3%</td>
</tr>
<tr>
<td>Hornblende</td>
<td>21.5%</td>
<td>5.5%</td>
<td>11.3%</td>
</tr>
<tr>
<td>Chlorite</td>
<td>16.4%</td>
<td>5.4%</td>
<td>9.0%</td>
</tr>
<tr>
<td>Ilmenite</td>
<td>6.4%</td>
<td>2.7%</td>
<td>6.3%</td>
</tr>
<tr>
<td>Iron Oxide</td>
<td>3.1%</td>
<td>/</td>
<td>11.0%</td>
</tr>
<tr>
<td>Test number</td>
<td>$\nu_i$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------------</td>
<td>---------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper beach 1 (UB1)</td>
<td>1.554</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper beach 2 (UB2)</td>
<td>1.454</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper beach 3 (UB3)</td>
<td>1.584</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper beach 4 (UB4)</td>
<td>1.440</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle beach 1 (MB1)</td>
<td>1.700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle beach 2 (MB2)</td>
<td>1.656</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pond 1 (PO1)</td>
<td>1.906</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pond 2 (PO2)</td>
<td>1.818</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.3 - Initial specific volumes for all tests on the Panzihu iron tailings before isotropic consolidation (Li, 2017).
<table>
<thead>
<tr>
<th>Test name</th>
<th>Soil type</th>
<th>$q_{\text{max}}$ (kPa)</th>
<th>$q_{CS}$ (kPa)</th>
<th>$p'_{CS}$ (kPa)</th>
<th>$p'_i$ (kPa)</th>
<th>$\Delta p'$ (kPa)</th>
<th>$\epsilon$</th>
<th>$\epsilon_{CS}$</th>
<th>$I_B$</th>
<th>$I_P$</th>
<th>$\Psi$</th>
<th>$\Psi_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Te_003_CIU</td>
<td>clean sand</td>
<td>140.5</td>
<td>135.6</td>
<td>190.0</td>
<td>100</td>
<td>90</td>
<td>0.917</td>
<td>0.941</td>
<td>0.04</td>
<td>0.53</td>
<td>-0.02</td>
<td>-0.02</td>
</tr>
<tr>
<td>Te_004_CIU</td>
<td>clean sand</td>
<td>260.2</td>
<td>260.2</td>
<td>160.0</td>
<td>300</td>
<td>140</td>
<td>0.815</td>
<td>0.885</td>
<td>0.00</td>
<td>1.88</td>
<td>0.03</td>
<td>0.01</td>
</tr>
<tr>
<td>Te_005_CAU</td>
<td>clean sand</td>
<td>144.2</td>
<td>144.1</td>
<td>100.0</td>
<td>100</td>
<td>-</td>
<td>0.943</td>
<td>0.940</td>
<td>0.00</td>
<td>1.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Te_008_CIU</td>
<td>clean sand</td>
<td>472.6</td>
<td>448.3</td>
<td>475.0</td>
<td>600</td>
<td>125</td>
<td>0.722</td>
<td>0.797</td>
<td>0.05</td>
<td>1.26</td>
<td>0.08</td>
<td>0.01</td>
</tr>
<tr>
<td>Te_013_CIU</td>
<td>clean sand</td>
<td>184.6</td>
<td>116.1</td>
<td>115.0</td>
<td>300</td>
<td>185</td>
<td>0.639</td>
<td>0.887</td>
<td>0.37</td>
<td>2.61</td>
<td>0.05</td>
<td>0.03</td>
</tr>
<tr>
<td>Te_014_CIU</td>
<td>clean sand</td>
<td>1,549.9</td>
<td>1,475.0</td>
<td>930.0</td>
<td>300</td>
<td>630</td>
<td>0.751</td>
<td>0.883</td>
<td>0.05</td>
<td>0.32</td>
<td>-0.13</td>
<td>-0.21</td>
</tr>
<tr>
<td>Te_015_CIU</td>
<td>clean sand</td>
<td>232.0</td>
<td>91.5</td>
<td>60.0</td>
<td>300</td>
<td>240</td>
<td>0.958</td>
<td>0.887</td>
<td>0.61</td>
<td>5.00</td>
<td>0.07</td>
<td>0.05</td>
</tr>
<tr>
<td>Te_020_CIU</td>
<td>clean sand</td>
<td>1,615.2</td>
<td>1,563.4</td>
<td>1,200.0</td>
<td>300</td>
<td>900</td>
<td>0.716</td>
<td>0.883</td>
<td>0.03</td>
<td>0.25</td>
<td>-0.17</td>
<td>-0.36</td>
</tr>
<tr>
<td>Te_023_CIU</td>
<td>clean sand</td>
<td>1,282.5</td>
<td>1,320.0</td>
<td>960.0</td>
<td>100</td>
<td>860</td>
<td>0.759</td>
<td>0.941</td>
<td>-0.03</td>
<td>0.10</td>
<td>-0.18</td>
<td>-1.19</td>
</tr>
<tr>
<td>Te_027_CIU</td>
<td>clean sand</td>
<td>45.0</td>
<td>2.3</td>
<td>0.1</td>
<td>100</td>
<td>100</td>
<td>1.022</td>
<td>0.941</td>
<td>0.95</td>
<td>1000.0</td>
<td>0.08</td>
<td>0.06</td>
</tr>
<tr>
<td>Te_037_CIU</td>
<td>clean sand</td>
<td>635.9</td>
<td>634.3</td>
<td>500.0</td>
<td>75</td>
<td>425</td>
<td>0.839</td>
<td>0.952</td>
<td>0.00</td>
<td>0.15</td>
<td>-0.11</td>
<td>-0.54</td>
</tr>
<tr>
<td>Te_038_CIU</td>
<td>clean sand</td>
<td>31.8</td>
<td>9.5</td>
<td>13.0</td>
<td>75</td>
<td>62</td>
<td>0.993</td>
<td>0.951</td>
<td>0.70</td>
<td>5.77</td>
<td>0.04</td>
<td>0.03</td>
</tr>
<tr>
<td>Te_05050_024_CIU</td>
<td>50%sand, 50%fines</td>
<td>222.9</td>
<td>50.0</td>
<td>13.0</td>
<td>75</td>
<td>210</td>
<td>0.600</td>
<td>0.561</td>
<td>0.78</td>
<td>3.33</td>
<td>0.04</td>
<td>0.02</td>
</tr>
<tr>
<td>Te_05050_040_CIU</td>
<td>50%sand, 50%fines</td>
<td>32.7</td>
<td>0.0</td>
<td>0.2</td>
<td>75</td>
<td>75</td>
<td>0.658</td>
<td>0.606</td>
<td>1.00</td>
<td>375.0</td>
<td>0.05</td>
<td>0.03</td>
</tr>
<tr>
<td>Te_07030_017_CIU</td>
<td>70%sand, 30%fines</td>
<td>192.2</td>
<td>0.0</td>
<td>0.5</td>
<td>300</td>
<td>300</td>
<td>0.733</td>
<td>0.603</td>
<td>1.00</td>
<td>600.00</td>
<td>0.13</td>
<td>0.10</td>
</tr>
<tr>
<td>Te_07030_019_CIU</td>
<td>70%sand, 30%fines</td>
<td>374.0</td>
<td>98.4</td>
<td>105.0</td>
<td>300</td>
<td>195</td>
<td>0.651</td>
<td>0.607</td>
<td>0.74</td>
<td>2.86</td>
<td>0.04</td>
<td>0.02</td>
</tr>
<tr>
<td>Te_07030_021_CIU</td>
<td>70%sand, 30%fines</td>
<td>739.6</td>
<td>750.0</td>
<td>700.0</td>
<td>300</td>
<td>400</td>
<td>0.544</td>
<td>0.607</td>
<td>-0.01</td>
<td>0.43</td>
<td>-0.06</td>
<td>-0.05</td>
</tr>
<tr>
<td>Te_07030_029_CIU</td>
<td>70%sand, 30%fines</td>
<td>42.3</td>
<td>0.0</td>
<td>0.1</td>
<td>75</td>
<td>75</td>
<td>0.748</td>
<td>0.562</td>
<td>1.00</td>
<td>750.00</td>
<td>0.09</td>
<td>0.06</td>
</tr>
<tr>
<td>Te_07030_032_CIU</td>
<td>70%sand, 30%fines</td>
<td>83.5</td>
<td>0.0</td>
<td>0.1</td>
<td>150</td>
<td>150</td>
<td>0.745</td>
<td>0.640</td>
<td>1.00</td>
<td>1500.00</td>
<td>0.11</td>
<td>0.08</td>
</tr>
<tr>
<td>Te_07030_036_CAU</td>
<td>70%sand, 30%fines</td>
<td>186.5</td>
<td>0.0</td>
<td>0.7</td>
<td>254</td>
<td>253</td>
<td>0.727</td>
<td>0.616</td>
<td>1.00</td>
<td>362.86</td>
<td>0.11</td>
<td>0.08</td>
</tr>
<tr>
<td>Te_07030_041_CAU</td>
<td>70%sand, 30%fines</td>
<td>206.0</td>
<td>65.6</td>
<td>80.0</td>
<td>225</td>
<td>145</td>
<td>0.666</td>
<td>0.620</td>
<td>0.68</td>
<td>2.81</td>
<td>0.05</td>
<td>0.02</td>
</tr>
<tr>
<td>TeF_022_CIU</td>
<td>fines</td>
<td>207.3</td>
<td>138.3</td>
<td>135.0</td>
<td>300</td>
<td>165</td>
<td>0.726</td>
<td>0.695</td>
<td>0.33</td>
<td>2.22</td>
<td>0.03</td>
<td>0.01</td>
</tr>
<tr>
<td>TeF_026_CIU</td>
<td>fines</td>
<td>168.2</td>
<td>167.7</td>
<td>300.0</td>
<td>100</td>
<td>200</td>
<td>0.695</td>
<td>0.734</td>
<td>0.00</td>
<td>0.33</td>
<td>-0.04</td>
<td>-0.05</td>
</tr>
<tr>
<td>TeF_030_CIU</td>
<td>fines</td>
<td>45.6</td>
<td>18.6</td>
<td>23.0</td>
<td>100</td>
<td>77</td>
<td>0.775</td>
<td>0.735</td>
<td>0.59</td>
<td>4.35</td>
<td>0.04</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Table 4.4 - Reanalysis of data from Carrera (2008).
Table 4.5 - Specific gravity calculations for Panzihua iron tailings.

<table>
<thead>
<tr>
<th>Sample type</th>
<th>Mass of pycnometer</th>
<th>Mass of pycnometer + dry sample</th>
<th>Mass of pycnometer + sample + water</th>
<th>Mass of pycnometer + water</th>
<th>Specific gravity</th>
<th>Average specific gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( W_p )</td>
<td>( W_{ps} )</td>
<td>( W_B )</td>
<td>( W_A )</td>
<td>( G_s )</td>
<td>( (3.184 + 2.950 + 3.054)/3 = 3.063 )</td>
</tr>
<tr>
<td>Mixed Sample 1</td>
<td>35.397g</td>
<td>50.105g</td>
<td>95.680g</td>
<td>85.592g</td>
<td>3.184</td>
<td>3.063</td>
</tr>
<tr>
<td>Mixed Sample 2</td>
<td>37.641g</td>
<td>52.372g</td>
<td>98.124g</td>
<td>88.386g</td>
<td>2.950</td>
<td></td>
</tr>
<tr>
<td></td>
<td>32.632g</td>
<td>47.318g</td>
<td>94.458g</td>
<td>84.595g</td>
<td>3.045</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.6 - Specific volume values before isotropic compression on Panzihua iron tailings for: (a) layered samples, and (b) mixed samples. Eq 4.3 uses initial water content method, Eq 4.4 uses dry unit weight method, Eq 4.5 uses final water content method. Green values selected; red values ignored.
<table>
<thead>
<tr>
<th>Test number</th>
<th>$v_i$ (before IC)</th>
<th>$\rho_0'$ (kPa)</th>
<th>Cycle period (s)</th>
<th>Cyclic amplitude (kPa)</th>
<th>No. of cycles to failure</th>
<th>Cyclic stress ratio (CSR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper beach 1 (UB1)</td>
<td>1.554</td>
<td>100</td>
<td>120</td>
<td>60</td>
<td>20</td>
<td>0.3</td>
</tr>
<tr>
<td>Upper beach 2 (UB2)</td>
<td>1.454</td>
<td>100</td>
<td>120</td>
<td>60</td>
<td>43</td>
<td>0.3</td>
</tr>
<tr>
<td>Upper beach 3 (UB3)</td>
<td>1.584</td>
<td>200</td>
<td>120</td>
<td>120</td>
<td>32</td>
<td>0.3</td>
</tr>
<tr>
<td>Upper beach 4 (UB4)</td>
<td>1.440</td>
<td>200</td>
<td>120</td>
<td>120</td>
<td>149</td>
<td>0.3</td>
</tr>
<tr>
<td>Middle beach 1 (MB1)</td>
<td>1.700</td>
<td>100</td>
<td>120</td>
<td>60</td>
<td>8</td>
<td>0.3</td>
</tr>
<tr>
<td>Middle beach 2 (MB2)</td>
<td>1.656</td>
<td>100</td>
<td>120</td>
<td>60</td>
<td>14</td>
<td>0.3</td>
</tr>
<tr>
<td>Pond 1 (PO1)</td>
<td>1.906</td>
<td>100</td>
<td>120</td>
<td>60</td>
<td>6</td>
<td>0.3</td>
</tr>
<tr>
<td>Pond 2 (PO2)</td>
<td>1.818</td>
<td>100</td>
<td>120</td>
<td>60</td>
<td>16</td>
<td>0.3</td>
</tr>
<tr>
<td>Mixed sample 1 (M1)</td>
<td>1.489</td>
<td>100</td>
<td>180</td>
<td>50</td>
<td>908</td>
<td>0.25</td>
</tr>
<tr>
<td>Layered sample 1 (L1)</td>
<td>1.518</td>
<td>100</td>
<td>180</td>
<td>50</td>
<td>1381</td>
<td>0.25</td>
</tr>
<tr>
<td>Mixed sample 2 (M2)</td>
<td>1.479</td>
<td>100</td>
<td>180</td>
<td>50</td>
<td>Infinite</td>
<td>0.25</td>
</tr>
<tr>
<td>Layered sample 2 (L2)</td>
<td>1.688</td>
<td>100</td>
<td>180</td>
<td>50</td>
<td>Infinite</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Table 4.7 - Summary table for all Panzihua iron tailings tests.

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Figure 4.2 - Sketch of tailings dam in Panzihua City with sampling locations (Li, 2017).

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5. Data Analysis and Results

5.1 Introduction

The main focus of this chapter is to investigate the mechanical behaviour of a mineral sand tailings from Australia through compression and shearing testing. Oedometer tests were conducted on reconstituted mineral sand tailings samples to identify whether non-convergent compression behaviour existed. Triaxial tests were carried out to investigate the susceptibility to cyclic liquefaction and also to see whether or not there were well defined NCLs and CSLs. Oedometer tests were also carried out on the iron tailings from Sweden to identify whether non-convergent compression behaviour existed in this material.

5.2 Material Characterisation

5.2.1 Mineral Sand Tailings from Australia

The tailings chosen for the majority of the testing in this study was a mineral sand tailings from about 100 km South of Perth, Australia. As discussed in Section 3.6.2, this was chosen because the separation process for the heavy minerals (generally rutile and zircon) was a gravimetric one, so obviating the need for excessive laboratory safety measures, because no chemicals were used in the process. To accentuate the transitional behaviour, the medium sand part of the fraction was removed by sieving; such grading separation often occurs when tailings are deposited hydraulically. The resulting grading curve is given in Figure 5.1, which has been determined with the Morphologi particle analyser, using the mean of nine analyses because of the small sample size. The remaining grading of the tailings is quite well graded, and compared with many gradings of tailings from Li et al., (2018) is finer-grained. The mean particle
size, $d_{50}$, is 0.087 mm and the coefficient of uniformity, $C_u$, is 4.09. This tailings is slightly less well graded than the pond or middle beach tailings found in the research by Li and Coop (2019) from an iron ore tailings in Panzihua City, China. The $C_u$ value for the pond material was 6.7 and was 10 for the middle beach material (Li and Coop, 2019). The characteristics of the mineral sand tailings after removing the coarser particles are therefore quite similar to many hydraulically placed tailings.

Li and Coop (2019) used the empirical chart proposed by Krumbein and Sloss (1963) to identify a sphericity, $S$, of 0.66 for the pond tailings and 0.58 for the middle beach tailings, and the same roundness, $R$, of 0.34. These relatively low values indicate angular to sub-angular shapes. The same method was used to identify a sphericity of 0.76 for the mineral sand tailings and a roundness of 0.79. The sphericity was also calculated using Equation 5.1, which is equivalent to the circularity defined by Wadell (1933), as mentioned in Section 3.5.3:

$$S = \frac{2\sqrt{\pi A}}{P_{real}}$$  \hspace{1cm} (5.1)

where $P_{real}$ is the particle perimeter and $A$ is the particle area. Using this equation, the sphericity from the Malvern Morphologi apparatus was 0.73. In comparison with Li and Coop (2019), these higher values indicate sub-angular to sub-rounded shaped particles. The grading is therefore similar, but the particle shapes are different and this is because the mineral sand tailings is not a crushed rock. The particles are more rounded and spherical as they have not been crushed.

As previously mentioned in Section 3.5.3, the Morphologi apparatus was used to construct a cumulative frequency graph for aspect ratio, which is used to describe shape and is defined as the particle width divided by particle length. Figure 5.2
displays quite a range of aspect ratios arising from the nature of the material, which underwent no crushing but may have been subjected to some abrasion during the separation process and the wind-blown transportation process from which the parent material originated.

The specific gravity, $G_s$, of the grains was found to be 2.529. This was measured following the procedures recommended in the ASTM D 854 (ASTM 2002). Many tailings tend to have relatively high $G_s$ values due to the presence of metal minerals, but in fact these tailings do not. The plastic limit, $w_p$, of the tailings was 30.1% and the liquid limit, $w_L$, was 59.6%. Loose samples made in the consolidometer were made at a water content determined by multiplying the liquid limit by 1.5 (89.3%). From XRD it was found that the mineralogy of the tailings was predominantly quartz (60%) with kaolin (22%) and feldspar (18%) (refer to Figures 3.34 and 3.35).

5.2.2 Iron Tailings from Sweden

An iron tailings was sent over from Sweden after determining that the mineralogy was safe. As discussed in Section 3.6.2, the arsenic content was low and did not exceed 2.3 ppm. From Table 3.2, the most abundant minerals found in the tailings were phosphorus, manganese, barium, chromium, vanadium and iron. The Morphologi apparatus was used to determine the grading curve which is given in Figure 5.3. The specific gravity, $G_s$, of the tailings was found to be 3.042 which was measured following the procedures recommended in the ASTM D 854 (ASTM 2002). Index tests could not be completed for as the tailings had very low plasticity.
5.3 Testing Programme

Sampling of undisturbed tailings is often impractical and as a result the laboratory tests in this study have been carried out on reconstituted samples. The investigation presented here was also a fundamental investigation of the effects of transitional behaviour. This requires samples of different initial densities which could not be achieved with intact samples. The oedometer and triaxial samples were reconstituted in pastes or slurries, depending on the water content used at the time of mixing, for the mineral sand tailings. Distilled water was gradually added into the dry tailings until the desired water content was reached. For the lower water contents this created a paste, and a slurry was created when higher water contents were used. Research by Shipton and Coop (2012) has suggested that for some transitional soils different sample preparation methods seemed to have no significant effect other than the different initial densities they created, but here the only difference in preparation was the water content of the mix.

A total of 31 triaxial tests were completed on the mineral sand tailings in this study. There were 20 undrained cyclic loading tests, including one overconsolidated sample, 2 undrained cyclic loading tests using lubricated end platens and 9 drained and undrained monotonic tests. A summary of this data is presented in Table 5.1. Four oedometer tests were carried out and details of these are given in Section 5.4.1 below.

Pastes and slurries were placed directly into the oedometer ring, but for the triaxial, the higher water content slurries were first consolidated in a separate consolidometer tube until they were stiff enough to be handled, while the denser pastes could be placed directly into the membrane held on the platen inside a mould. Dense paste samples were also created using the consolidometer to see how the behaviour
would compare with the platen prepared samples. A slurry or paste was spooned into the consolidometer before weights were added in stages and left for at least 24 hours.

Only oedometer tests were conducted on the iron tailings from Sweden, and the reason for this is outlined below.

5.4 Oedometer Tests

5.4.1 Mineral Sand Tailings from Australia

The oedometer apparatus used in this research had a ring diameter of 50 mm. A total of four tests were carried out on the mineral sand tailings, with initial specific volumes, \( v_i \), ranging from 2.081 to 2.883. A summary of the oedometer tests is presented in Table 5.2.

Oedometer tests were carried out to identify whether non-convergent compression behaviour existed. The oedometer compression data are shown in Figure 5.4. There is a clear non-convergence of compression paths from different initial states.

To quantify the degree of convergence, Figure 5.5 illustrates how the initial specific volume may be plotted against that at 225 kPa (\( v_{225} \)) and 5650 kPa (\( v_{5650} \)) for the oedometer tests, where \( v_{225} \) and \( v_{5650} \) correspond to specific loading stages. For consistency the initial specific volume was taken in each test at 20 kPa vertical stress (\( v_{20} \)). For soils with fully convergent paths the gradient of the data on this graph, defined here as \( m \), would be zero and for soils with perfectly parallel compression paths \( m \) would equal one (Ponzoni et al., 2014). The \( m \) values for the oedometer tests are 0.79 and 0.71 for the two stress levels, so the graph displays a clear lack of convergence. Ponzoni et al., (2014) found \( m \) values of 0.40 and 0.38 from tests on silts
and silty clays from the lagoon of Venice. The tests on the mineral sand tailings seem to give quite high $m$ values in comparison and the tailings are therefore highly transitional.

Li and Coop (2018) investigated a silt-sized gold tailings and found an $m$ value of 0.13 at 7MPa from oedometer tests indicating slight transitional behaviour. The tailings had a $d_{50}$ of 0.011 mm and $C_u$ of 7.3 so was fine and quite poorly graded. The oedometer curves only converged at very high stress levels, and convergence was slow, so their tailings was not fully transitional, however, a unique NCL was not reached, even at high stresses. A comparison with the mineral sand tailings would again indicate that the mineral sand tailings is highly transitional.

5.4.2 Iron Tailings from Sweden

A total of three tests were carried out on the iron tailings from Sweden, with initial specific volumes, $v_i$, ranging from 2.122 to 2.762. A summary of the oedometer tests is presented in Table 5.3.

Oedometer tests were carried out to identify whether non-convergent compression behaviour existed. The oedometer compression data for the iron tailings from Sweden are shown in Figure 5.6. From the three tests it is unclear whether there is non-convergence of compression paths from the different initial states. Test 2 and Test 3 do converge to approximately the same point, but it is unclear whether Test 1 would eventually converge with the other tests at higher stress levels.

To quantify the degree of convergence, Figure 5.7 illustrates how the initial specific volume may be plotted against that at 400 kPa ($v_{400}$) and 6400 kPa ($v_{6400}$) for the oedometer tests, where $v_{400}$ and $v_{6400}$ correspond to specific loading stages. For consistency the initial specific volume was taken in each test at 20 kPa vertical
stress, $v_{20}$. As previously mentioned in Section 5.4.1, for soils with fully convergent paths the gradient of $m$ would be zero, and for soils with perfectly parallel compression paths $m$ would equal one (Ponzoni et al., 2014). The $m$ values for these oedometer tests are 0.80 and 0.40 for the two stress levels, and so this graph does display a lack of convergence, although the reduction of $m$ with increasing stress indicates greater convergence than for the mineral sand tailings. The $m$ values for the oedometer tests on the mineral sand tailings were 0.79 and 0.71, and this material was highly transitional with little reduction of $m$. There is quite a difference between the two $m$ values for the iron tailings from Sweden although only three tests were carried out, so it might be advisable to carry out more tests for future work so that the transitional nature of the material can be confirmed, even if it is not strong.

Strong transitional behaviour was identified in the mineral sand tailings from Australia, and so the majority of the testing was carried out on this material for a thorough investigation. Because there was less clear evidence of non-convergence, it was decided that only oedometer tests would be conducted on the iron tailings from Sweden after material characterisation.

### 5.5 Scanning Electron Microscope – Mineral Sand Tailings

Two oedometer tests were carried out on the finer fraction of the mineral sand tailings. One test had a loose initial state and one test had a denser initial state. Weights were added in multiples of two every two hours starting from 100 g and finishing at 6.4 kg which is approximately 300 kPa vertical stress. Once each test was finished, the samples were carefully removed and cut to about 1 cm$^3$. The top and bottom of each sample was carefully recorded and the samples were left to air dry for 24 hours. After
this period, the samples were split in half to expose a fresh surface so that images of the grains could be captured using a SEM. The guidelines for using the SEM were presented in Section 3.7.4. Four images were each produced at different magnifications (x300, x1,500, x5,000 and x10,000) for the loose and dense samples. These images are presented in Figure 5.8. It was important to identify the top and bottom parts of the samples throughout testing to be able to analyse the orientation of the particles correctly.

Figure 5.8(a) displays images of a loose sample made using a water content of 89.33%, which is 1.5 times the liquid limit, and Figure 5.8(b) displays images of a dense sample made using a water content of 35%. From looking at the x300 magnification images, it is clear that the image on the right is denser than the one on the left. It looks as though the clay minerals are coating the coarser particles. In the x1,500 magnification images, the dense sample displays some orientated particles. There is a domain of diagonally orientated particles. This seems to be less obvious in the image of the loose sample. Again, there is a domain of orientated particles in the x5,000 magnification image for the dense sample. These are orientated horizontally. The particles are less orientated in the image of the loose sample. At a magnification of x10,000 a wide range of particle sizes can be seen. There seems to be aggregated clumps in both images, but more horizontally orientated particles in the dense sample. The loose sample image displays large voids with a heterogenous distribution.

Overall, there is a little more particle orientation in the denser sample, but the orientations do not correspond to vertical or horizontal directions. There is nothing particularly striking as a difference between the loose and the dense samples. The grading agrees with the particle sizes and some of the flatter particles are likely to be
the clay minerals. The silt particles would be in the 10s of microns but it looks like these have been covered with clay.

5.6 Isotropic Compression – Mineral Sand Tailings

The isotropic compression and shearing paths for the monotonic tests can be seen in Figure 5.9, and the isotropic compression data for the cyclic tests are given in Figure 5.10. The stress level is represented by the mean normal effective stress, \( p' \), defined as:

\[
p' = \left( \sigma'_a + 2\sigma'_r \right)/3
\]  

(5.2)

where \( \sigma'_a \) is the axial effective stress and \( \sigma'_r \) the radial effective stress. The initial mean effective stress, \( p'_i \), for samples that were cyclically sheared ranged from 100 kPa to 400 kPa. Monotonically sheared samples had a \( p'_i \) ranging from 50 kPa to 400 kPa. The triaxial samples were all isotropically consolidated after saturation by increasing the confining pressure by 3.5 kPa/hour for standard tests (see Section 3.2.2) and 2 kPa/hour for lubricated end platen tests (see Section 3.2.5). The triaxial isotropic compression was therefore carried out slowly but nevertheless the small “tails” at the end of each compression curve represent a small amount of undissipated pore pressure. Some tails have been plotted in Figure 5.9. A combined graph showing the compression data for all triaxial and oedometer tests is given in Figure 5.11. While the oedometer data appear almost parallel, there is some convergence in isotropic compression. For transitional soils, different apparatus will give different convergence rates depending on their combination of applied volumetric and shear strains (Todisco and Coop 2019). However, there is still no evidence of any unique single NCL up to 400 kPa. Test Mt2 was a paste sample (i.e., dense) that had been compressed in a
consolidometer, like the loose samples, but its compression behaviour is very similar to tests Cyc8 and Cyc11 that were pastes directly placed on the platen (Fig. 5.12).

The $m$ value graph for isotropic compression is given in Figure 5.13. The $m$ values for the triaxial tests are 0.73, 0.49 and 0.35. For the oedometer tests, the $m$ values show little change with larger stresses. A combined $m$ value graph for isotropic compression and one-dimensional compression is displayed in Figure 5.14. In comparison, the triaxial tests show a greater change in $m$ values and at lower stress levels indicating more convergence.

5.7 Monotonic Shearing – Mineral Sand Tailings

5.7.1 Introduction

The isotropic compression data were summarised in Table 5.1. Samples with different initial densities were created to identify whether or not non-convergent behaviour was present. Samples were prepared in two ways for the monotonic tests. Seven tests were prepared using a consolidometer and two tests were prepared directly on the platen. Sample preparation methods for triaxial testing were described in Section 3.2.

The samples were isotropically consolidated after saturation. The mean initial effective stresses, $p_i'$, the samples were sheared from ranged from 50 kPa to 400 kPa. The paths followed during shearing are shown on Figure 5.15. The compression curves are non-convergent and no unique IC NCL could be identified. No unique CSL could be identified and so five parallel CSLs have been plotted. The gradient was taken from the four tests that have a similar $v_i$. The CSL for Test 6 is slightly different to that of the other tests and the reason for this is described in the paragraph below. The CSLs appear to be straight, even at low $p'$ values, and there is no evidence of curvature. This
was even the case for Test 5 which was a loose sample sheared at a low $p'$ value. This shows similar behaviour to clays. It was hoped that these CSLs would be curved but in fact they are not, or not within the stresses the samples have been tested at in this study (refer to Section 4.3). Given the significant difference in initial density, and the fact that they would be expected to converge to a unique critical state specific volume on a unique CSL, the volume changes should have been very different for different initial densities, which they are not. It is clear that both loose and dense samples compress and are not converging to a unique CSL. Shipton and Coop (2015) found that for sand-clay mixtures, no matter how dense the sample is initially, the samples always compressed on shearing. Creating a sample dense enough to dilate was not possible, which was also the case here. Shipton and Coop (2015) found parallel CSLs for different initial densities in triaxial tests, and this is likely to be the case also here. Different sample preparation methods did not make a significant difference to the mechanical behaviour of the tailings as will be seen in the gamma plot in Section 5.7.5.

5.7.2 Block 2 Comparison

A second block was used for some tests which will be referred to as ‘block 2’ which had a slightly different colouration to the other mineral sand tailings tested (block 1). Tests were conducted on this sample to see if there were any differences in behaviour between two blocks. The grading of block 1 and block 2 was very similar and an average was taken to produce the overall grading curve seen in Figure 5.1. One monotonic test and two cyclic tests were carried out on block 2. The cyclic tests will be discussed in Section 5.8. The monotonic test on block 2 was Test 6. As can be seen in Figure 5.15, the test displays very similar behaviour in comparison with the other tests. The CSL for Test 6 is in a location consistent with those on block 1.
5.7.3 Volumetric Strain and Axial Strain

The volume change, or creep, was allowed to stabilise after isotropic compression before shearing commenced. Figure 5.16 is a plot of volumetric strain against time for drained Test 1 and undrained Test 5. For Test 1, there was stabilisation when the volumetric strain reached ~0.5%, but it should be noted that this was mostly consolidation and not creep. Test 5 was stable after approximately 1000 minutes. The monotonic tests were therefore left for approximately 24 hours after reaching the desired $p'$ before shearing. This allowed the rate of volume change to be small, which was also much smaller than the rate of strain during shearing.

Some of the monotonic tests were slightly incomplete due to mechanical constraints on the triaxial apparatus stopping any further shearing. The shearing paths were therefore completed through hyperbolic extrapolation up to 30% axial strain. This was done to estimate the volumetric strains at the critical states more accurately. Volumetric strain, $\varepsilon_v$, has been plotted against axial strain, $\varepsilon_a$, for all drained tests in Figure 5.17, and it can be seen that all the volumetric strains were contractive. With the exception of Test 9, all tests display contractive volumetric strains initially and are becoming stable. Figure 5.18 presents the extrapolated curve for monotonic Test 1. The specific volume could then be calculated from the extrapolated $\varepsilon_v$ to identify the ‘end of shearing’. These points were then plotted on Figure 5.9.

The curve in the shearing path for monotonic Test 9 is slightly unusual in Figure 5.17, but it corresponds to smaller volumetric strains at the start of shearing. It can be seen from the figure that the volumetric strain did stabilise at the end of the test and so there was no leak into the sample. The specific volumes calculated for this test also tie up so this seems to be correct.
The deviator stress, $q$, has been plotted against axial strain, $\varepsilon_a\%$, for drained and undrained monotonic tests in Figure 5.19. The undrained tests reached a maximum $q$ quite quickly and then began to level off after $\approx 1 \varepsilon_a\%$. There is a significant difference in the $q$ values for the drained tests which increased more gradually.

5.7.4 Stress-Strain Data and Stress Paths

The stress-strain data for all drained and undrained tests are given in Figure 5.20. To eliminate the effect of the confining pressure, the deviatoric stress, $q$, is normalised by the $p'$ at the start of shearing. It is disappointing to see that the graph displays so much scatter and that some of the tests are incomplete. Test 4 only plots up to $\approx 8 \varepsilon_a\%$. The original data has been checked and this was due to there being no more $\varepsilon_a$ movement available during shearing. There are some jumps in the data for Test 6. This was because of the noise on the pore pressure transducer. Many of the tests are tending to $q/p' = 1$ with the exception of Test 5. Again, hyperbolic extrapolation was used on incomplete tests up to $25 \varepsilon_a\%$. Figure 5.21 gives the extrapolated curve for monotonic Test 1 which levels off at around $q/p' = 1$.

The change in pore pressure, $\Delta u$, has been plotted against the axial strain, $\varepsilon_a\%$, for the undrained monotonic tests in Figure 5.22. At the end of Test 6, the changes in pore water pressure became stable, meaning that the test was complete. Test 5 seems to be gradually increasing but only very slowly. Test 7 reached a peak in the $\Delta u$ at approximately four $\varepsilon_a\%$. At this point, the $\Delta u$ started to gradually decrease.

The stress paths of the mineral sand tailings are shown in Figure 5.23. It can be seen that the end points of these triaxial tests define a unique CSL in the $q:p'$ space and the gradient of this line $M$ is 0.98. This is the average value within a lot of scatter, but the tests are generally still converging when they were terminated. This gives a
critical state angle of shearing resistance, $\phi'_cs$, of 24.9°. This was calculated using equation 5.3.

\[
sin\phi'_cs = \frac{3M}{6+M}
\] (5.3)

The M value of Test 6, which was on a sample taken from block 2, is slightly higher at 1.21. This gives a $\phi'_cs$ of 30.3°. The majority of the testing was carried out on samples reconstituted from block 1 even if both materials were very similar in the $v:lnp'$ plane, but as it can be seen there is a slight difference in terms of the M values and therefore the $\phi'_cs$ values. This would suggest that the different blocks seem to have slightly different particle shapes and sizes.

5.7.5 Preparation Method Analysis

Gamma, $\Gamma$, has been plotted against the initial specific volume at 20 kPa vertical stress, $v_{20}$, for the monotonic tests in Figure 5.24. $\Gamma$ is the projected intercept in the $v:lnp'$ plane of the CSL at 1 kPa. The locations of the CSLs offer another means of quantifying transitional behaviour. Test 6 exhibits similar behaviour to the other tests and plots just below the trendline. As can be seen in the figure, the two different sample preparation methods did not make a significant difference to the mechanical behaviour of the tailings. The gradient is defined as $P$, which has the same limiting values as $m = 1$ for perfectly transitional behaviour. The $P$ value is 0.49 which indicates there was no more convergence during shearing. As discussed in Section 5.6 the $m$ values for isotropic compression were 0.73, 0.49 and 0.35 at different stress levels. The $P$ value is in the middle range of the $m$ values. The $m$ values for the oedometer tests were 0.79 and 0.71. It is surprising that the $m$ values are higher in one-dimensional compression than isotropic compression. Ponzoni et al., (2014) found $P$ values of 0.72 and 0.58 for triaxial tests and $m$ values of 0.40 and 0.38 for oedometer
tests from tests on silts and silty clays from the lagoon of Venice. The $P$ value is typically higher than the $m$ because of the higher stresses reached in compression. However, the $P$ value is lower than some of the $m$ values for the mineral sand tailings which contradicts this.

5.7.6 Normalisation of Stress Paths

The stress paths have been normalised by equivalent pressures on the NCL, $p'_e$, as shown in Figure 5.25, and assuming a different NCL for every sample. The definition of $p'_e$ is given in Equation 5.4:

$$p'_e = \exp \left( \frac{N-v}{\lambda} \right)$$

(5.4)

where $N$ is the projected intercept of the isotropic NCL at 1 kPa and $\lambda$ is the gradient.

It is unclear why Test 4 plots lower than the other drained tests. Test 7 gives the appearance of being overconsolidated, however, all monotonic tests were normally consolidated. From Figure 5.15 it can be seen that the isotropic compression line is still curving for Test 7. For this very dense sample, it does not look to be on its unique NCL yet and gives the appearance of being overconsolidated in the normalisation figures.

Normalisation may be applied to the stress path to obtain the state boundary surface (SBS), which separates the possible and impossible states of a soil. A SBS can be identified in Figure 5.26. The CSL and NCL form part of the SBS. The undrained tests tend to plot below the SBS. Rendulic’s principle states that drained and undrained tests plot on the same SBS (Rendulic, 1936). These tests do not follow this principle, much like sand. Some of the drained tests give one curve and the undrained tests generally plot below the drained tests, which is quite common. The $q/p'_e$ values were further divided by $M$ so that all tests finished at the same point.
The stress paths have also been normalised by equivalent pressures on the CSL, \( p'_{cs} \), as shown in Figure 5.27, again using a different CSL for each sample. The definition of \( p'_{cs} \) for straight CSLs is given in Equation 5.5:

\[
p'_{cs} = \exp \left( \frac{\Gamma - v}{\lambda} \right)
\]  

(5.5)

where \( \lambda \) and \( \Gamma \) are the gradient and the intercept at 1 kPa of a straight CSL. Arrows have been added to the incomplete tests on the graph. The CSL has been plotted at point 0.98:1, where 0.98 is the M value for block 1 tests.

5.7.7 Stress Dilatancy Curves

The stress dilatancy curves for the drained monotonic tests on the mineral sand tailings have been plotted in Figure 5.28. The gradient of the volume strain-shear strain graph was calculated by using an odd number of data points in the regression, symmetrical about the current line. This is because the gradient is associated with the \( q/p' \) value for the middle line. The CSL was plotted as a point on the graph using the M value of 0.98. It was expected that all points would finish at the CSL. All of the tests should define the same line. Tests 1, 2 and 8 plot similarly, but tests 3 and 4 plot slightly lower than the CSL. This was not due to creep or the stress levels. None of the samples were dense enough to dilate. Test 9 was removed from this graph because of the unusual volumetric strains.

5.7.8 Key Findings from Monotonic Shearing

In summary, a number of compression and shearing tests were carried out on mineral sand tailings from Australia to characterise the behaviour. Two different blocks were discovered. Block 2 was found to exhibit a similar behaviour to block 1 in the \( v:\ln p' \) plane but the majority of testing was carried out on block 1. It was found that for reconstituted samples made at different initial densities using two different sample
preparations methods, no unique CSL could be plotted on a $v:\ln p'$ graph. Both loose and dense samples compressed on shearing but did not converge to a unique CSL indicating strong transitional behaviour. The CSLs were also straight, even at low $p'$ values, showing a similar behaviour to clays. In the $q:p'$ plane, however, a unique CSL could be defined.

5.8 Cyclic Behaviour – Mineral Sand Tailings

5.8.1 Introduction

The cyclic loading data are summarised in Table 5.1. Samples with different initial densities were created and a variety of CSRs were applied to examine the cyclic response. The CSR was introduced in Section 2.6.6. It is defined as the cyclic deviatoric stress, $q$, divided by twice the effective confining pressure, $\sigma'_c$. The CSRs chosen for this study were 0.125, 0.25 and 0.5. From looking at the results a CSR of 0.5 was probably too high as some tests failed before completing one cycle. In retrospect a CSR of 0.4 would have been a more suitable choice but this emerged too late to change the testing programme that was covid affected. Note that the threshold to determine cyclic failure or cyclic liquefaction was +/-5% $\epsilon_a$, as described in Section 2.4.3. As mentioned in Section 3.3.3, all axial strains were corrected for compliance.

Samples were prepared in two ways for the cyclic tests. Twelve samples were prepared using a consolidometer and two of these were tested using lubricated end platens. Ten samples were prepared directly on the platen. Sample preparation methods for triaxial testing were described in Section 3.2.
5.8.2 Unsuccessful Tests

There were five cyclic tests that were excluded from the data analysis and results. Two lubricated end platen tests were discounted. The first lubricated end platen test was unsuccessful as the drainage holes in the polished steel plate on the platen became clogged with tailings as there was no porous stone. This meant that the $p'$ value was not accurate because the pore pressure measurement was incorrect and therefore the effective stress was much lower than the required 200 kPa, and so the test failed on the first cycle. The steel plate was therefore adapted, as described in Section 3.2.5. There was an electrical fault in the laboratory during isotropic compression for the second lubricated end platen test which meant that the sample was destroyed. Three standard cyclic tests were not included in the results because the volumetric strain had not stabilised sufficiently before cyclic testing commenced. As previously mentioned in Section 3.2.2, no cyclic loading should be applied until the volumetric strain due to creep is stable. All of the values for the successful cyclic tests and monotonic tests were less than or equal to 0.002%/h as can be seen in Table 5.1.

5.8.3 Pore Pressure Response

All the successful cyclic tests have been plotted in Figure 5.29 which displays the change in pore pressure plotted against the number of cycles. Two tests, CYC3 and CYC4, were tests carried out on block 2 which was introduced in Section 5.7.2. Tests were carried out on the different blocks to see if there were any differences in behaviour between the two blocks. This is discussed below. All tests were normally consolidated with the exception of CYC19 which was overconsolidated. It can be seen from the graph that the tests that reached cyclic mobility or cyclic liquefaction saw a rapid increase in the $\Delta u$ in less than ~200 cycles. Those tests that levelled off horizontally were ‘infinite’ tests that were not close to failure. The same graph is
displayed in Figure 5.30 but a logarithmic scale has been used. This shows clearly that the infinite tests are really not approaching failure, and a greater number of cycles would not have caused them to fail.

Figure 5.31 shows the $\Delta u$ divided by the $p_i'$ for all cyclic tests. All tests approaching $\Delta u/p_i' = 1$ are reaching cyclic liquefaction. This is apparent in the axial strain graphs and this is discussed below. This graph has been further broken down into three separate graphs which correspond to the three different CSRs. Figure 5.32 displays the graph for the CSR of 0.125. Seven cyclic tests had a CSR of 0.125 including one test on block 2 and one lubricated end platen test. All tests were infinite tests and did not show signs of failure even after +1700 cycles. Figure 5.33 shows the graph for the CSR of 0.25. Seven cyclic tests had a CSR of 0.25 including one test on an overconsolidated sample, and one test using lubricated end platens. Five of the tests reached a high $\Delta u/p_i'$, often approaching 1, whilst two tests did not approach failure. These tests were CYC17 and CYC19. Test CYC17 was a sample with a low $\nu_i$, so the sample probably had not reached its NCL. This was made on the base of the platen and had a low cyclic amplitude of 50 kPa. Test CYC19 was the test on the overconsolidated sample. Note that the cycling for CYC19 was stopped after 556 cycles due to time constraints. The pore pressure response was checked before the test was stopped which indicated that it was an infinite test. The same data have been plotted in Figure 5.34 but only up to 200 cycles so that the curves for the tests that approached $\Delta u = 1$ can be seen. The shapes of the curves that reached failure are all similar, with LEP1 reaching failure in 66 cycles, CYC16 in 80 cycles, CYC10 in 131 cycles, CYC9 in 157 cycles and CYC5 in 212 cycles. Figure 5.35 shows the graph for the CSR of 0.5. Eight tests had a CSR of 0.5 included one test on block 2. All tests reached cyclic liquefaction. It can be seen that the control was not particularly good.
for CYC1 with the test failing before it reaches one cycle, and this rapid failure was
the cause of the poor control.

5.8.4 Axial Strain

The combined $\varepsilon_a\%$ plotted against the number of cycles to failure graph for all the
cyclic tests are given in Figure 5.36. As mentioned above, +/-5% $\varepsilon_a$ meant that tests
had reached cyclic failure. The tests that reached failure all did so in <250 cycles. The
same graph has been plotted on a logarithmic scale in Figure 5.37, again to show that
the infinite tests are not approaching failure. The linear graph has again been further
broken down into three separate graphs which correspond to the three different CSRs.
Figure 5.38 displays the graph for the CSR of 0.125. CYC2 shows an $\varepsilon_a\%$ of 0.4
which looks quite large on the graph but this is because the graph plots only to 0.5
$\varepsilon_a\%$. All other tests on this graph show an $\varepsilon_a\%$ of ~0.1. Figure 5.39 displays the graph
for the CSR of 0.25. The same data has been plotted in Figure 5.40 but only up to 200
cycles so that the curves for the tests that reached +/-5% $\varepsilon_a$ can be seen. This plot
shows that most of these tests fail in extension, as expected. Finally, Figure 5.41 gives
the graph for the CSR of 0.5. The majority of the tests reached failure in one cycle.
CYC4 takes 11 cycles to reach failure, and CYC8 takes 8 cycles. CYC4 was a sample
made from the block 2 material and CYC8 had a relatively low $v_t$ but was still higher
than that of CYC13 and CYC14 which failed in 1 cycle. This will be examined further
on in the chapter.

5.8.5 Stress Paths

Figure 5.42 gives three different stress paths for tests that ranged from one cycle to
failure, to an infinite cycle test. Figure 5.42(a) is for CYC7 which took just over one
cycle to reach failure which occurred in compression. The $p'$ decreased rapidly from
200 kPa and the stress path became less well controlled as failure approached. Figure
5.42(b) is for CYC10 which took 131 cycles to reach failure. The $p'$ decreased quickly at first but then slowed down and became more consistent before reaching the failure envelope. From Figure 5.33 we can see that the $\Delta u/p'_i$ approached one for CYC10 and this corresponded with the $\varepsilon_a$ reaching -5% in Figures 5.39 and 5.40, and so we know that failure occurred at 131 cycles. Figure 5.42(c) is for CYC15 which was an infinite test and did not fail after >2000 cycles. The $p'$ reduced by ~40 kPa, but the sample was not approaching failure. Note the basic data for all of the cyclic tests on the mineral sand tailings have been compiled in the appendix.

5.8.6 Block 2 Comparison

One of the tests on the block 2 material, CYC3, had similar testing parameters to test CYC2, which was on the block 1 material. These will therefore be compared to see if there was a different mechanical response between the two tests during cyclic testing. The $p'_i$ for both tests was 400 kPa and the CSR was 0.125. Test CYC2 had a $v_i$ of 2.022 and Test CYC3 had a $v_i$ of 2.192. The graph displaying the $\Delta u/p'_i$ plotted against the number of cycles is given in Figure 5.43. The plots are similar in shape and both tests exhibit similar behaviour in that they are both infinite tests even after >1000 cycles. Block 2 samples seem to behave fairly similarly to block 1 samples, although there was a greater pore pressure increase in CYC2. An orange line has also been plotted on this graph which displays the mean value over one cycle.

5.8.7 Lubricated End Platen Comparison

Figure 5.44 displays the $\Delta u/p'_i$ plotted against the number of cycles for lubricated end platen test 1 (LEP1) and CYC16. Both of these tests had similar testing parameters and so were chosen for comparison to see if the lubricated end platens had any effect on the results. These graphs also display the S-shaped curves as seen previously in Figure 2.8 of the literature review and Section 4.4 of Chapter 4. The shape of these
curves is commonly seen in pore pressure response graphs where cyclic liquefaction occurs. The $p'_i$ for both tests was 200 kPa and the CSR was 0.25. Test LEP1 had a $v_i$ of 2.012 and Test CYC16 had a $v_i$ of 2.016. Both tests fail in a similar number of cycles, with LEP1 failing after 66 cycles and CYC16 after 80 cycles. These tests are not failing at $\Delta u/p'_i = 1$ because the stress paths are not passing through a $p' = 0$, so the pore pressure is not reaching 1, but -5% $\varepsilon_a$ has been reached. The stress paths are shown in Figure 5.45 whilst the $\varepsilon_a$ graphs were given in Figures 5.39 and 5.40.

Lubricated end platen test 2 (LEP2) was compared with CYC20 and the graphs showing the $\Delta u/p'_i$ plotted against the number of cycles are given in Figure 5.46. The $p'_i$ for both tests was 200 kPa and the CSR was 0.125. Test LEP2 had a $v_i$ of 2.012 and Test CYC20 had a $v_i$ of 2.156. Both graphs display a similar shape and both show a decrease in the $\Delta u/p'_i$ as the cycles progress. Both tests are infinite tests and display no signs of failure even after >2000 cycles. The cycle amplitude of $\Delta u$ is smaller for LEP tests. This may be due to restrictions with the drainage and this may indicate that cycle periods should have been slower for the LEP tests. Test LEP1 was, however, already at a lower frequency at 10-minute cycles, but it was decided that LEP2 should have the same cycle period as all other tests at 5-minutes because it was going to be an infinite cycle test and so there weren’t going to be any large changes in pore pressure. Overall, however, it would seem that the addition of lubricated end platens had no significant effect on the results.

5.8.8 Overconsolidated Sample Comparison

All tests were normally consolidated with the exception of CYC19 which was overconsolidated. Figure 5.47 displays the $\Delta u/p'_i$ plotted against the number of cycles for CYC19 and CYC9. The $p'_i$ for both tests was 200 kPa and the CSR was 0.25. Test CYC19 had a $v_i$ of 2.087 and Test CYC9 had a $v_i$ of 1.863. CYC19 did not reach
failure after 2000 cycles, even though the $v_i$ was higher than CYC9, and CYC9 failed after 157 cycles. The testing parameters were fairly similar, but the test responses were very different between the two tests because CYC19 had been overconsolidated and therefore the sample had more resistance to cyclic loading than CYC9.

5.8.9 Cyclic Stress Ratio and the Number of Cycles to Failure

The isotropic compression data and cyclic paths for all 20 cyclic tests carried out in this research are displayed in Figure 5.48. The number of cycles to failure have been added to the graph with infinity symbols, $\infty$, added for tests that did not approach cyclic mobility or cyclic liquefaction after a significant number of cycles. As previously mentioned in Section 2.4.3, the threshold to determine failure was +/-5% $\varepsilon_a$. An alarm was set on the displacement so that cycling was stopped if +/-5 mm was reached, which is greater than +/-5% $\varepsilon_a$. If this did not occur, cycling continued for approximately 1700-2200 cycles which equates to 6-8 days with each cycle being 5-minutes. It was decided that this was a sufficient amount of time to determine whether or not the sample was approaching failure or not by tracking the pore pressure response. An exception to this was CYC19, as previously mentioned earlier on in this section, which was stopped early due to time constraints. Also, one LEP test had 10-minute cycle periods, as mentioned in Section 3.2.5. The total number of cycles to failure for each test are given in Table 5.1 and are also labelled in the figure at the end of the compression paths. All of the tests are undrained tests and so the cyclic paths are horizontal and are moving to the left as the $p'$ reduces. On these cyclic paths, 100 cycles to failure, 10 cycles to failure and 1 cycle to failure have been plotted where possible. The cyclic path terminated when the test was stopped or because the sample had reached failure.
The same graph is presented in Figure 5.49, but tests have been colour-coded by CSR. Tests with a CSR of 0.125 are in red, tests with a CSR of 0.25 are in blue, and tests with a CSR of 0.5 are in orange. This graph is further broken down into three separate graphs which display the cyclic paths and number of cycles to failure for each CSR.

Figure 5.50 displays the 0.125 CSR tests. A total of seven tests were carried out at this CSR, including one lubricated end platen test, and the $v_i$ values ranged from 1.880 to 2.192. All tests were infinite tests regardless of the $v_i$ and $p_i'$. Test CYC2 has been labelled on the graph which is the test that displayed a higher pore pressure response in Figure 5.32. This test had a higher creep value prior to cycling than the other tests with the same CSR, and so this could have been the cause of the greater pore pressure response. Perhaps the test should have been left longer before cycling commenced. Other than this test, there was no significant difference in the pore pressure responses between the tests.

Figure 5.51 displays the 0.25 CSR tests. A total of seven tests were carried out at this CSR, including one lubricated end platen test and one overconsolidated test, and the $v_i$ values ranged from 1.863 to 2.540. Figure 5.33 gave the pore pressure responses of the tests at this CSR, and CYC17 and the overconsolidated test (CYC19) were infinite cycle tests, while all other tests failed. Test CYC17 displayed a small reduction of $p'$ corresponding to a small change in pore pressure. It is likely that CYC17 had not reached its NCL before cyclic loading commenced. The pore pressure response of LEP1 was similar to that of CYC16, as described in Section 5.8.7, and both tests displayed similar cycles to failure, indicating that the lubricated end platens made little difference to the results. Leaving aside the infinite cycle tests, contours were added to the plot. Three contours have been added for 1 cycle to failure, 10 cycles to failure and
100 cycles to failure. They are all vertical within the scatter of data. An average of the $p'$ values was taken for the 1 and 10 cycle contours, but the 100 cycles to fail contour was an estimate, as some tests failed in under 100 cycles.

The contour for 1 cycle to failure indicates that all tests roughly end up at the same $p'$ value for 1 cycle to failure, regardless of the $p'_i$. All the $p'$ values at this contour are quite similar. For the 10 cycles to failure contour, the $p'_i$ does make a bit of difference in determining at what $p'$ there are 10 cycles to failure, for example, at 400 kPa $p'_i$ tests reach 10 cycles to failure at a higher $p'$ value than the 200 kPa $p'_i$ tests. Note that this contour is not entirely accurate as there is a bit of scatter in the data for 10 cycles to failure. It is difficult to say whether or not there is a clear trend for the 100 cycles to failure contour as the contour is an estimate, and there aren’t enough data points at 100 cycles to failure. Contours were added to Li’s (2017) data in Figure 4.8, in Chapter 4, to indicate two and ten cycles to failure. These contours were inclined and meant that both $p'$ and $v$ affected the number of cycles to failure. For conventional soils with NCLs, the contours were inclined. There is no convincing gradient with the mineral sand tailings.

There is no clear increase in the number of cycles as the $v_i$ reduces. For example, the test with the highest $v_i$ (2.540) had a $p'_i$ of 400 kPa and took 212 cycles to fail, and the test with the lowest $v_i$ (1.863) had a $p'_i$ of 200 kPa and took 157 cycles to fail. CYC17 had a $v_i$ of 1.891 and a $p'_i$ of 100 kPa at the start of cyclic testing, but was an infinite cycle test. There is therefore no correlation with $v_i$ and the $p'_i$.

The cyclic paths and number of cycles to failure for cyclic tests with a CSR of 0.5 are presented in Figure 5.52. A total of eight tests was carried out at this CSR with the $v_i$ values ranged from 1.843 to 2.345. The majority of the tests took one cycle to
fail. The test that took 11 cycles to fail was CYC4 and was a sample made from the block 2 material. This could be the reason the sample took 11 cycles to fail and not one cycle like the majority of the tests in the plot. The test that took eight cycles to fail was CYC8 and looks as though it was not on its NCL before cycling commenced, which could explain why it took more cycles to fail than the other tests. From this graph it can be seen that the $p_i'$ and $v_i$ make no difference to the number of cycles to failure. If the $v_i$ had been changed by means of overconsolidation, like test CYC19, then the number of cycles to failure would have been affected.

From examining the three plots, it would seem that the $v_i$ does not make a significant difference to the number of cycles to failure in undrained cycling for this material. This is a key feature of this transitional soil. This behaviour was seen in the literature review in Section 2.6.8 for monotonic loading, but this has now been seen in cyclic loading. It would also seem that the $p_i'$ also has no substantial affect of the number of cycles to failure.

The CSR has been plotted against the number of cycles to failure for all cyclic tests on the mineral sand tailings in Figure 5.53. The points plotted in purple had a $p_i'$ of 100 kPa, the points plotted in green had a $p_i'$ of 200 kPa, and the points plotted in red had a $p_i'$ of 400 kPa. The $v_i$ values have been plotted next to their corresponding points. An estimated interpreted trendline has also been plotted to follow all the normally consolidated tests apart from those that were made of the block 2 material and those that had not reached the NCL before cyclic loading commenced. Test CYC4 was a sample made from the block 2 material, tests CYC8 and CYC17 had not reached their NCLs before starting the cyclic loading and test CYC19 was an overconsolidated sample. The lubricated end platen tests have also been labelled on the plot, and exhibit similar behaviour to those tests that were carried out with similar parameters but with
the absence of lubricated end platens. The $v_i$ values are randomly scattered and the $p_i'$ values do not make a difference to the number of cycles to failure. The trendline indicates that as the CSR decreases, the number of cycles increases, as would be expected. This was seen in Figure 2.31 under Section 2.6.6 of the literature review.

This graph has been further broken down into three separate graphs which display the individual plots for the $p_i'$ of 100 kPa, 200 kPa and 400 kPa. The interpreted trendline from Figure 5.53 has been applied to all three of these graphs. Figure 5.54 is the plot of CSR against the number of cycles to failure for the tests with a $p_i'$ of 100 kPa. Four tests had a CSR of 0.5, one test had a CSR of 0.25 and one test had a CSR of 0.125 at this $p_i'$. The tests generally follow the interpreted trendline, with the exception of those that were mentioned previously, again showing that the number of cycles to failure increases as the CSR decreases. Figure 5.55 is the plot of CSR against the number of cycles to failure for the tests with a $p_i'$ of 200 kPa. Two tests had a CSR of 0.5, four tests had a CSR of 0.25 and three tests had a CSR of 0.125 at this $p_i'$. Again, other than the tests previously mentioned, and in this case CYC19 – OC, the tests generally follow the interpreted trendline. Finally, Figure 5.56 is the plot of CSR against the number of cycles to failure for the tests with a $p_i'$ of 400 kPa. Two tests had a CSR of 0.5, two tests had a CSR of 0.25 and three tests had a CSR of 0.125 at this $p_i'$. A similar behaviour is again seen in this graph showing the increase in the number of cycles to failure as the CSR decreases. As seen across all three graphs, the $v_i$ values are scattered randomly and the $p_i'$ values do not make a difference to the number of cycles to failure.
5.8.10  Cyclic Stress Ratio and Axial Strain

A $v:\ln p'$ graph displaying axial strain % and contours for tests with a CSR of 0.25 on the mineral sand tailings is presented in Figure 5.57. It was only worthwhile completing this plot for this particular CSR as the tests at this CSR had a reasonable number of cycles to failure, and so a range of axial strains, $\varepsilon_a\%$, could be plotted. The points at which the $\varepsilon_a$ reached 0.125% are indicated by red diamonds, orange diamonds indicate an $\varepsilon_a$ of 0.25%, aqua diamonds indicate an $\varepsilon_a$ of 0.5%, green diamonds indicate and $\varepsilon_a$ of 1%, pink diamonds indicate an $\varepsilon_a$ of 2%, and failure at an $\varepsilon_a$ of 5% is represented by purple diamonds. The contours for tests with a $p_t'$ of 400 kPa are shown in black and the contours for tests with a $p_t'$ of 200 kPa are shown in red. Yield points have also been added. It can be seen from the figure that all the contours are different for individual $p_t'$ values. The $\varepsilon_a\%$ increments are reached at higher $p'$ values for the tests with a $p_t'$ of 400 kPa than the tests with a $p_t'$ of 200 kPa, but overall, there is nothing really conclusive. There is no consistent variation in the yield points, but they are in roughly the same region for the different $p_t'$ values. The $v$ values are not having a consistent effect.

5.8.11  Cyclic Stress Ratio and Stiffness

The decay of stiffness during undrained cyclic loading for tests on the mineral sand tailings are presented in Figure 5.58. The stiffness is calculated as a secant between the peak and trough of each cycle. These plots are all based on external axial LVDT readings. The local axial LVDT readings have not been used as the data were found to be inaccurate. The armatures inside the LVDTs did not drop on the downward cycles because the LVDTs were always slightly tilted. This was due to the weight of the mounts and the LVDTs themselves. In order to work effectively these needed to
be perfectly vertical. The external readings were found to be accurate and are corrected for the flexibility of the apparatus.

The tests that failed in one cycle could not be plotted as there were not enough data points. The tests that had a cyclic amplitude \((+/- q)\) of 25 kPa are plotted in orange, tests with a cyclic amplitude of 50 kPa are plotted in blue, tests with a cyclic amplitude of 100 kPa are plotted in red, and tests with a cyclic amplitude of 200 kPa are plotted in green. The \(p_i'\) for each test has been labelled on the graph. Note that all the tests with a cyclic amplitude of 400 kPa failed in one cycle and are therefore not plotted on the graph. The variation in specific volume is indicated by the dashed type of the lines. The stiffnesses should be expected to depend on the density of the samples, but these results show that there is a weak trend with void ratio, as also seen in Li’s (2017) data in Figure 4.11. All of the tests with a \(p_i'\) of 100 kPa fail quickly, with the exception of the 25 kPa cyclic amplitude test. It was expected that the higher \(p_i'\) tests would have had higher initial stiffnesses, but this is not always the case in this graph.

The lubricated end platen tests (LEP1 and LEP2) have been plotted on the graph. It was expected that these tests would be much softer than the other tests because of the addition of grease and membrane discs to the platens and so this would affect the external LVDT data. However, it can be seen that the stiffnesses are only slightly lower for LEP1 than the corresponding test CYC16 which had no lubricated end platens. The same can be said for LEP2 and CYC20. Again it can be seen that the addition of lubricated end platens had no significant effect on the results, as was already mentioned in Section 5.8.7.

The tests that failed had high CSRs, and this can be seen in Figure 5.59. This graph is the same as the previous one, but tests have been colour-coded in terms of
CSR and not cyclic amplitude. Tests with a CSR of 0.5 are in purple, tests with a CSR of 0.25 are in pink and tests with a CSR of 0.125 are in blue. Most of the tests with a CSR of 0.25 are failing, with the exception of two tests. One of these tests is the oversesolidated test, and the other test is CYC17 which, as already discussed, was not likely to be on its NCL before cyclic loading commenced. The other tests that show a decay of stiffness also display a point where the stiffnesses rapidly decline. This was also seen in Section 4.2 for Li’s (2017) data in Figures 4.11 and 4.14, and these points were defined as ‘yield points’. These are discussed below. At approximately 20-50 MPa the stiffnesses drop rapidly as failure begins and this is at ~100 cycles. The tests with a CSR of 0.5 display rapid failure and some tests are failing on the first cycle. These tests all have low $p_i'$ values and the stiffnesses are low because the strains are large. Figure 5.60 is the same graph but the stiffnesses have been divided by $p_i'$. Normalisation for the stress level $p_i'$ might be expected to bring the stiffness plots closer together but not much difference can be seen between the two plots. There is again a weak trend with void ratio, and normalisation has not improved this.

The stiffnesses, yield points and contours for tests with a CSR of 0.25 have been plotted on a $v$:ln$p'$ graph in Figure 5.61. Again, it was only worthwhile completing this plot for this CSR as the tests at this CSR had a reasonable number of cycles to failure, and so a range of stiffnesses could be plotted. Stiffnesses of 150 MPa are indicated with a red triangle, stiffnesses of 100 MPa are shown with a green triangle, blue triangles indicate a stiffness of 50 MPa and stiffnesses of 20 MPa are indicated by orange triangles. It can be seen from the figure that for the tests with a $p_i'$ of 400 kPa the stiffness values are reached at higher $p'$ values than for the tests with a $p_i'$ of 200 kPa. Contours have been plotted for each value of stiffness for the tests at different
$p_i'$ values. These are all vertical meaning that the $p'$ does make a difference to the stiffness but the $v$ does not. This was also seen for the $\varepsilon_a\%$ previously in this chapter. The yield points have been added to this plot and contours have been drawn. From the two contours it can be seen that the yield points were at approximately 20 MPa but at different $p'$ values for each $p_i'$. Overall, the yield point depends on the $p_i'$, but not the $v_i$.

5.8.12 Cyclic Stress Ratio and State Parameter

The number of cycles to failure have been plotted against the $\Psi$ for all cyclic tests in Figure 5.62. The points plotted have been colour-coded in terms of CSR, with red points plotted for tests with a CSR of 0.125, blue points plotted for tests with a CSR of 0.25 and orange points plotted for tests with a CSR of 0.5. The effect of $\Psi$ was examined in Chapter 4 and Li’s (2017) cyclic test data were reanalysed. It was found that there was a slight trend in the data showing that as the $\Psi$ increased, the number of cycles to failure decreased. This was seen in the literature review in Figure 2.33 where the resistance to liquefaction generally decreased as the $\Psi$ increased. From the figure it can be seen that the only parameter having an effect on the results is the CSR, making the plot not particularly useful. There is no real trend in the data. The tests with a CSR of 0.25 were all more or less on their own NCLs and so the range of $\Psi$ values is narrow. It is interesting to note that CYC17 does not have a negative $\Psi$ given that it is thought to not be on its NCL. This graph is however an artefact of how the $\Psi$ is being calculated. An assumption has been made that all CSLs are parallel (for the block 1 samples) which might not be true.

A second $\Psi$ graph was plotted using only one CSL. Figure 5.63 gives the CSL from a regression through the end points of all monotonic tests. The $\Gamma$ value was taken
from Figure 5.24. As can be seen from the \( v : \ln p' \) graph, the CSL was not a credible choice as the monotonic tests are stopping a significant distance away from the CSL. Also, the tests that start underneath the CSL are moving in the wrong direction, and so the \( \Psi \) values are random, as can be seen in Figure 5.64. This plot does not seem to work well, and neither does the one in Figure 5.62. Ideas for future work would include tests looking at a wide range of \( \Psi \) values relative to their individual CSLs.

5.8.13 Key Findings from Cyclic Loading

From the cyclic loading testing it was again seen that block 2 samples seemed to behave fairly similarly to the block 1 samples although typically there were more cycles to failure for block 2 samples. Also, the addition of lubricated end platens had no significant effect on the results. As expected, the overconsolidated sample (CYC19) had much more resistance to cyclic loading than similar tests that had not been overconsolidated.

From examining the CSR and the number of cycles to failure, there was no clear increase in the number of cycles as the \( v_i \) reduced. The \( v_i \) values were scattered randomly. The \( p_i' \) values also did not make a difference to the number of cycles to failure. The contours for \( \varepsilon_a \% \) were vertical on the \( v : \ln p' \) graph for individual \( p_i' \) values. The \( p_i' \) did make a difference to the \( \varepsilon_a \% \) at a particular \( p' \), but the \( v \) did not. The yield points for stiffness were also dependent upon \( p_i' \) but not \( v_i \). The \( \Psi \) plots were not particularly useful as no real trends were seen because of the similar \( \Psi \) values. The CSR was the only parameter having an effect on the results in the plots.
5.9 Summary

Oedometer tests were carried out on the mineral sand tailings and these concluded that the material was highly transitional. In comparison, the triaxial tests showed a greater change in $m$ values and at lower stress levels indicating more convergence. From Figure 5.24 it was found that the two different sample preparation methods used for triaxial testing did not make a significant difference to the mechanical behaviour of the tailings. Compression and shearing tests were carried out on two different blocks of the same tailings. Block 2 was found to exhibit a similar behaviour to block 1 in the $\nu$:ln$p'$ plane and so the majority of testing was carried out on block 1. Both loose and dense samples compressed on shearing but did not converge to a unique CSL again indicating strong transitional behaviour. The CSLs were also straight, even at low $p'$ values. It would seem that the addition of lubricated end platens had no significant effect on the results.

From the cyclic loading results, it appears that the $\nu_l$ does not make a significant difference to the number of cycles to failure in undrained cycling for this material and this is a key feature of this transitional soil. Generally, the $p_l'$ did not make a large difference to the number of cycles to failure, but the $p'$ did make a difference to the $\varepsilon_{\%a}$ and stiffness response. The $\varepsilon_{\%a}$ and stiffness responses were again not significantly affected by the $\nu$.

From Figure 5.64, for the samples tested with a CSR of 0.5 with a conventional analysis using a single CSL, a wide range of $\Psi$ values made no difference to the behaviour. This was, however, not the preferred interpretation as there is good evidence that there was no unique CSL for the mineral sand tailings. This was also the same for the tests with a CSR of 0.125. There was no significant difference to the generation of pore pressures. For a conventional non-transitional soil, a difference in
the results would have been expected. This was seen in the literature review in Figure 2.33 where the resistance to liquefaction generally decreased as the $\Psi$ increased. However, there is no trend seen in the results for the mineral sand tailings.

From looking at the results it might seem unusual that CSRs of 0.5 and 0.125 were selected for testing when the responses were ~1 cycle to failure and infinite cycles to failure respectively. However, identifying that tests results are similar after testing a wide range of $p_i'$ and $\nu_i$ values is valuable. These factors made no difference to the results and this is important. It might have been expected to see a wide range of behaviours with the large differences in $\nu_i$ values but this was not the case.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Drained/undrained</th>
<th>Preparation method</th>
<th>$v_i$</th>
<th>$p_i'$ (kPa)</th>
<th>CSR</th>
<th>NC or OC</th>
<th>$\varepsilon_v$% after IC</th>
<th>No. cycs to failure</th>
</tr>
</thead>
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<td>NC</td>
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<td>NC</td>
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Table 5.1 - A summary of the triaxial tests on the mineral sand tailings. Cyc = cyclic test, LEP = lubricated end platen cyclic test, MT = monotonic test, $v_i$ = initial specific volume, $p_i'$ = initial mean effective stress, CSR = cyclic stress ratio, NC = normally consolidated sample, OC = overconsolidated sample, $\varepsilon_v$% after IC = volumetric strain % after isotropic compression, $\infty$ = infinite cycle test.
Table 5.2 - A summary of oedometer tests on the mineral sand tailings from Australia ($\sigma'_{v_{\text{max}}}$ = maximum vertical effective stress).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$v_i$</th>
<th>$\sigma'<em>{v</em>{\text{max}}}$ (kPa)</th>
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<td>4</td>
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Table 5.3 - A summary of oedometer tests on the iron tailings from Sweden.

<table>
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<th>Test No.</th>
<th>$v_i$</th>
<th>$\sigma'<em>{v</em>{\text{max}}}$ (kPa)</th>
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<td>3</td>
<td>2.122</td>
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</tbody>
</table>

Figure 5.1 - Grading curve of the mineral sand tailings from Perth, Australia.
Figure 5.2 - Cumulative frequency graph for aspect ratio of the mineral sand tailings.

Figure 5.3 - Grading curve of the iron tailings from Sweden.
Figure 5.4 - One-dimensional compression data for oedometer tests on the mineral sand tailings.

Figure 5.5 - Quantification of convergence of one-dimensional compression curves for oedometer tests on the mineral sand tailings.
Figure 5.6 - One-dimensional compression data for oedometer tests on the iron tailings from Sweden.

Figure 5.7 - Quantification of convergence of one-dimensional compression curves for oedometer tests on the iron tailings from Sweden.
Figure 5.8 - SEM images at different magnifications of: (a) a loose sample and (b) a dense sample from oedometer tests on the mineral sand tailings annotated by the Author.
Figure 5.9 - Isotropic compression data and shearing paths for three undrained and six drained monotonic tests on the mineral sand tailings.

Figure 5.10 - Isotropic compression data for twenty undrained cyclic tests on the mineral sand tailings.
Figure 5.11 - Isotropic compression data and shearing paths for monotonic tests, isotropic compression data for cyclic tests and oedometer compression data for tests on the mineral sand tailings.

Figure 5.12 - Isotropic compression data for Cyc8 and Cyc11, that were pastes directly placed on the platen and Mt2, that was a paste sample compressed in a consolidometer, all showing similar behaviour.
Figure 5.13 - Quantification of convergence of isotropic compression curves for triaxial tests on the mineral sand tailings.

Figure 5.14 - Quantification of convergence of one-dimensional compression curves for oedometer tests and isotropic compression curves for triaxial tests on the mineral sand tailings.
Figure 5.15 - Isotropic compression data, shearing paths and CSLs for three undrained and six drained monotonic tests on the mineral sand tailings.

Figure 5.16 - Volumetric strain plotted against time for monotonic tests 1 and 5 before shearing on the mineral sand tailings.
Figure 5.17 - Volumetric strain plotted against axial strain for all drained tests on the mineral sand tailings.

Figure 5.18 - Extrapolated curve for monotonic test 1 on mineral sand tailings.
Figure 5.19 - Deviator stress plotted against axial strain for drained and undrained monotonic tests on the mineral sand tailings.

Figure 5.20 - The stress-strain data for the mineral sand tailings under drained and undrained conditions.
Figure 5.21 - Extrapolated stress-strain curve for monotonic test 1 on the mineral sand tailings.

Figure 5.22 - Change in pore pressure plotted against axial strain for the undrained monotonic tests on the mineral sand tailings.
Figure 5.23 - Stress paths for all monotonic tests on the mineral sand tailings.

Figure 5.24 - Gamma plotted against the initial specific volume at 20 kPa vertical stress for all monotonic tests on the mineral sand tailings.
Figure 5.25 - Stress paths normalised by equivalent pressures on the NCL for all monotonic tests on the mineral sand tailings.

Figure 5.26 - Stress paths normalised by equivalent pressures on the NCL taking M into account for all monotonic tests on the mineral sand tailings.
Figure 5.27 - Stress paths normalised by equivalent pressures on the CSL for all monotonic tests on the mineral sand tailings.

Figure 5.28 - Stress dilatancy curves for the drained monotonic tests on the mineral sand tailings.
Figure 5.29 - Combined change in pore pressure plotted against number of cycles for all cyclic tests on the mineral sand tailings on a linear scale.

Figure 5.30 - Combined change in pore pressure plotted against number of cycles for all cyclic tests on the mineral sand tailings on a logarithmic scale.
Figure 5.31 - Combined change in pore pressure divided by the initial effective stress plotted against the number of cycles for all cyclic tests on mineral sand tailings.

Figure 5.32 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles for all cyclic tests with a CSR of 0.125 for the mineral sand tailings.
Figure 5.33 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles for all cyclic tests with a CSR of 0.25 for the mineral sand tailings.

Figure 5.34 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles for all cyclic tests with a CSR of 0.25 up to 200 cycles for the mineral sand tailings.
Figure 5.35 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles for all cyclic tests with a CSR of 0.5 for the mineral sand tailings.

Figure 5.36 - Combined axial strain plotted against the number of cycles for all cyclic tests on the mineral sand tailings on a linear scale.
Figure 5.37 - Combined axial strain plotted against the number of cycles for all cyclic tests on the mineral sand tailings on a logarithmic scale.

Figure 5.38 - Axial strain plotted against the number of cycles for all cyclic tests with a CSR of 0.125 for the mineral sand tailings.
Figure 5.39 - Axial strain plotted against the number of cycles for all cyclic tests with a CSR of 0.25 for the mineral sand tailings.

Figure 5.40 - Axial strain plotted against the number of cycles for all cyclic tests with a CSR of 0.25 up to 200 cycles for the mineral sand tailings.
Figure 5.41 - Axial strain plotted against the number of cycles for all cyclic tests with a CSR of 0.5 for the mineral sand tailings.
Figure 5.42 - Stress paths for (a) CYC7 which took one cycle to reach failure, (b) CYC10 which took 131 cycles to reach failure, and (c) CYC15 which did not fail after >2000 cycles (exaggerated scales in (b) and (c)).
Figure 5.43 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles with the mean values plotted in orange for CYC3 – BLK2 and CYC2 – BLK1.
Figure 5.44 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles with the mean values plotted in orange for (a) LEP1 and (b) CYC16.
Figure 5.45 - Stress paths for (a) CYC16, and (b) LEP1. Both stress paths did not reach a mean effective stress of zero.
Figure 5.46 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles with the mean values plotted in orange for (a) LEP2 and (b) CYC20.

Figure 5.47 - Change in pore pressure divided by the initial effective stress plotted against the number of cycles with the mean values plotted in orange for CYC19 - OC and CYC9.
Figure 5.48 - Isotropic compression and cyclic paths for all cyclic tests on the mineral sand tailings including the number of cycles to failure.

Figure 5.49 - Isotropic compression and cyclic paths for all cyclic tests on the mineral sand tailings, including the number of cycles to failure, colour-coded by CSR.
Figure 5.50 - Isotropic compression lines, cyclic paths and number of cycles to failure for all cyclic tests on the mineral sand tailings with a CSR of 0.125.

Figure 5.51 - Isotropic compression lines, cyclic paths and number of cycles to failure with contours for all cyclic tests on the mineral sand tailings with a CSR of 0.25.
Figure 5.52 - Isotropic compression lines, cyclic paths and number of cycles to failure for all cyclic tests on the mineral sand tailings with a CSR of 0.5.

Figure 5.53 - CSR plotted against the number of cycles to fail for all cyclic tests on the mineral sand tailings, colour-coded by the initial effective stress, with trendline added.
Figure 5.54 - CSR plotted against the number of cycles to fail for all cyclic tests on the mineral sand tailings with an initial effective stress of 100 kPa with trendline added.

Figure 5.55 - CSR plotted against the number of cycles to fail for all cyclic tests on the mineral sand tailings with an initial effective stress of 200 kPa with trendline added.
Figure 5.56 - CSR plotted against the number of cycles to fail for all cyclic tests on the mineral sand tailings with an initial effective stress of 400 kPa with trendline added.

Figure 5.57 - A $v:\ln p'$ graph displaying axial strain % and contours for tests with a CSR of 0.25 on the mineral sand tailings.
Figure 5.58 - Decay of stiffness during undrained cyclic loading for mineral sand tailings colour-coded by cyclic amplitude.

Figure 5.59 - Decay of stiffness during undrained cyclic loading for mineral sand tailings colour-coded by CSR.
Figure 5.60 - Decay of stiffness normalised for the initial effective stress during undrained cyclic loading for tests on the mineral sand tailings.

Figure 5.61 - A $\nu\ln p'$ graph displaying stiffnesses, yield points and contours for tests with a CSR of 0.25 on the mineral sand tailings.
Figure 5.62 - Number of cycles to failure plotted against the state parameter for cyclic tests on the mineral sand tailings.

Figure 5.63 - The individual CSL used to calculate state parameter values for the mineral sand tailings.
Figure 5.64 - Number of cycles to failure plotted against the state parameter for cyclic tests on the mineral sand tailings for one CSL.
6. Conclusions and Future Work

6.1 Introduction

After extracting the valuable minerals from the rock milling process, a significant amount of residual waste is left behind. This waste product is known as tailings. Tailings are an artificial material and the mechanical behaviour upon loading is different when compared with that of natural soils, although the behaviour is typically analysed with a traditional soil mechanics framework. It is essential to understand the mechanical behaviour of tailings materials so that tailings dams can be constructed and monitored safely. The purpose of this research was to identify the behaviour of more strongly transitional tailings, examining how this would influence its behaviour under both monotonic and undrained cyclic loading, for example, a seismic event.

Three materials were tested in this study. These were an iron tailings from the Wanniangou tailings dam in Panzihua City, China, a mineral sand tailings excavated from a mine about 100 km South of Perth, Australia, and an iron tailings from Sweden. Strong transitional behaviour was identified in the mineral sand tailings from Australia, and so the majority of the testing was carried out on this material for a thorough investigation.

In both isotropic and one-dimensional compression there was a clear non-convergence of compression paths from different initial states for the mineral sand tailings. More convergence was seen in isotropic compression than in one-dimensional compression, although a unique NCL could not be reached even at 400 kPa and monotonic drained shearing did not lead to unique critical states. As for other transitional soils the volume changes seen in compression and shearing are
significantly less different between loose and dense samples than they would be for conventional soils with unique NCLs and unique CSLs.

6.2 Iron Tailings from China

The work carried out by Li (2017) on the Iron tailings from the Wanniangou tailings dam in Panzihua City, China was reanalysed. The Author also carried out four cyclic tests on these tailings and the results were compared with Li’s. The Wanniangou tailings dam is located near residential areas and transportation infrastructure so it was important that the material was tested to analyse the liquefaction potential. Li excavated the material from three different locations: upper beach (UB), middle beach (MB), and pond (PO). This was to ensure a complete investigation of the behaviour of the tailings, as the particle size differed at each location.

Li carried out undrained cyclic loading tests on UB, MB and PO samples and from plotting his data it was found that the denser samples took more cycles to reach cyclic mobility than the looser samples. The axial strain contours, $\varepsilon_a\%$, were difficult to evaluate because of the limited range of $p'$ and $\nu$, but within the limited range the axial strains were almost the same. The differences in $\nu$ only had a small effect. The number of cycles to failure were plotted against $p'$ values and contours indicated that at a given $p'$ value the number of cycles to failure were greater for the denser samples. It was interesting that unique contours could be drawn and that these were similar for all three materials. These contours may have worked well because the CSLs were not so different for the three materials. This similarity could mean that the mechanical behaviour of the three materials might be alike.
The graph of axial strains was normalised for the different critical state line locations and contours were plotted for the 60 kPa and 120 kPa amplitude tests. The contours found in the $vn:\ln p'$ plane for axial strain and stiffness were slightly inclined. The factors that were most important here were the $p'$ and the amplitude because the specific volume only had a small effect.

The stiffness plots showed that there was a weak trend with void ratio or specific volume and that the different locations (UB, MB, PO) did not show any real significance in terms of the decay of stiffness. The stiffnesses plummeted after reaching the yield points. Vertical contours could be found through yield points on the $vn:\ln p'$ graph. Contours were also added through selected stiffness points which were parallel to one another and were close to being vertical. These vertical contours indicated that the controlling parameters were $p'$ and the amplitude but the volume was unimportant. The higher amplitude tests reached the yield points at a higher $p'$ values than the lower amplitude tests.

Li’s tests were also reanalysed in terms of state pressure index, $I_p$, state parameter, $\Psi$, and modified state parameter, $\Psi_m$. The data were plotted from eight of Li’s tests on cyclic loading; four tests for the tailings from the upper beach, two tests for the tailings from the middle beach and two tests the tailings from the pond. The number of cycles to failure were plotted against the $I_p$ and there was no real trend in the data, as tests could have the same $I_p$ value but very different numbers of cycles to failure. There was a slight trend in the number of cycles to failure plotted against the $\Psi$ graph that displayed that as the $\Psi$ increased, the number of cycles to failure decreased. This, however, was not the case for all tests. There was a significant difference between the tailings found at the different locations, and inconsistencies even for the tailings at one location. The data were then plotted to display the number
of cycles to failure against the $\Psi_m$. There was more spread in the data here than in the $\Psi$ plot and no clear trend was visible. The scatter increased for each individual soil location and there was no agreement between different soil locations.

Tests were carried out by the Author on the Panzihua tailings to replicate simplistically the layering in tailings dams. The layering is lost in reconstituted samples, so it was important to analyse the behaviour of this simple form of structure. Four successful cyclic tests on different samples were carried out in total; two on mixed slurry samples and two on layered samples. All tests had a CSR of 0.25. The specific volume values for mixed sample 1 and mixed sample 2 were similar, but the pore pressure response graphs, axial strain graphs and stress paths were significantly different. This was because they had different gradings. The second set of samples had a much higher fines content than the first set of samples. Layered sample 1 had a similar specific volume to the mixed sample and quite a similar pore pressure response to mixed sample 1. However, the grading in the two layers were too similar to conclude that layering had no effect. Layered sample 2 had the highest specific volume. Despite having a high specific volume, the layering did not have a big effect on the pore pressure response and the sample did not fail. This may therefore mean that layering may not have a significant effect on the response.

All tests mentioned above were then compared. The decay of stiffness was plotted for all tests and the addition of the Author’s tests confirmed the weak trend with specific volume and normalisation for $p_i'$ did not improve this. The stiffnesses for the mixed and layered sample tests were quite similar to the stiffnesses from the UB, MB and PO tests. A comparison was then made between the Author’s tests on mixed sample 2 and layered sample 2 and Li’s MB tests which had a similar grading. Test MB1 took 9 cycles to reach failure, whilst test MB2 took 18 cycles. Both mixed sample
2 and layered sample 2 were not close to reaching cyclic mobility. The layered sample had a higher specific volume than the mixed sample, but even though there was this difference, the behaviour tended to be quite similar. Mixed sample 1 and layered sample 1 had a grading closer to UB, and so a comparison was made between these sets of tests. The UB samples took considerably fewer cycles to reach failure than mixed sample 1 and layered sample 1. This was because the mixed sample test had a lower cyclic amplitude and a slightly different grading. Despite having a higher specific volume, the layering did not have a big effect on the pore pressure response, although the layered sample built up pore pressures slightly less rapidly than the mixed sample, even though it had a higher specific volume. In-situ samples will be layered because of the way they are deposited. Retrieving intact samples is very difficult and so these tests show that testing reconstituted samples should be conservative. Layered tests therefore liquefy less easily than mixed samples and so the effects of layering should not be a concern.

6.3 Mineral Sand Tailings

The tailings chosen for the majority of the testing in this study was a mineral sand tailings from about 100 km South of Perth, Australia as it was found to be highly transitional. The triaxial tests showed a greater change in $m$ values and at lower stress levels indicating more convergence than the oedometer tests.

A number of compression and shearing tests were carried out on the tailings to characterise the behaviour. Two different blocks were recovered. Block 2 was found to exhibit a similar behaviour to block 1 in its behaviour in the $\nu$:ln$p'$ plane from monotonic testing, but the majority of testing was carried out on block 1. From the
cyclic loading testing it was again seen that block 2 samples seemed to behave fairly similarly to the block 1 samples although typically there were slightly more cycles to failure for block 2 samples. The grading of each block was very similar. Also, the addition of lubricated end platens had no significant effect on the results and, as expected, the overconsolidated sample (CYC19) had much more resistance to cyclic loading than similar tests that had not been overconsolidated.

From the cyclic loading results, it would seem that the $v_i$ does not make a significant difference to the number of cycles to failure in undrained cycling for this material and this is a key feature of this transitional soil. Generally, the $p_i'$ did not make a large difference to the number of cycles to failure, but the $p'$ did make a difference to the $\varepsilon_{a} \%$ and stiffness response. The $\varepsilon_{a} \%$ and stiffness responses were again not significantly affected by the $v$.

With a conventional analysis using a single CSL, a wide range of $\Psi$ values made no clear difference to the behaviour. This was, however, not the preferred interpretation as there was no unique CSL for the mineral sand tailings. For a conventional non-transitional soil, a difference in the results would have been expected with the number of cycles reducing as the $\Psi$ increased, as was seen in the literature review, however, there is no clear trend seen in the results for the mineral sand tailings.

The tests with a CSR of 0.125 were all infinite cycle tests and the tests with a CSR of 0.5 all failed within a few cycles. Identifying that the test results were still similar despite testing such a wide range of $p_i'$ and $v_i$ values was valuable. It might have been expected to see a wide range of behaviours with differing $v_i$ values but this was not the case and this is important.
The literature review highlighted the need to investigate fabric effects through different sample preparation methods and how the method of sample preparation affects the nature and characteristics of the cyclic response of the soil. For the mineral sand tailings, it was found that for reconstituted samples made at different initial densities using two different sample preparations methods, no unique CSL could be plotted on a $v$:$\ln p'$ graph. Samples were either made in the consolidometer or were made directly on the platen. Both loose and dense samples compressed on shearing but did not converge to a unique CSL indicating strong transitional behaviour. It was found that the preparation methods, within the ones chosen for this research, made little difference to the mechanical behaviour of the tailings.

There was considerable evidence of transitional behaviour in some soils in the literature, but how transitional behaviour affects the susceptibility to cyclic liquefaction was absent. There was also little literature on the cyclic testing of transitional soils let alone the cyclic testing of transitional tailings. For this transitional tailings, the tests presented in this thesis show that the initial density does not make a significant difference to the mechanical behaviour in undrained cycling and initial density was found to be much less important than the cyclic stress ratio, CSR, and the initial stress level, $p_i'$.

The literature review also indicated that there is currently no overall framework for curved CSLs for undrained cyclic loading. However, for this mineral sand tailings, the CSLs were straight in the $v$:$\ln p'$ plane, even at low $p'$ values, showing a similar behaviour to clays in that respect.
6.4 Tailings Comparison

The reanalysis of the work done by Li (2017) on the iron tailings from the Wanniangou tailings dam in China showed inclined contours in the $\nu:\ln p'$ and normalised $\nu_n:\ln p'$ planes for the current number of cycles to failure. This indicated that both $p'$ and $\nu$ affected the number of cycles to failure. In comparison, the contours in the $\nu:\ln p'$ plane for the current number of cycles to failure were vertical for the mineral sand tailings so only $p'$ was having a significant effect and not $\nu$. The $p_i'$ values also did not make a difference to the number of cycles to failure.

Similar contours found in the $\nu_n:\ln p'$ plane for axial strain and stiffness were slightly inclined for the iron tailings from China. The $p'$ and the amplitude were affecting the results whilst the $\nu$ only had a small effect. For the mineral sand tailings, the contours for axial strain and stiffness were again almost vertical but were also dependent upon the $p_i'$.

The mechanical behaviour of the Panzihua iron tailings seemed to be similar across the three localities examined (UB, MB, PO). The location was not found to be important and neither was the $\nu$. The parameters that seemed to be the most important were $p'$ and cyclic amplitude and therefore CSR, which was similar to the findings for the mineral sand tailings.

For the mineral sand tailings, the number of cycles to failure plotted against the $\Psi$ graphs were not particularly useful as no real trends were seen because the range of $\Psi$ values were too small. The CSR was the only parameter having an effect on these results. There was a slight trend displayed in Li’s cyclic test data showing that as the $\Psi$ increased, the number of cycles to failure decreased. This was, however, not the case for all of the tests. There was a significant difference between the tailings found
at the different locations, and inconsistencies even for the tailings at one location. The
data were also plotted to display the number of cycles to failure against the $\Psi_m$. There
was more spread in the data here than in the $\Psi$ plot and no clear trend was visible.
Carrera (2008) examined the static liquefaction of fluorite tailings taken from the
Stava, Italy collapse in 1985. Carrera carried out a number of monotonic triaxial tests
on the sand and silt mixed at different percentages. These data were reanalysed in
terms of $I_B$ against $\Psi$ and no clear trend was visible. The $I_B$ was also plotted against
$\Psi_m$ and much better agreement was seen in this graph than in the $\Psi$ one, but there was
no agreement between the different soils. Overall, there seemed to be little use in using
the $\Psi_m$ as a prediction tool for both static and cyclic liquefaction when all the plots are
different for different soils.

6.5 Practical Implications

Many tailings are hydraulically placed in loose states. If a tailings is transitional, and
compression under the overburden pressure will not cause it to reach a unique NCL,
it might have been a concern that the initial placement density would have a significant
impact on subsequent behaviour. This turns out not to be the case for the mineral sand
tailings, and in compression the paths converge only slowly so the amount of
compression experienced will be not so different for different initial densities. In
shearing and cyclic loading, the difference in behaviour for different initial densities
is minor, while for a conventional soil there could be a large difference. It may
therefore not be so necessary to be concerned about the placement density for
transitional tailings. If the tailings had conventional behaviour the placement density
could be very important.
There is nothing about the grading shape or mineralogy that allows us to distinguish why this tailings is transitional. Previous works (Ponzoni et al., 2014; Todisco and Coop, 2019) have also failed to identify when and why transitional behaviour is seen in other soils. To identify whether the soil is transitional or not it must be tested and as yet there is no way to anticipate transitional behaviour.

6.6 Future Work

In studying the cyclic liquefaction behaviour of the mineral sand tailings at three different CSRs of 0.125, 0.25 and 0.5, it was found that in retrospect the selection of 0.5 was probably too high and 0.125 was probably too low, although it had been expected that the wide variation of densities would have caused different behaviours to be seen. Future work could be carried out on tests with a CSR of 0.4 and 0.2 for example.

The SEM images of the loose and dense samples for the mineral sand tailings were found to be similar. There was a little more particle orientation in the denser sample, but there is a further need to analyse the fabric to make firm conclusions. MIP (mercury intrusion porosimetry) testing will be carried out in Paris in the coming months which will examine the differences in the fabric at different initial densities.

The \( \Psi \) plots were not particularly useful for the mineral sand tailings as no real trends were seen because of the similar \( \Psi \) values. It would therefore be worthwhile examining a wide range of \( \Psi \) values relative to their individual CSLs.

Further testing could also be carried out on the iron tailings from China to examine denser layered samples, with a similar specific volume to mixed sample 2, to confirm whether or not layering does have an effect on the cyclic response. The
Swedish iron tailings could also be tested further to confirm the transitional behaviour of the tailings.

More general future work could include testing other transitional soils or tailings to see how they compare with the materials examined in this thesis. Other aspects of transitional tailings could also be investigated. Finally, intact samples of transitional tailings could also be tested to confirm whether or not sample preparation methods have an impact on results.
References


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Appendix

The basic data for all of the cyclic tests on the mineral sand tailings from Australia have been compiled in this appendix.

(a)

(b)
Figure A.1 - CYC1 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.2 - CYC2 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.3 - CYC3 (Block 2) graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.4 - CYC4 (Block 2) graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.5 - CYC5 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.6 - CYC6 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.7 - CYC7 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.8 - CYC8 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.9 - CYC9 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.10 - CYC10 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.11 - CYC11 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.12 - CYC12 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.13 - CYC13 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.14 - CYC14 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles.
Figure A.15 - CYC15 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.16 - CYC16 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.17 - CYC17 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.18 - CYC18 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.19 - CYC19 (OC) graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.20 - CYC20 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A.21 – LEP1 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.
Figure A. 22 - LEP2 graphs on the mineral sand tailings displaying (a) stress path, (b) displacement plotted against the number of cycles with mean values plotted in orange, and (c) change in pore pressure divided by the initial effective stress plotted against the number of cycles with mean values plotted in orange.