Geotechnical study of engineering behaviour of fouled ballast

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Department of
Civil, Mining and Environmental Engineering

Geotechnical Study of Engineering Behaviour of Fouled Ballast

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BSc (Civil Eng) Hons

This thesis is presented as part of the requirement for the
Award of the Degree of Doctor of Philosophy
of the
University of Wollongong

October 2012
CERTIFICATION

I, Nayoma Chulani Tennakoon, declare that this thesis, submitted in fulfillment of the requirements for the award of Doctor of Philosophy, in the School of Civil, Mining and Environmental Engineering, University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. The document has not been submitted for qualifications at any other academic institution.

The following publications are related to the research work conducted in this study:


-------------------------------------
Nayoma Tennakoon
ABSTRACT

Railways are among the fastest and most economic transport modes in Australia, and the improvements in track performance are the direct results of the increase in the volume of rail traffic, as well as the need for reducing the cost of rail maintenance. Ballast is the uniformly graded coarse aggregate placed underneath the sleepers whose main purpose is to facilitate rapid drainage and provide structural support for the heavy loads exerted by the passage of trains.

When the ballast voids are wholly or partially filled with the intrusion of fine materials, particle breakage and pumping of soft subgrade soil, the track can be considered as being “fouled”. In Queensland, on the average, 20% decrease in load carrying capacity is due to ballast fouling. The reduced void space in ballast significantly affects its hydraulic conductivity by reducing the drainage capacity of the track. In order to ensure acceptable track performance and longevity, it is pertinent to maintain rapid drainage conditions within the ballasted bed. However, fouling reduces the drainage capacity of the ballast, excess pore water pressure can be generated under the passage of fast moving trains (cyclic load), which further compromises track resiliency while contributing to increased maintenance costs. In addition, fouling causes differential settlement of the track and also decreases its load bearing capacity due to the reduction in internal friction of the granular assembly.

The design aspects and maintenance cost of ballasted tracks can be significantly reduced if an accurate estimation of different types of fouling material and associated degradation mechanisms can be properly quantified. To maintain the required track geometry with fouled ballast, frequent maintenance activities should be carried out. However the decision for practicing engineers of where and when to perform the
maintenance operations due to lack of adequate information of fouled ballast condition is often difficult to make. Therefore, it is important to understand both the mechanical and hydro-mechanical characteristics of fouled ballast for different proportions of fouling, because, this will significantly assist towards optimising the time frame for maintenance while effectively minimising the maintenance costs.

A series of large scale hydraulic conductivity tests with specimen size of 500 mm x 500 mm high, were conducted with different proportions of fouling to study the relationship between the extent of fouling and hydraulic conductivity. Since the hydraulic conductivity obtained from laboratory experiments were one-dimensional given that two-dimensional flow conditions may prevail in reality, a numerical analysis was conducted using SeepW (2007a) to quantify the drainage capacity of ballast under different degrees of fouling. Subsequently, a quantitative classification for drainage in relation to the degree of fouling, which is very useful tool for practical engineers, is presented in this thesis. The analysis showed that both the location and extent of fouling played an important role when assessing the overall drainage capacity of track. Based on the research outcomes, ballast cleaning using the undercutting method is recommended when the Void Contaminant Index (VCI) of the top 100mm of the ballast layer exceeds 50%. When the shoulder ballast is fouled by more than 50% VCI, then it should be either cleaned or replaced to maintain acceptable drainage. This is because, if shoulder ballast is fouled to a high level (VCI > 50 %) then ‘poor drainage’ of the track can occur even if the other ballast layers are relatively clean.

In order to establish the relationship between the extent of fouling and the associated strength-deformation properties, a series of large scale (300 mm diameter by 600 mm
height) monotonic triaxial tests were carried out for different levels of fouling for confining pressures in the range of 10-60 kPa. Based on the laboratory findings, a novel empirical relationship between the peak deviator stress and VCI has been proposed to assist the practitioner in their preliminary track condition assessment. At a significantly high level of fouling (VCI > 50%), a considerable drop in shear strength can affect the stability and load carrying capacity of the track.

A series of large scale cyclic drained triaxial tests has been conducted to investigate the effects of fouling on settlement and ballast degradation, along with a number of loading cycles, simulating realistic field loading conditions. The experimental investigation showed that an increase in fouling always increases the permanent axial strain at a given number of cycles, while an increase in fouling lowered the compressive strain at final N > 500,000 cycles. This suggests that at excessive level of fouling the dilation of the ballast layer is initiated as the number of load cycles increases. Not surprisingly, the corresponding magnitude of resilient strain was shown to decrease with an increasing degree of fouling. Apart from the detrimental effects discussed earlier, fouling leads to a reduction in particle breakage.

A constitutive model for clay fouled ballast is formulated using bounding surface framework under monotonic loading and drained condition. The model is validated with the large scale triaxial experiments carried out in this research.
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1 INTRODUCTION

1.1 General Background

Railways offer an efficient and economic mode of transportation in Australia and in many countries around the world. Railways in Australia play a crucial role in its economy by conveying freight and bulk commodities between major cities and ports, and carrying passengers, albeit predominantly in urban areas. There are more than 43,000 km of broad, narrow, standard, and dual gauge rail tracks in Australia (Salim, 2004) (Figure 1.1). In recent years, however, road transport has become more competitive in relation to rail and other modes of transportation, mainly due to improvements in road infrastructure and decreasing costs of road transportation. These trends have increased the pressure on railway industries to improve their efficiency by limiting the expense of maintenance and infrastructure.

For several reasons, conventional rail tracks are positioned on a coarse granular medium (i.e., ballast); these include economy (availability and abundance), rapid drainage, and high load bearing capacity. Rail ballast usually contains uniformly graded material (10-60mm in size) which creates a sufficiently large pore structure to facilitate rapid (free) drainage. To sustain good performance and long service life, it is essential to maintain rapid drainage in a ballasted track at all times (Selig and Waters, 1994).

During track operation, fine particles can accumulate within the ballast voids due to: (a) breakage of angular corners and sharp edges, (b) infiltration of fines from the...
surface, and (c) subgrade soil pumping under conditions of excessive cyclic loads and saturated soil. Fouling of ballast makes it effectively less angular, and decreases its overall shear strength. Excessive fouling also generates excess pore water pressure under fast moving trains (i.e., high cyclic loading), which reduces the resiliency and stability (Indraratna et al., 2010).

Figure 1.1. Australia’s rail network (adopted from Salim, 2004)
1.2 Statement of the Problem

Fouling of ballast is one of the major reasons behind the deterioration of track geometry. Sources of fouling can be attributed to particle degradation, infiltration of fine foreign particles from the surface, sleeper wear, as well as sub-ballast and subgrade infiltration (Indraratna et al., 2011). There are two common types of fouling that can be seen in Australia, namely, coal fouling (surface infiltration, Figure 1a) due to spilling of coal from wagons, and clay pumping due to soft, unstable subgrade. In low-lying coastal areas where the subgrades are usually saturated, the fines (clays and silt-size particles) of subgrade can be pumped up into the ballast layer as slurry under cyclic loading, when trains operate in the absence of a proper subbase or filter layer (Indraratna et al., 2002b; Selig and Waters, 1994). The pumping of subgrade clay is a major cause of ballast fouling (Figure 1.2-a).

Coal has been a major source of energy supply in Australia and other countries such as the United States. As the demand for coal increases with the growing energy needs, the transportation of coal in Australia relies strongly on rail transport. As the coal is transported, some drops onto the track from the overfilled wagons and infiltrates into the ballast layer under subsequent cyclic loading by trains. Coal fouling can most frequently be observed in the states of Queensland and New South Wales in Australia (Figure 1.2-b).
Figure 1.2: Ballast fouling due to (a) clay pumping in New South Wales, Australia and (b) coal fouling in Queensland, Australia

It is believed that fine fouling materials may lubricate the ballast and reduce its internal frictional angle. In addition, as a result of the progressive entrapment of fines, the reduced drainage capacity of the track can generate excessive pore water pressure and thereby reduce the effective bearing capacity of the track. Ballast fouling usually increases irrecoverable deformations and may lead to differential track settlement, which in turn leads to a subsequent increase in the number of maintenance cycles of the rail track. Figure 1.3 shows typical problems of ballast fouling caused by poor drainage and differential track settlement.
Very limited research has been conducted on the behaviour of fouled ballast, even though worldwide, the level of funds invested in track maintenance is substantial. For instance, the annual cost of removing fouled ballast has exceeded $15 million in the state of New South Wales (Australia) alone since mid-2000s, and in excess of 300,000 tonnes of ballast is used to replace the fouled ballast (Indraratna et al., 2011). These maintenance costs and concomitant effort can be significantly reduced if an accurate estimation of track deterioration caused by the fouling material can be predicted. By adopting an effective maintenance program, the quarrying of ballast can be reduced with favourable environmental implications and improved productivity.
1.3 Objectives of the Research

Research on track capacity under fouled conditions would be an important step towards reducing the cost and optimising the time required for effective track maintenance. A critical amount of fouling should be identified to evaluate the service life of ballast and prevent an unacceptable geometry. The primary objective of this research study is to investigate the strength, deformation, and degradation of ballast under monotonic and cyclic loadings for different degrees of clay fouling. The secondary objective is to study the drainage capacity of the track under different degrees of fouling.

The specific objectives of this research are as follows:

1. Quantification of ballast fouling in terms of available fouling indices and introduction of a new fouling index.

2. Laboratory investigations on the hydraulic conductivity of fresh and fouled ballast using a large scale permeability chamber, and an assessment of the drainage condition of the track using two-dimensional numerical modelling.

3. Laboratory investigation of stress-strain and degradation response of fresh and fouled ballast under monotonic (static) loading at various confining pressures using the large scale triaxial apparatus. The results of this study will provide a direct comparison between the strength, deformation, and degradation characteristics of fresh and fouled ballast.
4. Study the deformation and degradation of fresh and fouled ballast under cyclic loading. The results of the cyclic test will reveal the performance of fouled ballast compared to fresh ballast under realistic track loading.

5. Development of constitutive model for clay fouled ballast. The constitutive model will enable rail engineers to predict deformations of clay fouled ballast for a given load magnitude and degree of clay fouling, so that an appropriate and economical maintenance scheme can be implemented.

1.4 Scope and Limitations of the Research

Within the scope of this thesis, only one gradation of ballast, satisfying the specification of AS 2758.7 (1996) together with clay fouling, has been considered. A monotonic loading test program was conducted under three possible confining pressures (10 kPa, 30 kPa and 60 kPa) to represent the field conditions. Cyclic load testing was conducted under a typical confining pressure which is more likely to exist in track (i.e. @ 10 kPa). A triaxial cyclic loading program was conducted at a frequency of 20 Hz (144 km/hr train speed), subjected to a maximum and minimum cyclic loading of 230 kPa and 45 kPa, respectively, representing a 25 tonne axle load.
1.5 Outline of the Thesis

This thesis is divided into 7 chapters, including the Introduction. A summary of the remaining chapters is given below:

In Chapter 2, the current state of research on the behaviour of ballast and ballast fouling is critically reviewed after presenting a brief overview of different components of rail track structure, types of loadings, and track deterioration issues. Four key aspects of this research, cyclic loading, static loading, and degradation and drainage, were given an equally important emphasis in the discussion.

Chapter 3 describes the new fouling index-Void Contaminant Index (VCI), and its advantages compared to other fouling indices available in the literature. This chapter also describes the procedure for determining the VCI in the field, for the convenience of practicing engineers.

Chapter 4 presents the drainage characterisation of fouled ballast. This includes a laboratory test program, and the associated results and discussions on the hydraulic conductivity of fouled ballast. An analytical model to predict hydraulic conductivity is also presented. Further, a two-dimensional numerical model has been implemented in the SEEP/W finite element software package to simulate more realistic track conditions. A classification of drainage conditions is introduced to classify drainage according to the degree and pattern of fouling.

Chapter 5 presents the behaviour of drained clay fouled ballast under monotonic loading. The experimental findings and discussions on the current test results,
including variations in the deviator stress, volumetric response, peak deviator stress, friction angle, Mohr-Coulomb shear strength, and particle breakage under different confining pressures are also discussed. Moreover, this chapter describes the non-linear strength envelope and a novel empirical relationship to capture the detrimental effects of clay fouling on the performance of ballasted tracks. Finally, the critical state parameters which are essential for constitutive modelling of clay-fouled ballast are presented and discussed.

In Chapter 6, the behaviour of clay fouled ballast under cyclic loading is discussed based on laboratory investigations. This includes variations in deformation, resilient modulus and particle degradation, for varying degrees of fouling.

In Chapter 7, a new constitutive model for fresh and clay fouled ballast based on the bounding surface concept is presented. The constitutive model includes shearing under monotonic loading from both isotropic and anisotropic initial stress states.

Conclusions and Recommendations from the current study are given in Chapter 8. It summarises the findings of this research study dealing with the mechanical and hydro-mechanical aspects of fouled ballast.

A list of References and Appendices follow in Chapter 8.
2 LITERATURE REVIEW

2.1 Introduction

Ballast is one of the most important components in a rail track. Ballast is the crushed granular load bearing material which supports the stresses induced by the passage of trains, through the rail and sleeper assembly. The clay fines resulting from subgrade pumping foul and degrade the ballast, directly contributing to track settlement. Understanding the behaviour of fouled ballast during monotonic and cyclic loading plays a key role in reducing the maintenance costs of railway track, while optimising passenger comfort. Also the understanding of drainage aspects of fouled ballast is important as poor drainage directly affects to the service life of the track. Many research studies have been conducted in the past to understand the complex behaviour of ballast under monotonic and cyclic loading.

This chapter presents a literature review related to the deformation, degradation mechanisms and drainage characteristics of clean and fouled ballast, and their serious implications on track stability. The chapter is divided into six main sections: section 2.2 presents ballasted track components, section 2.3 presents track forces, and section 2.4 discusses the factors affecting the behaviour of fresh ballast. Section 2.5 discusses the prevailing substructure problem including a general literature review on ballast fouling and their effects on ballasted track. Section 2.6 discusses existing track maintenance technique followed by prediction of ballast service life in section 2.7.
2.2 Ballasted Track Components

Rail track is comprised of superstructure and substructure. Superstructure consists of rail, sleepers, and the fastening system, whereas substructure consists of ballast, sub-ballast (capping) and subgrade (Figure 2.1). The superstructure is separated from the substructure by the sleeper-ballast interface.

Rails are the longitudinal steel members which are in constant contact with the wheels. The connection between the rail and the crosstie is called the fastening system, whose sole purpose is to hold the rails against the sleepers and resist vertical, lateral, longitudinal, and overturning movements of the rail (Selig and Waters 1994). Sleepers are located at the interface of the ballast. The main functions of sleepers are to distribute the wheel loads transferred by the rails and fastening system to the supporting ballast.

Figure 2.1: Typical section of a ballasted rail track (modified after Selig and Waters, 1994)
2.2.1 Ballast

Ballast is usually composed of medium to coarse gravel sized particles (10-60mm). The source of ballast (parent rock) varies from country to country, depending on the quality and availability of the rock, and the economy. Common ballast materials include Rheolite, dolomite, gneiss, granite, basalt, limestone, and blast furnace slag (Lackenby 2006). Latite Basalt is a common type of ballast in New South Wales, Australia. The main functions of ballast are (Indraratna et al., 2011):

- To provide a firm and stable platform, and support the sleeper uniformly with high bearing capacity
- Transmit the high imposed pressure at the sleeper/ballast interface to the layer of subgrade at a reduced and acceptable level of stress
- Provide adequate stability to the sleepers against the vertical, longitudinal, and lateral forces generated by train movements within the designed speed limits
- Provide dynamic resiliency
- Provide adequate resistance against crushing, attrition, bio-chemical and mechanical degradation, and weathering
- Provide minimal plastic deformation to the track structure
- Provide adequate hydraulic conductivity for drainage purposes
- Facilitate maintenance operations
- Inhibit the growth of weeds
- Absorb noise, and
- Provide adequate electrical resistance.
2.2.1.1 Properties of ballast

To fulfil the above functions, ballast should have certain characteristics such as particle size, shape, gradation, surface roughness, particle density, bulk density, strength, durability, hardness, toughness, resistance to attrition and weathering, as described in Australian Standard AS 2758.7 (1996) or specification TS 3402 (2001). Table 2.1 summarises the recommended physical criterion for ballast in Australia (Found and Martin, 2002; Indraratna et al., 1997; Standards Australia, 1996), Canada (Gaskin and Raymond, 1976; Raymond, 1985) and the USA (Gaskin and Raymond, 1976; Hay, 1953). Ballast particle sizes tend to be uniform in nature and Figure 2.2 shows the typical distributions currently being used.
Table 2.1: Ballast Specifications in Australia, USA and Canada (after Indraratna et al., 2011)

<table>
<thead>
<tr>
<th>Ballast property</th>
<th>Australia</th>
<th>USA</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate Crushing Value</td>
<td>&lt; 25%</td>
<td>....</td>
<td>....</td>
</tr>
<tr>
<td>LAA</td>
<td>&lt; 25%</td>
<td>&lt; 40%</td>
<td>&lt; 20%</td>
</tr>
<tr>
<td>Flakiness Index</td>
<td>&lt; 30%</td>
<td>....</td>
<td>....</td>
</tr>
<tr>
<td>Misshapen Particles</td>
<td>&lt; 30%</td>
<td>....</td>
<td>&lt; 25%</td>
</tr>
<tr>
<td>Sodium Sulphate Soundness</td>
<td>....</td>
<td>&lt; 10%</td>
<td>&lt; 5%</td>
</tr>
<tr>
<td>Magnesium Sulphate Soundness</td>
<td>....</td>
<td>....</td>
<td>&lt; 10%</td>
</tr>
<tr>
<td>Soft and Friable Pieces</td>
<td>....</td>
<td>&lt; 5%</td>
<td>&lt; 5%</td>
</tr>
<tr>
<td>Fines (&lt; No. 200 sieve)</td>
<td>....</td>
<td>&lt; 1%</td>
<td>&lt; 1%</td>
</tr>
<tr>
<td>Clay Lumps</td>
<td>....</td>
<td>&lt; 0.5%</td>
<td>&lt; 0.5%</td>
</tr>
<tr>
<td>Bulk Unit Weight</td>
<td>&gt; 1200 kg/m³</td>
<td>&gt; 1120 kg/m³</td>
<td>....</td>
</tr>
<tr>
<td>Particle Specific Gravity</td>
<td>&gt; 2.5</td>
<td>....</td>
<td>&gt; 2.6</td>
</tr>
</tbody>
</table>

2.2.2 Sub-ballast (Capping layer)

It is common practice in Australian Rail Track System to place a layer called sub-ballast (also called a capping layer) between the ballast and the subgrade. It usually consists of well graded aggregate or a mixture of gravel and sand. The main functions of sub-ballast are (Indraratna et al., 2011):

- reducing the traffic induced stress at the bottom of the ballast layer to a tolerable level for the top of the subgrade;
- extending the subgrade frost protection;
- preventing interpenetration of subgrade and ballast (separation function);
- preventing upward migration of fine material emanating from the subgrade;
- preventing subgrade attrition by ballast, which in the presence of water, leads to the formation of slurry, which effectively prevents this source of pumping;
- shedding water, i.e., intercepts water coming from the ballast and directs it away from the subgrade to ditches at either side of the track; and
- permits the drainage of water that might be flowing upwards from the subgrade.

2.2.3 Subgrade

Subgrade is the ground (formation) on which the rail track structure is constructed. It may be naturally deposited soil or artificially filled and compacted material (e.g. rail embankment). It should have adequate load bearing capacity and enough stiffness to resist the load coming from ballast or sub-ballast.

2.3 Track Forces

It is imperative that the forces imposed by train loads onto the ballast bed be known, in order to design and predict the behaviour of ballasted track during its operation. These loads are a combination of static load and a dynamic component superimposed onto the static load. The major factors affecting the magnitude of dynamic load component are (Jeffs and Tew, 1991):

- Speed of train
- Static wheel load
Wheel diameter
Vehicle unsprung mass
Track condition (including track joints, track geometry, and track modulus)
Track construction
Vehicle condition

It is common to express the designed vertical wheel load empirically as a function of the static wheel load, as given below (Jeffs and Tew, 1991):

\[ P_d = \phi P_s \]  \hspace{1cm} (2.1)

where \( P_d \) = designed wheel load (kN), incorporating dynamic effects,

\( P_s \) = static wheel load (kN), and

\( \phi \) = dimensionless impact factor (>1.0).

The most comprehensive method for determining the impact factor (\( \phi \)) is that developed by The Office of Research and Experiments (ORE) of the International Union of Railways (Jeffs and Tew, 1991). In this technique the impact factor is assessed completely based on the measured track forces of locomotives (ORE, 1965). This impact factor is defined in terms of dimensionless speed coefficients \( \alpha' \), \( \beta' \) and \( \gamma' \), as given by:

\[ \phi = 1 + \alpha' + \beta' + \gamma' \]  \hspace{1cm} (2.2)
where \( \alpha' \) and \( \beta' \) are related to the mean value of the impact factor, and \( \gamma' \) is related to the standard deviation of the impact factor.

The coefficient \( \alpha' \) depends on track irregularities, and vehicle suspension and speed. Although it is difficult to correlate \( \alpha' \) with irregularities in the track, it has been found that for the poorest case, \( \alpha' \) increases with the cubic function of speed, and was empirically expressed by:

\[
\alpha' = 0.04 \left( \frac{V}{100} \right)^3
\]  

(2.3)

where \( V \) = vehicle speed (km/hour).

The coefficient \( \beta' \) is the contribution of the impact factor due to a shift in the curves of the wheel load, and may be expressed by either Equation 2.4 or 2.5:

\[
\beta' = \frac{2d.h}{G^2}
\]  

(2.4)

\[
\beta' = \frac{V^2(2h + c) - 2c.h}{127 Rg} - \frac{2c.h}{G^2}
\]  

(2.5)

where \( G \) = horizontal distance between the centre lines of the rails (m),

\( h \) = vertical distance from the top of the rail to the centre of the vehicle’s mass (m),

\( d \) = super elevation deficiency (m),
\( c = \) super elevation (m),

\( g = \) acceleration due to gravity (m/sec\(^2\)),

\( R = \) radius of curve (m), and

\( V = \) vehicle speed (km/hour).

The last coefficient \( \gamma' \), depends on the speed of the vehicle, the condition of the track (e.g. age, hanging sleepers etc.), the design of the vehicle, and how well the locomotives have been maintained. It has been found that the coefficient \( \gamma' \) increases with the speed, and can be approximated by the following expression:

\[
\gamma' = 0.10 + 0.017 \left( \frac{V}{100} \right)^3
\]  \hspace{1cm} (2.6)

A typical distribution of wheel load to the rails, sleepers, ballast, sub-ballast and subgrade, is shown in Figure 2.2.
2.4 Factors Governing the Behaviour of Ballast

The geotechnical properties of fresh ballast are basically governed by four factors, which are summarised in Figure 2.3.
2.4.1 Characteristics of ballast

The mechanical behaviour of ballast varies on the characteristics of the constituent particles, such as size, shape, surface roughness, particle crushing strength, and resistance to attrition.

Several researchers have studied the effects of particle size on the shear behaviour of granular materials but there are some inconsistencies amongst their findings. Gupta (2009), Al-Hussaini (1983), and Kolbuszewski and Frederick (1963) indicated that the angle of shear resistance increases with a larger particle, whereas Marachi et al. (1972) presented experimental evidence that the angle of internal friction decreases.
with an increase in maximum particle size. Indraratna et al. (1998) observed comparable findings in their study and mentioned that the peak friction angle decreased slightly with an increase in grain size at low confining pressure (< 300 kPa) while at high levels of stress (> 400 kPa), the effect of particle size on the friction angle becomes negligible.

There are obvious and observable deviations in the shear strength of different shaped particles. As an example, the shearing angle of a rounded particle is less than that of highly angular particles of the same size.

Surface roughness is regarded as one of the key aspects that govern the angle of internal friction and hence, the strength and stability of ballast. The phenomenon ‘friction and frictional force’ is based on the roughness of the loaded face, so the shear strength of ballast and other aggregates depends on this frictional force.

The strength of the parent rock is most likely to be the most important factor directly governing the degradation of ballast, and indirectly, the vertical and lateral deformation of the track because it includes both compressive and tensile strength. Under the same boundary conditions and loading, weak particles will break easier and with more plastic deformation than strong particles.

2.4.2 Characteristics of bulk aggregate

The characteristics of bulk aggregate such as particle size distribution, void ratio (or density), and the degree of saturation, contribute significantly to the mechanical characteristics. Several researchers have studied the effects of particle size
distribution on the strength and deformation aspects of aggregates. Thom and Brown (1988) conducted a series of repeated load triaxial tests on crushed dolomite with similar maximum particle sizes, but they varied the gradation from well-graded to uniform. According to their findings, with an increase in uniformity, the elastic shear modulus and permeability increases, while the densities and effective friction angles were decreasing. As expected, well graded or broadly graded ballast is stronger due to its void ratio being smaller than uniform ballast (Jeffs and Marich, 1987; Thom and Brown, 1988; Atalar et al., 2001).

It has been well recognised that the void ratio in a porous medium (e.g. soil and rock aggregates) significantly affects its mechanical behaviour (Terzaghi and Peck, 1948; Roscoe et al., 1963; Schofield and Wroth, 1968; Indraratna et al., 1997). Generally, aggregates having a lower initial void ratio (i.e. a higher initial density) are stronger in shear and generate a smaller settlement than aggregates with a higher initial void ratio (i.e. a lower initial density).

Indraratna et al. (1997) conducted 1-D compression tests to investigate the effects of saturation on the deformation and degradation of ballast. They observed a sudden increase in ballast settlement by about 2.6 mm due to sudden flooding. They concluded that saturation increased settlement by about 40% of dry ballast.

2.4.3 Loading characteristics

The deformation and degradation behaviour of ballast is strongly dependent on the external loading characteristics. The magnitude of confining pressure, previous
loading history, current state of stress, number of load cycles, loading frequency and amplitudes are among the key parameters that govern track deformation.

It is common in soil mechanics that with an increase in confining pressure, the shear strength increases. Several researchers have studied the significant effects of confining pressure on the strength and deformation of soils and granular materials from the earliest days of soil mechanics (Terzaghi and Peck, 1948; Roscoe et al., 1958; Drucker et al., 1957, Vesic and Clough, 1968). Marsal (1967) was one of the pioneers who closely studied the effect of confining pressure on the deformation and particle breakage of rockfills. He tested aggregates of basalt and granitic gneiss under high confining pressures (5–25 kg/cm²), and observed that the shear strength is not a linear function of acting normal pressure. Charles and Watts (1980) and Indraratna et al. (1993) also reported a pronounced non-linearity of the failure envelope for coarse granular aggregates at low confining pressure. Vesic and Clough (1968) studied the shear behaviour of sand under low to high pressures and concluded that a mean normal stress exists beyond which the curvature of the strength envelope vanishes and the shear strength is not affected by the initial void ratio. They called it ‘breakdown stress’ (σ_B), because it represents the stress level at which all dilatancy effects disappear and beyond which particle breakage becomes the only mechanism, in addition to simple slip, by which shear deformation takes place.

To investigate the effects of the stress history on the behaviour of ballast, Diyaljee (1987) conducted a series of laboratory cyclic tests. He concluded that a previous stress history of more than 50% of the currently applied cyclic deviator stress significantly decreases the plastic strain accumulation in ballast, whereas a previous
stress history of less than 50% of the currently applied cyclic deviator stress does not contribute to the accumulation of plastic strain. His findings agree with the research previously carried out by the Office of Research and Experiments (1974).

The current state of stress also influences the deformation and degradation of ballast. Many Constitutive models (Roscoe et al., 1958; Muir Wood, 1990; Schofield and Wroth, 1968; Lade and Duncan, 1975) showed that the increment of plastic strain depends on the state of stress and other factors. As the state of stress and another state variable (void ratio) of a soil element moves towards the critical state, the rate of plastic shear strain corresponding to any load increment becomes higher. According to the above theories, at the critical state the shear strain continues to increase at a constant stress and constant volume.

The influence of the number of load cycles on the accumulation of plastic deformation of ballast and other granular media is well recognised. An increase in the number of load cycles generally increases the settlement and lateral deformation of granular particles, including ballast (Indraratna et al., 2006; Suiker et al., 2005). The degree and rate of deformation at various load cycles depends on the static strength of the material, the cyclic load applied, and the confining pressure and loading frequency (Indraratna et al., 2011; Lackenby et al., 2007).

Shenton (1975) conducted a series of cyclic loading tests, varying the frequency from 0.1 to 30 Hz (low frequency range), while maintaining other variables such as the confining pressures and constant load amplitude. From the results he concluded that the frequency of loading does not significantly influence the deformation of ballast
under the selected confining range. However, Indraratna et al. (2009) conducted a series of cyclic loading tests, varying the frequency from 10 to 40 HZ while maintaining all other conditions the same, and concluded that frequency greatly influenced the deformation of Latite ballast. An increase in the frequency increases both the axial strains and the dilations.

It is a very common observation that the loading amplitudes change the deformation and degradation behaviour of ballast, whereas an increase in the loading amplitude increases both the axial strain as well as the dilation trend (Stewart, 1986; Indraratna et al., 2011 and Lackenby et al., 2007).

2.4.4 Particle degradation

All granular aggregates subjected to stresses above a certain level exhibit considerable particle breakage (Marsal, 1967; Terzaghi and Peck, 1948; Vesic and Clough, 1968; Hirschfield and Poulos, 1963; Bishop, 1966; Lee and Farhoom and, 1967; Lee and Seed, 1967; Bilam, 1971; Miura and O-hara, 1979; Hardin, 1985). Some researchers indicate that particle breakage can even occur at low confining pressure (Lade et al., 1996; Miura and O-hara, 1979; Indraratna and Salim, 2002; Lackenby et al., 2007). The most widely used particle breakage indices are the breakage index $B_g$ by Marsal (1967), the breakage potential $B_p$ by Hardin (1985), and the Ballast Breakage Index BBI by Indraratna et al. (2005).

Marsal’s method involves the assessment of change in the overall grain size distribution of aggregates after the test, where the specimens were sieved before and
after each test. From the changes recorded in particle gradation, the difference in percentage retained on each sieve size \( (W_k = W_{ki} - W_{kf}) \) is computed; where \( W_{ki} \) represents the percentage retained on sieve size \( k \) before the test, and \( W_{kf} \) is the percentage retained on the same sieve size after the test. Marsal also noted that some of these differences were positive and some were negative, so he defined the breakage index \( B_g \), as the summation of the positive values of \( W_k \), expressed as a percentage.

Hardin (1985) introduced a relative breakage index as;

\[
B_r = \frac{B_t}{B_p}
\]

(2.7)

where \( B_t \) and \( B_p \) is the total breakage and breakage potential respectively.

The potential for a particle of soil to break increases with its size because the normal contact forces in a soil element increase with particle size and the probability of micro-cracks in a given particle increases with its size. Therefore the breakage of particles of soil in a sample of rockfill under moderate stresses will be quite evident, whereas very high stresses are required to crush silt size particles. Hardins (1985) referenced the breakage of particles to the largest silt size, 0.074 mm. The purpose of this was to define their breakage potential \( B_p \), which is equal to the area between the line defining the upper limit of the silt size \( D = 0.074 \) mm, and the initial particle size distribution (PSD) curve (Figure 2.4). The total breakage \( B_t \) is equal to the area between the initial and final PSD, as shown in Figure 2.4.
Using a similar technique, Lackenby et al. (2007) introduced a new index called the Ballast Breakage Index (BBI) to quantify ballast breakage while understanding the key features in the changing PSD curves for the ballast. They reported that previous triaxial testing on ballast had shown that particle degradation causes a shift in the initial particle size distribution towards smaller particle sizes, while the maximum size remained unchanged before and after loading. Therefore, instead of defining the breakage potential by a single minute size particle, they considered the arbitrary boundary of maximum breakage, which in reality is more appropriate. The definition of the BBI is explained in Figure 2.5.
2.5 Track Substructure Problems

Under heavy traffic loads the rail track gradually deviates from its desired geometry. Neil (1976) has classified substructure failure into two categories, viz. the failure caused by a loss of shear strength and the infiltration of soil slurry (pumping). The most common track substructure problems are:

- differential track settlement
- ballast degradation
- subgrade failure
- ballast fouling
- poor drainage
2.5.1 Differential Track Settlement

Track settlement is a serious issue when it doesn’t occur uniformly along a short length (or breadth) of track, as shown in Figure 2.6 (Gaskin and Raymond, 1976; Selig and Waters, 1996). Ballast has been identified as the main cause of average and differential settlement of railway track with the passage of traffic (Suiker, 1997; Ionescu et al., 1998; Indraratna et al., 2001). Other causes are attributed to the sub-ballast and subgrade. Inadequately confined ballast, and ballast fouling combined with poor track drainage, increases the differential track settlement. Impact loads, the rapid rate of breakage, and localised lateral deformation of ballast also contributes to differential settlement. The pumping of slurry (clay pumping) from the subgrade into the sub-ballast and layers of ballast can also cause vertical and/or horizontal track misalignment.

Figure 2.6: Track suffering from differential track settlement (adopted from Suiker, 1997)
2.5.2 Ballast Degradation

Most ballast degradation occurs due to train loading (Gaskin and Raymond, 1976). More degradation occurs due to the impact load exerted due to wheel and rail imperfections, mainly at crossings and intersections. However, there are other factors such as transportation and handling, weathering, tamping, and the use of compaction machines which contribute to the breaking down of sharp angular particles of ballast (Selig and Waters, 1994). Figure 2.7 shows a track suffering from excessive ballast breakage. The breaking down of ballast into smaller sized particles fouls the track and inhibits drainage. In addition, ballast breakage decreases particle angularity and diminishes its internal frictional angle. Most breakage occurs at the corners of the particles because of its high angularity, although splitting is also occurs, especially at crossings and intersections.
2.5.3 Subgrade failure

Subgrade failure results from either poor subgrade conditions or poor drainage. Low lying areas containing large volumes of soft compressible clays can sustain high excess pore water pressure during train loading, and if it’s in a poorly drained state, excess pore pressures can lead to a swift reduction in effective stresses which cause subgrade failure. The use of prefabricated vertical drains (PVDs) helps dissipate excess pore water pressures by radial consolidation before they can reach critical
levels (Indraratna et al., 2011). These PVDs continue to dissipate excess pore water pressures even after the cyclic load stops. Figure 2.8 shows application of PVDs.

Figure 2.8: Application of prefabricated vertical drains (Colbond, Netherlands)

2.5.4 Ballast fouling

The accumulation of fine particles into the voids of ballast or contamination is defined as “ballast fouling”. Selig and Waters (1994) expressed the sources of ballast fouling as follows:
1) Ballast breakdown:
   
i) Handling
   
ii) Thermal stress from heating
   
iii) Freeze thaw effect
   
iv) Chemical weathering
   
v) Tamping damage
   
vi) Traffic damage
   
vii) From compaction machine

2) Infiltration from the ballast surface:
   
i) Delivered with ballast
   
ii) Dropped from trains
   
iii) Wind blown
   
v) Water borne
   
iv) Splashing from adjacent water spots
   
v) Meteoric dirt

3) Sleeper (tie) wear

4) Infiltration from underlying granular layers
   
i) Break down of old track bed
   
   ii) Sub-ballast particle migration from inadequate gradation

5) Subgrade infiltration
Extensive field and laboratory studies conducted in North America (Selig and Dello Russo, 1991) have concluded that ballast breakdown is the main source of track fouling. However under typical Australian track environments (Figure 2.10), the...
infiltration of fines from sub-ballast and subgrade accounts for up to 58% of fouling, followed by 20% surface infiltration, 20% ballast degradation, and the rest from concrete sleeper wear (Indraratna et al., 2011). Feldman and Nissen (2002) stated that for coal freight tracks in Australia, coal dust accounts for 70% - 95% of the contaminants.

![Pie chart showing different sources of ballast fouling](modified after Indraratna et al., 2011).

**Figure 2.10:** Comparison of different sources of ballast fouling from coal fouled, low lying tracks (modified after Indraratna et al., 2011).

### 2.5.4.1 Quantification of ballast fouling

There are numerous fouling indices available in literature for quantifying ballast fouling. Selig and Waters (1994) have defined the Fouling Index \(FI\) as a summation of percentage (by weight) passing the 4.75mm (No.4) sieve and 0.075mm (No.200) sieve.

\[
FI = P_{4.75} + P_{0.075} \tag{2.8}
\]
\[ P_4 = \text{percentage (by weight) passing the 4.75mm (No.4) sieve} \]

\[ P_{200} = \text{percentage (by weight) passing the 0.075mm (No.200) sieve} \]

Selig and Waters (1994) defined another term for the fouling index and called it “percentage fouling;” the ratio of the dry weight of the fouling material (material passing through a 9.5mm sieve) to the dry weight of the total sample. They further suggested sub-dividing the degree of fouling into five categories according to the fouling index given in Table 2.2.

<table>
<thead>
<tr>
<th>Category</th>
<th>( F_I )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean</td>
<td>[ F_I &lt; 1 ]</td>
</tr>
<tr>
<td>Moderately clean</td>
<td>[ 1 &lt; F_I &lt; 10 ]</td>
</tr>
<tr>
<td>Moderately fouled</td>
<td>[ 10 &lt; F_I &lt; 20 ]</td>
</tr>
<tr>
<td>Fouled</td>
<td>[ 20 &lt; F_I &lt; 40 ]</td>
</tr>
<tr>
<td>Highly fouled</td>
<td>[ F_I &gt; 40 ]</td>
</tr>
</tbody>
</table>

Ionescu (2004) modified Selig’s fouling Index to suit the Australian specifications of ballast

\[ FI_p = P_{13.2} + P_{0.075} \quad (2.9) \]

\[ P_{13.2} = \text{percentage (by weight) passing the 13.2mm sieve} \]

\[ P_{0.075} = \text{percentage (by weight) passing the 0.075mm sieve} \]

Ionescu (2004) defined another fouling Index \((FI_D)\) to capture the resiliency characteristics of ballast:

\[ FI_D = \frac{D_{90}}{D_{10}} \times 100\% \quad (2.10) \]
D$_{10}$ = a particle size corresponding to 10% finer

D$_{90}$ = a particle size corresponding to 90% finer

Queensland Rail was using another fouling index called the D-bar method (Feldman and Nissen, 2002). The D-bar is a number which represents the weighted geometric average of particle sizes passing through grading sieves from a full sample.

The above fouling indices are based on weight wise. This means they may lead to a misinterpretation of the actual degree of fouling if the fouled material contains more than one type of material having considerably different specific gravities (e.g. coal and pulverised rock).

Alternatively, Feldman and Nissen (2002) defined the Percentage Void Contamination (PVC) as the ratio of the bulk volume of fouling material to the initial volume of ballast voids (i.e. when it was clean).

$$PVC = \frac{V_{vf}}{V_{vb}} \times 100$$

(2.11)

$V_{vf}$= bulk volume of fouling material

$V_{vb}$= volume of voids within a total ballast

The parameter $V_{vf}$ needs to be calculated after the fouling material has been compacted (Feldman and Nissen, 2002)

Table 2.3 shows a comparison of commonly used fouling indices.
Table 2.3: Comparison of all Fouling indices

<table>
<thead>
<tr>
<th>Category</th>
<th>Clean</th>
<th>Moderately clean</th>
<th>Moderately fouled</th>
<th>Fouled</th>
<th>Highly fouled</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fouling index (Selig and Waters 1994) (%)</td>
<td>&lt;1</td>
<td>1 to &lt;10</td>
<td>10 to &lt;20</td>
<td>20 to &lt;40</td>
<td>≥40</td>
</tr>
<tr>
<td>Percentage of fouling (%)</td>
<td>&lt;2</td>
<td>2 to &lt;9.5</td>
<td>9.5 to &lt;17.5</td>
<td>17.5 to &lt;34</td>
<td>≥34</td>
</tr>
<tr>
<td>FI_d (Ionescu 2004)</td>
<td>&lt;2.1</td>
<td>2.1 to &lt;4</td>
<td>4 to &lt;9.5</td>
<td>9.5 to &lt;40</td>
<td>≥40</td>
</tr>
<tr>
<td>PVC/ (%)</td>
<td>0-20</td>
<td>-</td>
<td>20-29</td>
<td>&gt;30</td>
<td>-</td>
</tr>
</tbody>
</table>

2.5.4.2 Effect of ballast fouling

The effect of ballast fouling depends on the amount and properties of the fouled materials. Sand and fine gravel size fouling material will increase the shear strength and stiffness, if particles of coarse ballast still form the ballast skeleton. However, the porosity ratio and resilient will decrease. Generally, a higher degree of fouling from particles of coarse sand and gravel will not considerably increase the maintenance costs (Selig and Waters, 1994).

Loss of performance mainly occurs when the fouling material consists of clay and silt size particles. In Queensland and NSW major problems of track maintenance are attributed to ballast fouling from the infiltration of coal and clay pumping.
Ballast fouling reduces the fulfilment of its major functions described in section 2.2.1, but the particular effect depends on the properties and type of fouling material, as well as the degree of fouling. When the ballast voids are filled with fine material, the drainage capacity of the track is reduced. Moreover, due to the lubrication effects of fouling material, internal friction angle of the ballast decreases, which lowers its shear strength.

Tutumler et al. (2008) conducted Large scale direct shear (shear box) laboratory tests at the University of Illinois on samples of granite ballast obtained from the Powder River Basin (PRB) joint line in Wyoming to measure the strength and deformation characteristics of both clean (new) and ballast aggregates fouled with coal dust. They observed that as the degree of fouling increased, the shear strength always decreased. Figure 2.11 shows the shear strength envelopes at 200kPa normal stress for ballast fouled with coal and clay dust, under wet and dry (at optimum moisture content) conditions.
Another attempt was made by Budiono et al. (2004) to investigate the effect of fouling (dry coal dust) on ballast settlement, and they concluded that at small vertical pressures, the degree of fouling appeared to have no effect on settlement, but at large pressures, the increase in the degree of fouling increases the plastic settlement. The results of the cyclic tests showed that when PVC=18%, the pressure-settlement curve coincided reasonably well with the fresh ballast, but when the PVC=55% plastic
settlement was lower than for the specimen of fresh ballast. This indicates that the fouling material filling the ballast voids increases the elastic settlement. Due to the confining effect, the three specimens became stiffer with an increase in cyclic loading. At a low number of cycles (up to 2 cycles for PVC=18% and 3 cycles for PVC=55%) the pressure/settlement was higher than those for fresh ballast. They derived the following equation for plastic settlement ($p$) in relation to the number of cycles ($N$)

$$p = 0.96N (N+2.33) \quad \text{for PVC}=18\% \quad (2.12)$$

$$p = 0.96N (N+1.88) \quad \text{for PVC}=55\% \quad (2.13)$$

However, these equations are only validated for a small number of cycles (<10) and the fouling materials used were in a dry state, whereas previous studies (Huang et al., 2008b) concluded there is a significant increase in settlement when the fouling materials are in a wet state.

Most of deformation models were developed only for clean ballast, but many railroad investigators (Han and Selig, 1996) pointed out that a great deal of the deformation of a ballast bed stems from fouling. They conducted a ballast box test at various degrees of fouling, and under cyclic loading, and then recorded the settlements. They then concluded that as the degree of fouling increased, so too did the initial and final settlement. Furthermore, ballast fouled with moist sand and dry clay resulted in less settlement at all degrees of fouling whereas ballast fouled with moist clay and moist silt resulted in a marked settlement. However, they couldn’t conduct some tests up to 100,000 cycles due to restrictions by the actuator displacement (max 75mm), so
it is necessary to conduct comprehensive laboratory experiments taking confining pressure into account

Currently there were very limited literatures on the triaxial behaviour of fouled ballast. Therefore, in order to understand the strain-strain behaviour of fouled ballast, it is logical to first examine the behaviour of sand mixed with fines. Holtz and Willard (1956) may have been the earliest to investigate the shear strength of clayey soils with gravel contents varying from 0% to 65%. They concluded that the friction angle increased and apparent cohesion decreased when the content of gravel increased. With gravel content greater than 50%, the effect of the granular part of the mixture was readily apparent.

Ni et al. (2004) conducted a series of tri-axial tests for host sand and host sand mixed with 9% silica and kaolin, and concluded that an increase of 9% (by weight) of silica (non-plastic fines) increased the shear strength of the host sand while an increase of 9% of kaolin (plastic fines) decreased the shear strength of the host sand. However this has not been validated for the whole range of fine contents where the density of the granular materials remains unchanged.

Jafari and Shafiee (2004) conducted a series of consolidated undrained triaxial tests under three different initial confining pressures (100, 300 & 500kPa) for different proportions of kaolin and gravel content, as shown in Figure 2.12. They concluded that an increase in the amount of gravel generally increases the shear strength of the mixture.
Figure 2.12: Consolidated undrained tri-axial test for a mixture of kaolin and gravel at various confining pressures (Data from Jafari and Shafiee, 2004)
None of the above investigations considered the constant density of the granular skeleton, because it was obvious that the shear strength changes if the density of the granular skeleton changes. However, the above investigations have helped in our understanding of the general behaviour of a mixture of clay and granular materials.

2.5.5 Poor Drainage

Drainage plays an important role in the stability of rail track substructure, as fouling of ballast causes a decrease in its drainage capacity. In saturated tracks, poor drainage can lead to a build up of excess pore water pressure under train loading. If the permeability of the layers of substructure becomes markedly low, train loading induces a considerable build up excess pore water pressure which is often not dissipated sufficiently before the next train load is applied. Thus, the residual pore pressure accumulates with increasing load cycles, which often leads to a drastic reduction in the load bearing capacity of the track. Figure 2.13 shows rail tracks suffering from inadequate drainage. In the case of poor drainage, the following problems may occur in the track:

- Decreased ballast shear strength, stiffness, and load bearing capacity
- Increased track settlement
- Softening of subgrade
- Formation of slurry and clay pumping under cyclic loading
- Ballast attrition by jetting action and freezing of water
- Sleeper degradation by water jetting
All the above problems will degrade the performance of the track and demand additional maintenance effects.

Figure 2.13: Tracks suffering from poor drainage (Adopted from Lackenby, 2006)
2.5.5.1 Track drainage

The primary purpose of drainage is to remove water from the substructure of the track as quickly as possible and keep the load bearing stratum relatively dry. To accomplish this objective, the load bearing layer (ballast) is usually composed of coarse and uniformly graded aggregates with large voids that ensure a sufficiently high permeability. Since the ballast is laid on fine grained subgrade, a filtering layer (sub-ballast) is usually placed below the ballast in order to prevent the upward ingress of fines.

Water can penetrate into the load bearing stratum from four different sources (Indraratna et al., 2011):

- Precipitation (rain and snow)
- Surface flow from adjacent hill slopes
- Upward seepage from the subgrade, and
- High groundwater table in low lying coastal regions.

Track substructure should be designed and constructed so as to drain the water into nearby drainage ditches or pipes. Internal drainage is usually ensured by placing a layer of sub-ballast having an appropriate gradation (Indraratna et al., 2011).
2.5.5.2 Drainage requirements

To design a satisfactory drainage system, it is imperative to first examine the conditions of the subsurface, ground water, and climate. Subsurface investigations must be carried out to characterise the subgrade soils including type, layering, and permeability. The proposed drainage system should have sufficient capacity to drain the highest expected rate of water during the designed life of the system. The first requirement is to keep the ballast clean enough to ensure a sufficiently high permeability for rapid drainage (Selig and Walter, 1994). Secondly, the surface of the sub-ballast and subgrade should be sloped towards the sides. The third requirement is to provide a suitable means (channel or conduit) of carrying the away the water which emanates from the substructure, as shown in Figure 2.14.

![Figure 2.14: Schematic illustration of a track drainage system (adopted from Indraratna et al., 2011)](image)

As mentioned previously, an increasing degree of fouling significantly reduces the drainage capacity.
Many researchers have attempted to model the hydraulic conductivity of granular soil.

For fairly uniform sand, Hazen (1911) proposed an empirical relationship as:

\[ k = cD_{10}^2 \]  

(2.14)

- \( k \) = coefficient of permeability/(cm/s)
- \( c \) = empirical constant
- \( D_{10} \) = effective size /(mm)

A theoretical formulation for the coefficient of permeability generally referred to the Kozeny-Carman equation (Carrie, 2003) is given by:

\[ k = \frac{1}{C_s S_v^2 T^2} \frac{\gamma_w}{\mu} \frac{e^3}{(1+e)} \]  

(2.15)

where

- \( k \) = coefficient of permeability
- \( C_s \) = shape factor
- \( S_v \) = surface area per unit volume
- \( T \) = Tortuosity
- \( \gamma_w \) = unit weight of water
- \( \mu \) = absolute coefficient of viscosity
- \( e \) = void ratio
Those models work well for some types of granular materials such as sands and silts, but with coarse grained aggregate such as ballast, with its larger and inter-connected pore structure, the change of hydraulic conductivity with respect to the porosity, is usually insensitive unless a large amount of fines such as coal and clay have accumulated in the voids. In order to represent the hydraulic conductivity ($k$) of a mixture of granular soil and fine grained soil, Koltermann and Gorelick (1995) proposed:

$$k = \frac{d_{fp}^2 \phi_{fp}^3}{180(1 - \phi_{fp})^2}$$

(2.16)

where $\phi_{fp}$ is the composite porosity of the mixture and $d_{fp}$ is the representative diameter of the grain.

The above model fails to represent fouled ballast in the track because it assumes the fine particles to be distributed uniformly throughout the voids, whereas in the field, fouling materials increasingly accumulate towards the bottom of the ballast layer (i.e. vertical migration under vibration and rainfall ingress and subsequent compaction upon the passage of trains).

### 2.6 Track Maintenance

Rail track requires regular maintenance to remain in a good working state, especially for high speed train corridors. Inadequate maintenance can lead to a speed restriction being imposed to avoid accidents. Rail tracks progressively deform vertically and
laterally under the influence of varying traffic loads and speeds. Moreover, ballast degradation, weakness of the subgrade and deficient drainage also leads to track maintenance.

Worldwide, track maintenance is a costly routine exercise with a major portion of the maintenance budget being spent on geotechnical problems (Raymond et al., 1975; Shenton, 1975 and Indraratna et al., 1998). In many countries, including the USA, Canada, and Australia, hundreds of millions of dollars are spent each year on large terrains of rail track, predominantly on maintaining the ballast (Raymond et al., 1975; and Indraratna et al., 1998).

The use of available resources and timely adoption of innovative technologies to improve the riding quality and safety levels, while reducing the maintenance costs, still remains a challenging task for the engineers. In this Chapter the conventional and the very latest methods used for track maintenance are explained.

2.6.1 Track maintenance techniques

In early days, track maintenance used to be a hard laborious task, requiring the manual handling of lining bars to correct irregularities in horizontal alignment (line), and the manual handling of tamping and jacks to correct vertical irregularities (surface). Currently, maintenance is facilitated by a variety of specialised tamping machines (tampers).
2.6.1.1 Ballast tamping

Ballast tamping is a worldwide practice used to correct and restore the track geometry. It consists of lifting the track and laterally squeezing the ballast beneath the sleeper to fill the void spaces generated by the lifting operation. Thus the sleepers retain their elevated positions. These steps involved vibrations. Figure 2.15 shows a typical tamping machine used for track maintenance and Figure 2.16 gives a closer view of the machine showing tamping tines and lifting rollers. The lifting and lining rollers grip the head of the rails and can lift the track to a predetermined level. It can also move the rails laterally to re-align the track.

![Typical Tamping Machine](image.png)

Figure 2.15: A typical tamping machine used for track maintenance
Figure 2.16: A closer view of the tamping machine showing tamping tines and lifting rollers (Adopted from Indraratna et al., 2011)

Figure 2.17 shows the tamping tines penetrating into the ballast layer. Tamping is an effective process for re-adjusting the track geometry, although it can cause ballast breakage and reduce the lateral resistance of the track due to an insufficient packing of ballast particles. Subsequent buckling often accompanies this process. Furthermore, loosening the ballast by tamping causes high settlement in the track, which means that tamping is eventually needed again over a shorter period of time, and in the long run, the ballast layer gradually holds the track geometry. Eventually fouled ballast must be either replaced, or cleaned and re-used in the track. In addition to loosening of ballast, tamping rearranges and reorients the particles which then form new particle contacts. This accelerates the breakage of ballast while increasing the frictional resistance. (Selig and Waters, 1994)
Figure 2.17: Tamping tines penetrating the ballast layer. (Adopted from Indraratna et al., 2011)

Figure 2.18 shows the sequence of events involved in a tamping cycle (Selig & Waters, 1994):
2.6.1.2 Stone blowing

The stone blower is a technique that forcefully injects stone to the surface of the existing ballast and is an alternative form of maintenance. It is relatively a new, mechanised method of reinstating railway track to its desired line and level (Anderson et al., 2001 and Key, 1998). Before mechanised tamping appeared, track are re-levelled by ‘hand shovel packing’, where the sleepers are raised and fine aggregates are shovelled into the voids with minimum disturbance to the well
compacted ballast. The mechanised version of this process is known as ‘pneumatic ballast injection’ or ‘stone blowing’ (Anderson et al., 2001).

The stone blowing process operates in four stages (Selig & Waters, 1994):

1) The geometry of the existing track is measured;

2) The precise track lift required at each sleeper to restore it to an acceptable level is calculated;

3) The volume of stone that needs to be blown beneath the sleeper is deduced from the known relationship between the volume of added stone and residual lift and,

4) The track is then stone blown.

Different stages of the stone blowing process are shown in Figure 2.19 (Selig & Waters, 1994). At (A) the sleeper rests in the ballast before adjustment. At (B) the sleeper is raised to create a void below. At (C) the stone blowing tubes are driven down alongside the sleeper. At (D) a measured quantity of stone is blown by compressed air into the void. This quantity can be determined when compacted by subsequent traffic which will raise the sleeper by the amount needed to achieve the required track geometry. At (E) the stone blowing tubes are withdrawn, leaving the blown stone resting on the surface of the ballast that was originally supporting the sleeper. At (F) the sleeper is lowered onto the surface of the blown stone where it will be compacted by subsequent traffic. Figure 2.20 shows a typical stone blowing machine.
Figure 2.19: Stone blowing process (Selig & Waters, 1994)

Figure 2.20: Stone blowing machine (Esveld, 2001)
Anderson et al. (2001) reported the real track data measured in the UK both before and after stone blowing (Figure 2.21). They concluded that this technique improves the profile of the track significantly. Before stone blowing appeared, deterioration in the track was monitored over time, as revealed by the increasing standard deviation (Figure 2.21). In contrast, after stone blowing, the quality (represented by the standard deviation) and longevity of the track improved significantly. Although stone blowing results into reduced settlement under repeated loading, repetitive stone blowing can result in a large percentage of fines and reduced drainage. It is also not clear whether stone blowing will be useful in areas where track settlement is mainly due to soft subgrade (Indraratna et al., 2002).

Figure 2.21: Improvement in the vertical profile of track after stone blowing [after Anderson et al. (2001)]
2.6.1.3 Ballast cleaning and ballast renewal

When ballast becomes excessively fouled, it malfunctions even after using other maintenance techniques (e.g. tamping or stone blowing). In such circumstances the contaminated ballast must be cleaned or replaced with fresh ballast. The cleaning and renewal of ballast is a costly and time consuming exercise that can also disrupt traffic flow, and therefore it is not frequently undertaken.

The decision about which appropriate corrective measure should be undertaken is governed by the condition of the site and an in-situ investigation of the track layers, including the sub-surface profile. Cleaning fouled ballast is usually carried out by a track mounted cleaner, as shown in Figure 2.22. The cleaner excavates the portion of ballast below the sleepers by a chain with ‘excavating teeth’ attached, conveys it up to a vibrating screen, which separates the dirt (fines) from the coarser aggregates (Esveld, 2001). The dirt is then conveyed away to line side or spoil wagons for disposal, while the now clean ballast is returned for re-use in the track.

The ballast cleaner usually separates the fines from the particles of ballast to ensure there is a uniform depth of compacted and clean layer of ballast resting on the smooth cut surface of a compacted layer of sub-ballast. However, past experience indicates that the cutter bar is not able to cut the geometrically smooth surface required for the compacted layer of sub-ballast due to mechanical vibrations and operator dependent cutting depths (Selig and Waters, 1994).
When ballast becomes excessively fouled, it may need complete replacement with fresh ballast rather than on-track cleaning. In these circumstances the cleaning machine removes the fouled ballast and conveys it into the wagons. The conveyor/hopper wagons are then moved to a discharge side for stock piling and/or recycling. Figure 2.23 shows a typical large stockpile of waste ballast at a Sydney suburb (Chullora, NSW).
2.7 Prediction of Ballast Service Life

Feldman and Nissen (2002) and Ionescu (2004) have proposed a method for predicting the life cycle of ballast based on fouling. They emphasised that this process requires thorough monitoring and investigation of fouling before the fouling rate could be calculated. They proposed that samples of ballast should be investigated from every two kilometres along a section of track to estimate the average degree of fouling for the given track section. The fouling rate can then be calculated as;
Rate of Fouling = \frac{\text{Average degree of fouling along a track section}}{\text{Life of ballast}} \hspace{1cm} (2.17)

If the allowable degree of fouling is known, the maintenance cycle (Ballast maintenance period) can be determined by dividing the allowable degree of fouling from the rate of fouling as;

\text{Ballast maintenance period} = \frac{\text{Allowable degree of fouling}}{\text{Rate of fouling}} \hspace{1cm} (2.18)

As will be demonstrated in the following chapters, the role of ballast fouling by clay and its implication on track performances constitutes the essence of this research study.
3 NEW FOULING INDEX

3.1 Introduction

Most of the fouling indices discussed in Chapter 1 are based on mass ratios, including those proposed by Selig & Waters (1994), Ionescu (2004) and the D-bar method introduced by Queensland Rail (Feldmen and Nissen, 2002). None of these represent the influence or the degree of void clogging. The fouling index introduced by Feldman and Nissen (2002) is much better than the mass based fouling indices since it gives an indication based on the volume of voids within the ballast.

The volume of fouling material in their method needs to be calculated after compacting the fouling material (Feldmen and Nissen, 2002), and that does not always represent the actual volume of fouling accurately in a track environment. In view of the above, a new parameter, Void Contaminant Index (VCI) is proposed herewith that can capture the role of different fouling materials as a modification to the PVC.

\[
VCI = \frac{V_f'}{V_{vb}} \times 100
\]  

(3.1)

where \(V_f'\) is actual volume of fouling material within the ballast voids. The detailed information for the field procedure in order to obtain these parameters is given in the section 3.2. By substituting the relevant soil parameters, Equation (3.1) can be re-written as:

\[
VCI = \frac{(1+e_f)}{e_b} \times \frac{G_{sb}}{G_{sf}} \times \frac{M_f}{M_b} \times 100
\]  

(3.2)
where

\[ e_b = \text{Void ratio of clean ballast} \]
\[ e_f = \text{Void ratio of fouling material} \]
\[ G_{sb} = \text{Specific gravity of clean ballast} \]
\[ G_{sf} = \text{Specific gravity of fouling material} \]
\[ M_b = \text{Dry mass of clean ballast} \]
\[ M_f = \text{Dry mass of fouling material} \]

For example, a value of \( VCI = 50\% \) indicates that half of the total voids in the ballast is occupied by the fouling material. The effect of fouling on geotechnical characteristics such as permeability and shear strength depends on the type of fouling materials (e.g. coal vs. clay). Therefore, a proper understanding of the nature of fouling materials is pertinent irrespective of the quantity of fouling. For example, sand and coal fouling may not decrease the overall permeability of the track significantly, while clay fouling can decrease the track drainage more dramatically (Selig and Waters, 1994).

The laboratory tests to measure their FI, PVC and VCI values were carried on clay-fouled ballast (simulated with kaolin as the fouling material), sand-fouled ballast (simulated with clayey fine sand as fouling material) and coal-fouled ballast. Figure 2 shows the comparison between \( FI, \ PVC, \) and \( VCI \) for various percentages of fouling.
For instance, let us consider 15% fouling by mass for coal-fouled, clay-fouled, and sand-fouled ballast, where the corresponding VCI values are 78%, 65%, and 52%, respectively. The associated values of FI for these different percentages are 16, 28, and 15, respectively. It is clear that the coal-fouled and sand-fouled ballast give a
very close value to each other (difference of 16-15 = 1) in spite of the difference in the specific gravities of coal and sand (quartz), compared to the difference in VCI (78-52 = 26). The PVC values for the three fouling materials are 54%, 48%, and 42%, but these three values are less widely spread (42-54%) compared to the range of VCI values (52-78%). Therefore, VCI is more sensitive to changes in the type and extent of fouling, apart from being more realistic, as it is the only method of characterising fouling that incorporates the specific gravity of the fouling material. The initial placement density of the ballast in the actual rail track is often ascertained as a standard practice in Australia. Most Australian standards for ballast (AS 2758.7, 1996; TS 3402, 2001) recommend the range of in-situ densities of the ballast. While authors agree that these can vary in the field depending upon the tamping efforts, they can still be considered as reasonable estimates. The ballast degradation also substantially contributes to ballast fouling. This in turn justifies the need for a more rational parameter such as VCI which can consider the effect of types of fouling material such as coal, clay, sand, and mineral filler resulting from ballast breakage. The need for additional laboratory tests such as specific gravity, moisture content, and proper field sampling procedure should be encouraged in order to avoid costly track maintenance works which are often governed by an inaccurate assessment of fouling based on mass based fouling indices such as FI.

3.2 Determination VCI in the Field

The method for determining the in-situ ballast density inspired after Selig and Waters (1994) has been adopted to determine VCI. The ballast is excavated in several layers so that the layer of fouled ballast is properly identified. Figure 3.2 presents the field
test set up. The test device is composed of (a) top and bottom cylindrical moulds with known volumes, (b) the base plate, (c) the top plate, and (d) the displacement gauge.

Figure 3.2: Field test set up for determining VCI

The stepwise procedure is illustrated below for the case of two layers (Figure 3.2):

1) Remove the first layer of ballast and mark (or measure) its thickness to establish a datum, and then fill the hole with a known volume of water ($V_1$).

2) Remove the second layer of ballast.

3) Fill the remaining hole with a known volume of water ($V_2$).
4) Using 9.5 mm sieve, separate the fouling material from the ballast particles.

5) Determine the dry weights of the clean ballast \(M_{b1}, M_{b2}\) and the dry weights of the fouling material \(M_{f1}, M_{f2}\) for layers 1 and 2 respectively.

6) Determine the specific gravities of the ballast particle \(G_s\) and fouling material \(G_{sf}\)

7) Calculate the initial void ratio of ballast \(e_b\) for the initial density of the ballast \(\rho_{b}\) when the track was constructed.

\[
e_b = \left( \frac{G_s \rho_w}{\rho_b} \right) - 1
\]  

(3.3)

8) Calculate the void ratio of the ballast in both layers using following equations

\[
e_{b1} = \left( \frac{G_s \rho_w}{\rho_{b1}} \right) - 1
\]  

(3.4)

\[
e_{b2} = \left( \frac{G_s \rho_w}{\rho_{b2}} \right) - 1
\]  

(3.5)

where \(\rho_{b1}\) and \(\rho_{b2}\) are the density of ballast in layer 1 and 2 respectively as determined below;

\[
\rho_{b1} = \frac{M_{b1}}{V_1}
\]  

(3.6)

\[
\rho_{b2} = \frac{M_{b2}}{V_2}
\]  

(3.7)
8) Calculate the void ratio of fouling materials \((e_{f1}, e_{f2})\) for layers 1 and 2, respectively.

\[
e_{f1} = \left( e_{b1} \frac{M_{b1}}{M_{f1}} \frac{G_{sf1}}{G_{sb}} \right) - 1 \tag{3.8}
\]

\[
e_{f2} = \left( e_{b2} \frac{M_{b2}}{M_{f2}} \frac{G_{sf2}}{G_{sb}} \right) - 1 \tag{3.9}
\]

9) Determine the VCI for each layer substituting \(G_{sb}, G_{sf1}, G_{sf2}, M_{f1}, M_{f2}, M_{b1}, M_{b2}, e_{f1}, e_{f2}, \) and \(e_b\) using Equation 3.6 and 3.7.

\[
VCI_1 = \frac{(1 + e_{f1})}{e_b} \times \frac{G_{sb}}{G_{sf1}} \times \frac{M_{f1}}{M_{b1}} \times 100 \tag{3.10}
\]

\[
VCI_2 = \frac{(1 + e_{f2})}{e_b} \times \frac{G_{sb}}{G_{sf2}} \times \frac{M_{f2}}{M_{b2}} \times 100 \tag{3.11}
\]

One of the salient benefits of this approach is that it accurately assesses how the fouling materials are distributed within the pore structure of the ballast that is lacking in previously established indices such as \(FI\) and \(PVC\). The track drainage capacity is also governed by both the location and extent of fouling, and this information can be accurately obtained by using the field procedure described here. Also when there are different fouling materials with different specific gravities, the resulting different volumes of fouling materials occupying the ballast voids can be correctly captured using this VCI, as shown earlier in Figure 3.1.
4 DRAINAGE CHARACTERISATION OF FOULED BALLAST

4.1 Introduction

In order to ensure acceptable track performance, it is necessary to maintain good drainage within the ballast layer. A series of large scale constant head hydraulic conductivity tests were conducted with different levels of fouling to establish the relationship between the void contamination index and associated hydraulic conductivity. Subsequently, a numerical analysis was executed to simulate a more realistic two-dimensional flow under actual track geometry that captures the drainage capacity of ballast in relation to the void contamination index. In the context of observed test data, the drainage condition of the track could be classified into different categories together with a classification chart that captures the degree of fouling. The contents of this chapter have already been considered in track maintenance schemes in the States of Queensland and New South Wales.

4.2 Large Scale Permeability Test

To investigate the effect of fouling on the overall hydraulic conductivity of ballast, a series of large scale permeability tests were conducted on:

i) a clean ballast that was artificially fouled (fouling materials are shown in Table 4.1) in the laboratory under controlled test conditions, and

ii) samples of fouled ballast obtained from multiple field sites
There are two distribution patterns of fouling material within the ballast voids that can be observed in a fouled, ballasted track. Firstly, fouling material infiltrates from the top of the track and settles to the bottom, as shown in Figure 4.1 (non-uniformly distributed fouling, e.g., coal fouling).

![Schematic diagram of non-uniform fouled ballast](image)

Figure 4.1: Schematic diagram of non-uniform fouled ballast

In the second case, fouling material accumulates within the voids of ballast due to subgrade pumping, as shown in Figure 4.2 (uniformly distributed fouling e.g., clay fouling). Both fouling patterns were simulated in the large scale permeability test described in this section.
4.3 Materials

4.3.1 Laboratory simulated fouled specimens

The gradation of clean ballast obtained from Bellambi, NSW is illustrated in Figure 4.3 together with the gradation specified by AS 2758.7 (1996). Fouling materials having different gradation curves (Figure 4.3) were used. The properties of clean ballast and fouling materials that have been used in permeability testing are shown in Table 4.1. When ballast is mixed with coal fines, clayey fine sand and kaolin clay, it is denoted in this chapter as coal-fouled, sand-fouled, and clay-fouled ballast, respectively. An initial void ratio \( e_b \) of 0.88 was determined by saturating the ballast with a known volume of water, and then applying the relevant weight-volume relationships.
4.3.2 Real track-fouled specimens

In order to evaluate in-situ track fouling conditions, samples of coal-fouled ballast were collected from sites at Rockhampton (Queensland, Australia), Bellambi (New
South Wales, Australia) and samples of clay-fouled ballast from Sydenham (New South Wales, Australia). Laboratory tests to measure their $FI$, $PVC$ and $VCI$ values and large-scale hydraulic conductivity tests were carried out on the samples retrieved at these locations.

4.4 Specimen preparation

4.4.1 Laboratory-based fouled specimens

A large-scale permeameter (Figure 4.4) was used to measure the hydraulic conductivity associated with different levels of fouling.

![Large Scale Permeability apparatus](image-url)

Figure 4.4: Large Scale Permeability apparatus
This chamber can accommodate ballast specimens of 500 mm in diameter and 300-500 mm in height. The sample size ratio (diameter of test sample to the maximum particle size of ballast) should be greater than 6 in order to minimise the effects of the sample size (Marachi et al., 1972, Indraratna et al., 1993). According to AS 1289.6.7.3-1999, the height of the specimen should be greater than at least 5 times the maximum particle size. Also, the thickness of the ballast layer in Australian rail track varies between 300 mm and 500 mm. In view of this, a specimen height of 500 mm was considered appropriate in this study. In order to prevent the wash out of fine particles, a filter membrane was placed at the base of the ballast layer while maintaining a free drainage boundary. The test specimen was placed above the filter membrane and compacted in four equal layers to represent typical field density. Figure 4.5 shows a schematic diagram of the large-scale permeability test apparatus.
There are two common types of fouling that can be seen in Australia, namely, coal fouling (surface infiltration, Figure 4.6-a) due to coal spilling from wagons and clay pumping due to the instability of soft subgrade (Figure 4.6 b).
Two fouling patterns were simulated in the laboratory. For the case of non-uniformly distributed fouling, the ballast was compacted and then the fouling material was added from the top and allowed to infiltrate downwards with percolating water. To simulate uniformly distributed fouling, a given volume of kaolin was pre-mixed with the ballast aggregates and then compacted in 5 layers. For 100% VCI, kaolin was placed at the bottom of the permeameter and then the layer of ballast was placed on top of it and compacted using a vibrating plate until the required height was achieved for each layer, with the excess kaolin inevitably squeezed out to the top. The total volume and the weight of the ballast and its gradation were kept equal for each test to maintain a similar initial porosity (or similar voids volume within the ballast). The initial pore structures for all the samples were kept comparable to each other as much as possible. The initial porosities varied within the range of 0.408 to 0.416.
4.4.2 Track-fouled specimens

The specimens obtained in the field were re-compacted in 5 layers to obtain a height of 500mm. The bulk density of fouled ballast was similar to the field density measured at the sites before the specimens were collected. When field samples were tested in the laboratory, though the samples were in disturbed state the it was observed that fouling material settled down to the bottom due to percolation of water and compaction using the vibratory hammer.

4.5 Testing Procedures

A study of one-dimensional flow was imperative for investigating the influence of the degree of fouling on the overall hydraulic conductivity of fouled ballast. To study the effect of fouling, further extensive laboratory tests were carried out on lab-fouled specimens by varying the VCI from 0 to 100. A total of 29 tests (Table 4.2) consisting of 11 tests on coal-fouled ballast, 11 tests on sand-fouled ballast, and 7 tests on clay-fouled ballast were performed using the large scale permeability apparatus (AS: 1289.6.7.1). For field-obtained specimens, a total of 3 laboratory tests were conducted.
Table 4.2: Details of the experimental program

<table>
<thead>
<tr>
<th>Test Material</th>
<th>Test Number</th>
<th>Void Contaminant Index (VCI), %</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coal-fouled ballast</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(lab-fouled specimens)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CO1</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>CO2</td>
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<td></td>
</tr>
<tr>
<td>CO3</td>
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<td>CO8</td>
<td>57</td>
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</tr>
<tr>
<td>CO9</td>
<td>77</td>
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</tr>
<tr>
<td>CO10</td>
<td>94</td>
<td></td>
</tr>
<tr>
<td>CO11</td>
<td>100</td>
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</tr>
<tr>
<td><strong>Sand-fouled ballast</strong></td>
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<td></td>
</tr>
<tr>
<td>(lab-fouled specimens)</td>
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<tr>
<td>S1</td>
<td>0</td>
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<tr>
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<td></td>
</tr>
<tr>
<td>S11</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td><strong>Clay-fouled ballast</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(lab-fouled specimens)</td>
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<td></td>
</tr>
<tr>
<td>CL1</td>
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</tr>
<tr>
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<tr>
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<td>75</td>
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<td>CL6</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>CL7</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td><strong>Clay-fouled ballast</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(field-obtained specimens)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F01 (Sydenham site)</td>
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</tr>
<tr>
<td><strong>Coal-fouled ballast</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(field-obtained specimens)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F02 (Bellambi site)</td>
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<td></td>
</tr>
<tr>
<td><strong>Coal-fouled ballast</strong></td>
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<td></td>
</tr>
<tr>
<td>(field-obtained specimens)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F03 (Rockhampton site)</td>
<td>72</td>
<td></td>
</tr>
</tbody>
</table>
Parson (1990) reported that for fresh ballast, linear Darcy’s law is still valid at low hydraulic gradients of < 4. Therefore, Darcy’s law considering laminar flow was adopted in this study. The fouled specimen was saturated for at least 24 hours. These tests were conducted under a steady state flow subjected to a 1.5 m head of water using an adjustable overhead tank. The steady flow conditions were ensured by obtaining three consecutive k values with a minimum variation of less than 1%.

The hydraulic conductivity of the equipment, including the base materials (crushed uniformly graded ballast 63 mm size aggregates), was tested before the placement of the ballast to obtain the loss coefficient of permeability of the equipment. The loss coefficient of permeability was incorporated to obtain an accurately measured permeability.

4.6 Results and Discussions

Figure 4.7 shows the variation of hydraulic conductivity with relation to the void contaminant index for coal fouled ballast and clayey fine sand fouled ballast. As expected, the overall hydraulic conductivity always decreases with an increase in the VCI. The current test results show that a 5% increase of VCI decreases the hydraulic conductivity by a factor of at least 200 and 1500 for ballast contaminated by coal and fine clayey sand, respectively. However, this reduction in permeability would not significantly affect the required minimum drainage capacity for acceptable track operations. Beyond a VCI of 75%, any further reduction in the hydraulic conductivity becomes marginal as it approaches the hydraulic conductivity of the fouling material itself. The above observations are also in line with the laboratory measurements of
sand-gravel mixtures reported by Jones (1954), whereby a high percentage of sand (greater than 35%) in gravel would provide a hydraulic conductivity close to that of the sand itself.

Figure 4.7: Variation of hydraulic conductivity vs. void contaminant index for coal fouled ballast and clayey fine sand fouled ballast

Figure 4.8 shows the variation of hydraulic conductivity for clay- fouled ballast for the case where the fouling material has been uniformly distributed. At low levels of VCI (less than 10% VCI), the overall hydraulic conductivity of ballast is relatively unaffected. Beyond VCI =90%, the overall permeability of fouled ballast is almost the same as that of kaolin.
4.7 Analytical and Empirical Models for Hydraulic Conductivity

In the past, various researchers have attempted to model the hydraulic conductivity of granular soils; e.g., Hazen (1911) and Casagrande (1937) empirical relations and Kozeny-Carman (Kozeny, 1927; Carman, 1956) analytical equation (Salem, 2001; Carrie, 2003; Costa, 2006; Yin, 2009; Courcelles et al., 2011). While these models work well for some types of granular materials such as sands and silts, for coarse-grained aggregate such as ballast having a larger and inter-connected pore structure, the change of hydraulic conductivity with respect to the porosity is usually insensitive, unless a large amount of fines are accumulated within the voids. To
represent the hydraulic conductivity \( (k) \) of a mixture of granular and fine-grained soil, Koltermann and Gorelick (1995) proposed:

\[
k = \frac{d_{fp}^2 \phi_{fp}^3}{180(1 - \phi_{fp})^2}
\]  

(4.1)

where, \( \phi_{fp} \) is the composite porosity of the mixture and \( d_{fp} \) is the representative grain diameter. The above model assumes fine particles to be uniformly distributed throughout the voids. For example, in the field, coal fouling accumulates more towards the bottom of the ballast layer, i.e., by vertical migration under rainwater percolation and vibration from the passage of trains. Therefore, to determine the equivalent permeability of ballast contaminated with non-uniformly distributed fouled material, a layer by layer simplification may need to be considered. An analytical model based on a twin layer permeability theory is considered herewith, assuming only the vertical flow (Figure 4.1) and the bottom layer has 100 % VCI of fouling.

According to Figure 4.1, the volume of ballast voids occupied by fouling material \( (V_{vb,h}) \) within the ballast layer of height \( (h) \) can be written as:

\[
V_{vb,h} = e_b V_{sb} \frac{h}{L}
\]  

(4.2)

where, \( e_b \), \( V_{sb} \) and \( L \) are the void ratio of clean ballast, solid volume of the clean ballast, and height of the overall ballast layer, respectively. The dry density of the fouling material \( (\rho_{sf}) \) can be written as:
\[ \rho_s = \frac{G_{sf}}{(1 + e_f)} \rho_w \]  

(4.3)

where, \( e_f \) and \( G_{sf} \) are the void ratio and specific gravity of the fouling material, respectively, and \( \rho_w \) is the density of water. The dry mass of fouling material \( (M_f) \) can now be written as:

\[ M_f = V_{vb,h} \rho_{sf} \]  

(4.4)

Combining Equations (4.2) and (4.3), the dry mass of fouling material, \( M_f \) can be calculated by:

\[ M_f = \frac{e_b}{1 + e_f} \times \frac{G_{sf}}{G_{s,b}} \times M_b \times \frac{h}{L} \]  

(4.5)

Assuming Darcy flow to be perpendicular to the surface, the overall hydraulic conductivity \( (k) \) at the clean and fouled ballast layers in series can be represented by:

\[ k = \frac{L}{\left( \frac{L-h}{k_b} + \frac{h}{k_f} \right)} \]  

(4.6)

In the above, \( k_b \) and \( k_f \) are the hydraulic conductivities of clean and fouled layers of ballast, respectively. Experimental data will also confirm later that when \( VCI \) is very high (greater that 90\%), the hydraulic conductivity of fouled ballast layer attains almost the same value as that of the fouling material itself (Figure 4.7). As the \( VCI \)
of the bottom layer of fouled ballast \((h)\) is 100\%, the hydraulic conductivity of that layer \(k_f\) as represented in Equation (4.6) can be assumed to be the same as that of the fouling material itself. By combining Equations (3.2 in Chapter 3), (4.5), and (4.6) the equivalent hydraulic conductivity for ballast mixed with the contaminating fines (e.g. coal) can be obtained as:

\[
k = \frac{k_b \times k_f}{k_f + \frac{VCI}{100} \times (k_b - k_f)}
\]  

(4.7)

The calculated hydraulic conductivity of fouled ballast based on Equation 10 was close to that of the coal-fouled material obtained from sites at Rockhampton (Queensland), Bellambi (NSW) and clay-fouled ballast from Sydenham (NSW). Figure 4.7 shows that computed values of hydraulic conductivity based on Equation (4.7) are in good agreement with the experimental data. Figure 4.9 shows that the proposed Equation (4.7) offers a better prediction than that by Koltermann and Gorelick (1995), who did not address the non-uniformity of fouling.
4.8 Determination of Track Drainage Capacity using a Two-Dimensional Seepage Model

As flow through a ballast track can occur in both vertical and horizontal directions, a 2-D seepage analysis was conducted using the finite element software, SEEP-W (GeoStudio, 2007a and 2007b), to determine the drainage capacity with respect to various fouling conditions. Hydraulic conductivity values corresponding to different VCI obtained from the experimental results (Figure 4.8) were used as input parameters in the analysis. For most large-size granular materials, the hydraulic conductivity of the granular assembly tends to be isotropic. This has been proven in many past studies carried out for rockfill materials (Hirschfeld, 1973). The difference in values of $k_h$ and $k_v$ for coarse aggregates is considerably less than those for fine
grained materials such as silt and clay. The pore structure for coarse granular materials along the vertical or horizontal directions is random, and therefore in this study \( k_v \) and \( k_h \) are often assumed to be same. The vertical cross section of a typical Australian track is shown in Figure 4.10a and due to symmetry, a finite element discretisation of one half track is considered in Figure 4.10-b.

Three types of boundary conditions were applied to the finite element model. While a free drainage boundary condition was used at the top of the shoulder ballast layer, impermeable layers were used at the sides.
surface, along the centreline and at the bottom of the ballast bed, an impermeable boundary was applied at the bottom of the ballast bed. A hydraulic head equal to the height of the track was assumed at the top surface for calculating the steady state discharge ($q$). The erosion of fouled materials is neglected in this simplified model.

To simulate 3 possible scenarios for track fouling, three models were carried out for clay (kaolin) fouled ballast.

Figure 4.11: Fouled ballast patterns (a) Model 1, (b) Model 2 and (c) Model 3
Model 1: Newly constructed track: The entire track is divided into three equal horizontal layers (100 mm each) and the hydraulic conductivity values corresponding to different VCI values are used (Figure 4.11a).

Model 2: Fouling track subjected to undercutting: The track is divided into two horizontal layers and the bottom ballast layer is characterised by a VCI of 100%, while the top layer contains clean ballast (Figure 4.11b). The thickness of the clean layer of ballast is varied to determine the minimum depth of clean ballast required to satisfy acceptable drainage.

Model 3: Track subjected to shoulder cleaning: The whole track is divided into 4 parts, shoulder ballast and 3 horizontal ballast layers with different values of VCI (Figure 4.11c).

Based on the experimental results as shown in Figure 4.8, the hydraulic conductivity with VCI relationship has been used for the finite element model.

4.8.1 Classification of the track drainage

Based on Pilgrim (1997) and ARTC (2006), the rainfall in Australia usually varies from 125 mm/hr to 175 mm/hr from one state to another. In this study a maximum rainfall intensity of 150 mm/hr was adopted and this would correspond to a critical flow rate ($Q_c$) of 0.0002 m$^3$/s over the unit length of the track.

From the seepage analysis, the maximum drainage capacity ($Q$) of the ballast can be determined for various levels and conditions of fouling. When the track drainage capacity is equal to or lower than what is required for a given rainfall rate, then the
fouled track is considered to be impermeable. In this context, a ratio between the computed track drainage capacity and the critical flow \( \frac{Q}{Q_c} \) is introduced as a dimensionless index to classify the drainage condition as stipulated in Table 4.3.

<table>
<thead>
<tr>
<th>Drainage classification</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free Drainage</td>
<td>( \frac{Q}{Q_c} &gt; 100 )</td>
</tr>
<tr>
<td>Good drainage</td>
<td>( 10 &lt; \frac{Q}{Q_c} &lt; 100 )</td>
</tr>
<tr>
<td>Acceptable drainage</td>
<td>( 1 &lt; \frac{Q}{Q_c} &lt; 10 )</td>
</tr>
<tr>
<td>Poor Drainage</td>
<td>( 0.1 &lt; \frac{Q}{Q_c} &lt; 1 )</td>
</tr>
<tr>
<td>very Poor</td>
<td>( 0.001 &lt; \frac{Q}{Q_c} &lt; 0.1 )</td>
</tr>
<tr>
<td>Impervious</td>
<td>( \frac{Q}{Q_c} &lt; 0.001 )</td>
</tr>
</tbody>
</table>

If the ratio \( \frac{Q}{Q_c} \) equals 1, the track becomes saturated under the given rainfall. When the ratio \( \frac{Q}{Q_c} \) is greater than 1, the track drainage is classified into various categories i.e. “Acceptable drainage”, “Good drainage”, as well as “Free drainage” and when it becomes less than 1, the drainage is classified as “Poor drainage”, “Very poor drainage”, and “Impervious” based on the output of the numerical SEEP/W analysis. It is pertinent to know that the permeability values used in the SEEP/W analysis were chosen in accordance with the drainage criteria specified by Terzaghi and Peck (1967).
4.8.2 Seepage Data Interpretation

Figure 4.12 shows a typical output of numerical analysis using SEEP-W software. Rain water percolating from the top boundary moves laterally outwards due to the impermeable boundary at the bottom. A shift in the direction of flow at the interface between clean and fully fouled ballast ($VCI = 100\%$) induces a greater travel path and thus inhibits a rapid dissipation of water.

![Figure 4.12: Typical output of numerical Seepage analysis (Model 2)](image)

Tables 4.4 to 4.6 and Figure 4.12 present the results obtained from the analysis of Models 1, 2 and 3, respectively.
Table 4.4: Track drainage classification based on Model 1

<table>
<thead>
<tr>
<th>case</th>
<th>VCI (%)</th>
<th>Q/Qc</th>
<th>Drainage classification</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Layer 1</td>
<td>Layer 2</td>
<td>Layer 3</td>
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</tr>
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</tr>
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</tr>
<tr>
<td>1-23</td>
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<td>0</td>
<td>75</td>
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</tbody>
</table>
Table 4.5: Track drainage classification based on Model 2

<table>
<thead>
<tr>
<th>Case</th>
<th>Clean ballast layer thickness</th>
<th>( Q/Q_c )</th>
<th>Drainage classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1</td>
<td>0.01</td>
<td>0.426</td>
<td>Poor Drainage</td>
</tr>
<tr>
<td>2-2</td>
<td>0.02</td>
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<td>Acceptable Drainage</td>
</tr>
<tr>
<td>2-3</td>
<td>0.025</td>
<td>3.1</td>
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</tr>
<tr>
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<td>0.03</td>
<td>3.7</td>
<td>Acceptable Drainage</td>
</tr>
<tr>
<td>2-5</td>
<td>0.05</td>
<td>7.4</td>
<td>Acceptable Drainage</td>
</tr>
<tr>
<td>2-6</td>
<td>0.1</td>
<td>20</td>
<td>Good Drainage</td>
</tr>
<tr>
<td>2-7</td>
<td>0.2</td>
<td>60</td>
<td>Good Drainage</td>
</tr>
<tr>
<td>2-8</td>
<td>0.3</td>
<td>110</td>
<td>Free Drainage</td>
</tr>
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</table>

Table 4.6: Track drainage classification based on Model 3

<table>
<thead>
<tr>
<th>Case</th>
<th>VCI (%)</th>
<th>Layer 1</th>
<th>Layer 2</th>
<th>Layer 3</th>
<th>Layer 4</th>
<th>( Q/Q_c )</th>
<th>Drainage criteria</th>
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<tbody>
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<td>3-1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>110</td>
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<td>1.7</td>
<td>Acceptable</td>
</tr>
<tr>
<td>3-3</td>
<td>50</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>92</td>
<td>Good Drainage</td>
</tr>
<tr>
<td>3-4</td>
<td>50</td>
<td>50</td>
<td>0</td>
<td>0</td>
<td>69</td>
<td>Good Drainage</td>
<td></td>
</tr>
<tr>
<td>3-5</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>0</td>
<td>0.165</td>
<td>Poor Drainage</td>
<td></td>
</tr>
<tr>
<td>3-6</td>
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<td>50</td>
<td>25</td>
<td>0</td>
<td>15</td>
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<td></td>
</tr>
<tr>
<td>3-7</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>92</td>
<td>Good Drainage</td>
<td></td>
</tr>
<tr>
<td>3-8</td>
<td>100</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>69</td>
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<td></td>
</tr>
<tr>
<td>3-9</td>
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<td>100</td>
<td>25</td>
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<td></td>
</tr>
<tr>
<td>3-10</td>
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<td>100</td>
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<td>0.0000318</td>
<td>Impervious</td>
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</tr>
<tr>
<td>3-11</td>
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<td>0</td>
<td>0</td>
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<td>50</td>
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<td>0</td>
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</tr>
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<td>3-13</td>
<td>100</td>
<td>100</td>
<td>50</td>
<td>0</td>
<td>0.113</td>
<td>Poor Drainage</td>
<td></td>
</tr>
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<td>3-14</td>
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<td>0</td>
<td>0</td>
<td>25</td>
<td>14</td>
<td>Good Drainage</td>
<td></td>
</tr>
<tr>
<td>3-15</td>
<td>25</td>
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<td></td>
</tr>
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<td>50</td>
<td>0</td>
<td>0</td>
<td>25</td>
<td>10.6</td>
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<td></td>
</tr>
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<td>3-17</td>
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<td>50</td>
<td>0</td>
<td>25</td>
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<td></td>
</tr>
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<td>25</td>
<td>25</td>
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<td>Acceptable</td>
<td></td>
</tr>
<tr>
<td>3-19</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>25</td>
<td>0.161</td>
<td>Poor Drainage</td>
<td></td>
</tr>
<tr>
<td>3-20</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>25</td>
<td>11</td>
<td>Good Drainage</td>
<td></td>
</tr>
<tr>
<td>3-21</td>
<td>100</td>
<td>100</td>
<td>0</td>
<td>25</td>
<td>7.1</td>
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<td></td>
</tr>
<tr>
<td>3-22</td>
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<td>100</td>
<td>25</td>
<td>25</td>
<td>4.6</td>
<td>Acceptable</td>
<td></td>
</tr>
<tr>
<td>3-23</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>25</td>
<td>0.0000318</td>
<td>Impervious</td>
<td></td>
</tr>
<tr>
<td>3-24</td>
<td>100</td>
<td>50</td>
<td>0</td>
<td>25</td>
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<td>3-26</td>
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<td>50</td>
<td>25</td>
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<td></td>
</tr>
<tr>
<td>3-27</td>
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<td>0</td>
<td>0</td>
<td>50</td>
<td>0.11</td>
<td>Poor Drainage</td>
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</tr>
<tr>
<td>3-28</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>50</td>
<td>0.091</td>
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<td></td>
</tr>
</tbody>
</table>
The drainage classification in Table 4.4-4.6 has been selected based on the track drainage classification in accordance with the drainage capacity criteria (Table 4.3) adopted in the current study. Based on Model 1 (Figure 4.11-a), as long as the top layer of ballast is clean, the track can be classified either as ‘free drainage’ or as ‘acceptable drainage’. In contrast, if the top layer has $VCI > 50\%$ lying on relatively clean bottom layer of ballast, then the drainage capacity can be considered to be ‘poor’.

As expected, when all the layers have $VCI > 50\%$, then the track is considered to be of ‘very poor drainage’, and thereby requires maintenance. This seepage analysis implies that it is not always mandatory to replace the entire ballast volume unless the top layer of the track is also fouled with a $VCI$ exceeding 50%. In practice, the common and convenient ballast maintenance schemes includes either cleaning the

<table>
<thead>
<tr>
<th>3-29</th>
<th>25</th>
<th>25</th>
<th>25</th>
<th>50</th>
<th>0.076</th>
<th>Very Poor</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-30</td>
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<td>Very Poor</td>
</tr>
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<td>0</td>
<td>50</td>
<td>0.077</td>
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</tr>
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<td>3-32</td>
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<td>Very Poor</td>
</tr>
<tr>
<td>3-33</td>
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<td>50</td>
<td>0.0616</td>
<td>Very Poor</td>
</tr>
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<td>3-34</td>
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<td>25</td>
<td>50</td>
<td>0.0613</td>
<td>Very Poor</td>
</tr>
<tr>
<td>3-35</td>
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<td>50</td>
<td>0.045</td>
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</tr>
<tr>
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<td>0.069</td>
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</tr>
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<td>3-37</td>
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</tr>
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<td>100</td>
<td>50</td>
<td>0.0000313</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-39</td>
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<td>0</td>
<td>50</td>
<td>0.0534</td>
<td>Very Poor</td>
</tr>
<tr>
<td>3-40</td>
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<td>100</td>
<td>50</td>
<td>50</td>
<td>0.0275</td>
<td>Very Poor</td>
</tr>
<tr>
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<td>0</td>
<td>100</td>
<td>0.0000175</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-42</td>
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<td>0</td>
<td>100</td>
<td>0.0000148</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-43</td>
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<td>0</td>
<td>100</td>
<td>0.0000175</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-44</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>100</td>
<td>0.0000175</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-45</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0.0000148</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-46</td>
<td>100</td>
<td>100</td>
<td>0</td>
<td>100</td>
<td>0.0000175</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-47</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>8.67x10^{-6}</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-48</td>
<td>100</td>
<td>50</td>
<td>0</td>
<td>100</td>
<td>0.0000148</td>
<td>Impervious</td>
</tr>
<tr>
<td>3-49</td>
<td>100</td>
<td>100</td>
<td>50</td>
<td>100</td>
<td>0.0000118</td>
<td>Impervious</td>
</tr>
</tbody>
</table>
shoulder ballast or the top ballast (under cutting), or both. This analysis clearly suggests that replacing or cleaning the ballast from the shoulder can be adequate, when the top layer of ballast has a VCI less than 50%. It can also be seen that when the VCI of the shoulder ballast exceeds 50%, it acts as a flow barrier and the drainage capacity of the track decreases significantly, to be subsequently categorised as having ‘poor drainage’. Moreover, only cleaning the shoulder ballast will be ineffective if the top layer of ballast is fouled significantly (VCI > 50%). Under these circumstances, cleaning the ballast via under cutting or total replacement by maintenance machinery should be used. This analysis also shows that as long as a clean ballast thickness of at least 100 mm is available at any time, then the overall track will have sufficient drainage.

4.9 Summary

A study of one-dimensional flow was imperative for investigating the influence of the degree of fouling on the overall hydraulic conductivity of fouled ballast. An analytical approach based on a twin layer permeability concept was proposed to predict the hydraulic conductivity of fouled ballast with a non-uniform distribution of fouling material with depth. This analytical approach was well supported by a series of constant head permeability tests carried out using a specially designed large scale permeability apparatus. The results confirmed that the hydraulic conductivity decreased with the increase in VCI and that the critical conditions in view of track maintenance would occur when VCI exceeded 50% for clay fouling. Initially, even a small increase in VCI leads to a significant decrease in the hydraulic conductivity of
the ballast, but beyond a certain limit of VCI (50% for coal and 90% for clay) the hydraulic conductivity of fouled ballast converges to that of the fouling materials itself.

Based on the hydraulic conductivity of ballast having different VCI, the drainage capacity of the track was determined using a two-dimensional, finite element seepage analysis applied to actual track geometry. It was shown that both the location and extent of fouling played an important role when assessing the overall track drainage capacity. The drainage condition of the track was proposed based on typical high rainfall intensity in Australia and the corresponding track drainage capacity. Ballast cleaning using the undercutting method is recommended when the VCI of the top 100mm of the ballast layer exceeds 50%. When the shoulder ballast is fouled to more than 50% VCI, then the cleaning or replacement of the track shoulder is also required to maintain an acceptable track drainage capacity. If the shoulder ballast is fouled to a high level (i.e. VCI > 50%), then ‘poor drainage’ can occur even if the other layers of ballast are relatively clean.

The maintenance chart (Figure 4.13) developed on the basis of current testing and analysis offers very useful guidelines for facilitating the decisions made by track engineers. The VCI and its implications have already been adopted by some rail organisations in the States of Queensland and New South Wales under the auspices of the Cooperative Research Centre for Rail Innovation (CRC-Rail). Nevertheless, the contents of this chapter have been based on a limited number of divisions within the ballast bed with several conveniently selected levels of fouling. To evaluate the track drainage capacity to a higher level of accuracy, then a more sophisticated
numerical model having a larger number of discretised ballast layers with a wider variation of corresponding *VCI* values will be required.

![Maintenance Chart](image)

**Figure 4.13: Maintenance Chart**
5 BEHAVIOUR OF CLAY FOULED BALLAST UNDER MONOTONIC LOADING

5.1 Introduction

This chapter presents a laboratory investigation of clay-fouled ballast under monotonic loading using a series of large scale triaxial tests conducted on latite basalt, a commonly used rail ballast of volcanic origin in Australia. Consolidated drained triaxial tests were conducted under three different levels of confining pressure and varying degrees of clay fouling. The stress-strain and degradation characteristics are discussed in detail. More importantly, this chapter describes the non-linear strength envelope and a novel empirical relationship to capture the detrimental effects of clay fouling on the performance of ballasted tracks. Finally critical state parameters which are essential for the constitutive modelling of clay-fouled ballast are presented and discussed.

5.2 Experimental Procedure

In order to understand the effect of clay fouling on the stress-strain and degradation behaviour of ballast for different levels of fouling (used kaolin as fouling material), a series of large scale consolidated drained triaxial tests under monotonic loading conditions were carried out at confining pressures ranging from 10-60 kPa, that are representative of field conditions (Table 5.1).
Table 5.1: Experimental programme for triaxial testing

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Void Contaminant Index, VCI(%)</th>
<th>Confining pressure, $\sigma_3$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>50</td>
<td>60</td>
</tr>
<tr>
<td>5</td>
<td>80</td>
<td></td>
</tr>
</tbody>
</table>

In typical Australian tracks, field measurements indicate that the lateral confining pressure provided by the sleepers, shoulder, and crib ballast rarely exceeds 60 kPa (Indraratna et al., 2010). Suiker et al. (2005) have indicated that a confining pressure range of 10-60 kPa would cover most in-situ conditions in European tracks. Based on track maintenance records in New South Wales, the maximum fouling levels prior to replenishing with fresh ballast have been in the proximity of $VCI @ 65\%$. Therefore, more emphasis is given to the clay-fouled ballast tested in the range of $10\% \leq VCI \leq 80\%$. However, to investigate the behaviour of clay-fouled ballast beyond this range, when all the clean ballast voids are occupied by clay fouling (i.e. at 100% VCI), an additional test series under the above confining pressures were simulated and discussed in a separate section of this chapter.

Unlike conventional geo-materials such as sands and gravel, attributed to the larger physical dimensions of ballast (whose nominal size in Australia is between 13 mm and 63 mm), the conventional triaxial apparatus cannot be used for testing. When the ratio of specimen diameter to maximum particle size exceeds, the effects of boundary
become increasingly insignificant (Marachi et al., 1972). The large scale triaxial apparatus (Figure 5.1) designed and built at the University of Wollongong ensures that the sample diameter to maximum particle size ratio exceeds 6 for ballast.

Figure 5.1: Large Scale triaxial apparatus designed and build at University of Wollongong
Figure 5.2 shows a schematic diagram of this large scale triaxial apparatus. This apparatus can accommodate a specimen 600 mm high and 300 mm in diameter. It consists of six primary components; the main cylindrical chamber that houses the specimen, a vertical loading unit (actuator) and servo controller, confining and back pressure control units, a volume change measurement device, and the digital data acquisition system. It comprises a dynamic actuator capable of applying deviator stresses up to 2.1 MPa and a maximum loading frequency of 60 Hz. The vertical strain was determined by recording the relative position of the actuator at regular intervals (permanent deformation) or during a given cycle (resilient deformation). Volumetric deformations were recorded using a ‘volume change’ device attached externally to the test rig (Figure 5.2). When the specimen is compressed, the water enters the ‘volume change’ device causing the internal piston to move up, while reverse is true for dilation. By measuring the movement of the piston with a linear variable differential transducer (LVDT), the change in volume is recorded. Both confining pressure and pore-water pressure (mid and bottom of the specimen) variations were recorded using transducers. Further details of the components and measuring techniques of this test apparatus can be found elsewhere (Indraratna, 1996; Indraratna et al., 1998).
Figure 5.2: Schematic illustration of large scale triaxial chamber (modified after Indraratna et al., 1998)
A 7mm thick cylindrical rubber membrane is used to prepare and confine the specimen. Given the nature of sharp angular aggregates, the 7 mm thick membrane was enough to prevent puncture during the preparation and testing of the specimen (both 3 mm and 5mm thick membranes punctured previously during shearing). The Young’s modulus of the membrane was determined to be 4300 kPa following the method described by ASTM D4767-04 and Bishop and Henkel (1962). The pore water pressure can be measured using miniature transducers inserted at the mid plane of the specimen. Consolidated drained conditions ensured that the excess pore water pressure recorded was always insignificant during testing.

The geotechnical properties of ballast change with its initial density (or void ratio), as does the particle size distribution ($PSD$) and degree of saturation (Indraratna et al., 1998, 2011, Suiker et al., 2005). The initial $PSD$, density, and void ratio of ballast were kept almost identical in all specimens to capture realistic track conditions. Fresh samples of ballast were obtained from Bombo quarry (near the city of Wollongong). It has been pointed out in earlier studies that as the sample size ratio (i.e. diameter of the test specimen to the maximum particle size) exceeds 6, the influence of boundary or size effects become increasingly insignificant (Marachi et al., 1972, Indraratna et al., 1993).

5.2.1 Material and Particle Size Distribution

Latite basalt is frequently used for Rail Track in New South Wales, Australia, and its physical and durable attributes are shown in Table 5.2. Latite basalt is a dark coloured volcanic (igneous) rock containing the main minerals of feldspar,
plagioclase, and augite. The particle size distribution (PSD) of the clean tested ballast used in this study is plotted in Figure 5.3, together with the recommended Australian Standards (AS 2758.7, 1996), where the maximum grain size falls between 50 to 60 mm. Commercial kaolin clay was used as fouling material, and its properties are given in Table 5.3.

Table 5.2: Characteristics of fresh ballast (after Indraratna et al., 1998)

<table>
<thead>
<tr>
<th>Characteristics test</th>
<th>Test result</th>
<th>Recommendations by Australian Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Durability</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aggregate crushing value</td>
<td>12%</td>
<td>&lt; 25%</td>
</tr>
<tr>
<td>Los Angeles Abrasion</td>
<td>15%</td>
<td>&lt; 25%</td>
</tr>
<tr>
<td>Wet attrition value</td>
<td>8%</td>
<td>&lt; 6%</td>
</tr>
<tr>
<td><strong>Strength</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pont load index</td>
<td>5.39 MPa</td>
<td>-</td>
</tr>
<tr>
<td><strong>Shape</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flakiness</td>
<td>25%</td>
<td>&lt; 30%</td>
</tr>
<tr>
<td>Misshapen particles</td>
<td>20%</td>
<td>&lt; 30%</td>
</tr>
</tbody>
</table>
Figure 5.3: Particle size distribution of test materials

Commercial kaolin was used to simulate clay fouling, and its properties are given in Table 5.3.

Table 5.3: Properties of test material

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity of the ballast, $G_{sb}$</td>
<td>2.75</td>
</tr>
<tr>
<td>Specific Gravity of the kaolin, $G_{sf}$</td>
<td>2.51</td>
</tr>
<tr>
<td>Bulk Unit Weight of the clean ballast, kN/m$^3$</td>
<td>14.3</td>
</tr>
<tr>
<td>Initial void ratio of clean ballast</td>
<td>0.88</td>
</tr>
<tr>
<td>Bulk unit weight of fouling material (kaolin), kN/m$^3$</td>
<td>8.88</td>
</tr>
<tr>
<td>Initial Void ratio of fouling material (kaolin)</td>
<td>1.75</td>
</tr>
<tr>
<td>Plastic limit of kaolin (%)</td>
<td>26.4</td>
</tr>
<tr>
<td>Liquid Limit of kaolin (%)</td>
<td>52.1</td>
</tr>
<tr>
<td>Uniaxial compressive strength ($\sigma_c$) of the parent rock, MPa</td>
<td>130</td>
</tr>
</tbody>
</table>
5.2.2 Specimen preparation

Three methods were used to prepare specimens for different levels of VCI, and all the ballast specimens were sieved and thoroughly mixed to ensure consistency. Figure 5.4 shows a schematic diagram of fouling material filling the ballast voids.

Method 1: Preparation of clean ballast specimens

A specimen of clean ballast was divided into four equal portions, and then each portion was then placed inside the chamber (Figure 5.5-a) and compacted with a vibrating plate to a height of 150mm. A rubber pad underneath the vibrating plate was used to prevent particle breakage during placement. All the layers were placed and compacted until a final height of 600mm was attained. The specimen was then saturated from the base upwards.

Method 2: Clay fouled ballast with $10\% \leq VCI \leq 80\%$

The amount of clay needed for a predetermined VCI was calculated for each test specimen. Then a quarter of the clay mixture was mixed with a quarter portion of ballast using the concrete mixer, and then placed inside the upright cylindrical membrane (Figure 5.5-b & c). The moisture content of clay during mixing was slightly more than the liquid limit of 52%. A vibrating plate was used to compact the specimen following the sequential procedure previously explained for the subsequent layers. A visual inspection was conducted to ensure that the fouling material covered the entire surface area of the ballast during and after mixing in the concrete mixer.
Method 3: Clay fouled ballast with 100% VCI;

Similar to Method 1, a sample of fresh ballast was divided to four equal portions and saturated. The clay was then placed in the chamber first and any entrapped air was removed using mild vibration. Next, a quarter of the fresh ballast was placed above the clay and compacted with the vibrating plate to a height of 150mm. Any excessive clay leaching to the top was then removed. This process ensured that all the voids in the ballast were completely filled. The other layers were also compacted sequentially to a final height of 600mm (Figure 5.5-d).

Figure 5.4: Schematic diagram of (a) clean, (b) partially fouled and (c) totally fouled ballast
Figure 5.5: Sample preparation methods: (a) 7mm thick membrane supported by two split moulds; (b) Ballast is mixed with kaolin slurry; (c) First layer of fouled ballast inside the membrane; (d) Last layer for the 100% VCI fouled ballast; (e) At the end of sample preparation

The effect of the membrane was considered and corrected in two stages (a) during specimen preparation and, (b) during specimen testing. During specimen preparation, a mild vacuum suction (4-6 kPa), depending on the VCI, was applied between the split mould and the membrane. This small suction was sufficient for the membrane to fully attach to the split mould, without transferring any significant confinement to the ballast specimen. The resulting circumferential strain of the membrane was determined to be @ 1.5-2.0%. Before removing the split mould, an internal suction of up to 5 kPa was applied to the interior of the ballast specimen through the vacuum/drainage valve, as shown in Figure 5.2, to ensure that the ballast assembly...
was stable, i.e., insignificant deformation from the originally prepared dimensions of 300mm in diameter and 600mm in height. The split cell was removed within 10-15 minutes after the specimen had been prepared.

After preparing the test specimen, the outer cell chamber was placed in position and connected to the axial loading actuator (Figure 5.5-e and Figure 5.1). An internal suction was maintained and then removed once the cell pressure and back pressure applied in tandem exceeded 5 kPa. For all the tests, a back pressure of 80 kPa was applied to obtain sample saturation with Skempton’s B value approaching unity (B > 0.98). Isotropic consolidation under net confining pressures of 10, 30, and 60 kPa (Table 5.1) incorporates the initial back pressure (i.e. applied cell fluid pressure minus back pressure). Depending on the value of the VCI, saturation of the fouled ballast could take up to several hours, whereas for fresh ballast the value of B usually approached 0.99 within 20 minutes. A fully drained condition was ensured by leaving the drain valves open and applying a sufficiently low axial shearing rate of 5.5 mm/min.

During testing, the required membrane correction was carried out according to ASTM D4767-04 plus other contributions from La Rochelle (1988), Henkel and Gilbert (1952), and Duncan and Seed (1967). Considering the specimen strains, the deviator stress was corrected using the membrane properties and dimensions based on the elastic theory capturing hoop stress correction. The relatively small stress distribution attributed to the weight of the specimen was considered in the effective stresses. The volumetric strain was measured using a piston within the volume change device that responded to water entering the (dilating) specimen or leaving
the(compressing) specimen, and a linear variable differential transducer (LVDT) recording the movement of the piston (Indraratna et al., 1997). The LVDT had an accuracy of 0.1 mm, which was equivalent to 0.0083% volumetric strain. A visual inspection after testing revealed that the ballast aggregates remained fully coated with clay. A sieve analysis was carried out to measure the extent of particle breakage after each test.

5.3 Results and Discussion

5.3.1 Stress-Strain Response

Figures 5.6 to 5.10 show the stress-strain and volume change behaviour during isotropically consolidated drained tests at different confining pressures for varying degrees of fouling (0 to 80% VCI). As expected, the peak deviator stress \( q = \sigma_1' - \sigma_3' \) increases with an increase in the confining pressure for all specimens. The post peak response of ballast is characterised by the strain-softening behaviour during testing.
Figure 5.6: Stress-strain and volume change behaviour during isotropically consolidated drained tests at different confining pressures on 0% VCI (Fresh ballast)
Figure 5.7: Stress-strain and volume change behaviour during isotropically consolidated drained tests at different confining pressures on 10% VCI
Figure 5.8: Stress-strain and volume change behaviour during isotropically consolidated drained tests at different confining pressures on 25% VCI.
Figure 5.9: Stress-strain and volume change behaviour during isotropically consolidated drained tests at different confining pressures on 50% VCI
Figure 5.10: Stress-strain and volume change behaviour during isotropically consolidated drained tests at different confining pressures on 80% VCI

In order to compare the stress-strain behaviour of fouled ballast for different VCI, they are re-plotted, as shown in Figure 5.11. The upper plots of Figure 5.11 illustrate the stress-strain behaviour of fouled ballast (VCI = 10-80%) in contrast to fresh
ballast \((VCI = 0\%\) at an increasing confining pressure. The stress-strain curves for clean ballast are similar to the various samples of clean ballast reported in earlier studies. Several plots for other aggregates, including greywacke rockfills, and Bombo and Chullora basalts, are given by Indraratna et al., (2011). As expected, when the confining pressure \((\sigma_3')\) is increased from 10 to 60 kPa, the peak deviator stress and initial deformation modulus increased for all levels of fouling. As expected, when the degree of fouling increases the peak deviator stress decreases significantly. However, beyond \(VCI = 25\%,\) the subsequent decrease in the peak deviator stress becomes gradual, compared to fresh ballast, and for \(VCI = 10\%.\) Moreover, the stress-strain curves of fouled ballast for \(VCI > 25\%\) do not indicate a distinctly prominent strain-softening behaviour whereby significant clay fouling \((VCI > 25\%)\) shows an increasingly more ductile post-peak response.

The stress-strain plots in Figure 5.11 are plotted to axial strains @ 28% (limit of actuator). Fresh ballast always shows a higher deviator stress than fouled ballast at any given axial strain, but this difference tends to decrease at high axial strains. If the tests could be continued to larger axial strains considerably beyond the current limit of the equipment, pronounced particle degradation and re-compaction may occur, possibly approaching a critical state friction angle.

The bottom plots of Figure 5.11 show the changes in volumetric strain with the axial strain for varying levels of fouling and increasing confining pressure. The measured data is best interpreted by looking at the compression zone and dilation zone separately. In the compression zone (plotted as positive values), an increasing \(VCI\) generally shows a reduced compression of the fouled specimen as the voids between
the ballast grains are now occupied by clay acting as a void filler. Nevertheless, in
the case of $VCI = 10\%$ all three specimens (at $\sigma^\prime = 10$, 30, 60 kPa) indicated a slight
increase in compression compared to their fresh ballast counterparts. This may be
attributed to the small amount of clay that is coating the ballast grains acting as a
lubricant, and thereby facilitating the specimens to attain a slightly higher
compression. With respect to dilation, the highly fouled specimens showed a
decrease in the rate and magnitude of dilation at axial strains exceeding @ 20%,
while an increase in $\sigma^\prime$ from 10 to 60 kPa significantly suppressed dilation in all the
specimens. It may be argued intuitively that the addition of kaolin in sufficient
quantities may have contributed to a ‘binding’ effect that impeded the tendency of
the aggregates to dilate. Moreover, the specimens that are highly fouled ($VCI = 50\%$
and 80%) began to dilate swiftly at a lower axial strain after an initially reduced
compression compared to clean ballast.
Figure 5.11: Stress-strain behaviour of clean and fouled ballast during isotropically consolidated drained tests at confining pressures of (a) 10kPa, (b) 30kPa and (c) 60kPa, respectively.
Figure 5.12 illustrates that the peak dilation rate of all the ballast specimens decreases with an increasing Voids Contaminant Index (VCI). It is clear that as the VCI increases beyond 40% or so, the corresponding peak dilation rate remains approximately constant at all values of $\sigma_3'$. In practice, track instability usually occurs due to an excessive lateral deformation of the ballast as a result of low confining pressure (Selig and Waters, 1994; Indraratna et al., 2011). Increased confining pressure decreases dilation that makes the ballast assembly more stable (Suiker et al., 2005). If dilation during shearing can be minimised by providing sufficient field restraints (Lackenby et al., 2007), the overall stability of track can be significantly improved. From a practical point of view, Figure 5.12 implies that a small amount of clay fouling ($VCI = 10-20\%$) can be beneficial in reducing the high rate of dilation of clean ballast, especially at small confining pressures, although excessive fouling would naturally pose unfavourable implications on track drainage, as well as reducing the shear strength.
By plotting the peak deviator stress with VCI (Figure 5.13), the following relationship that represents the normalised shear strength of fouled ballast may be used in a preliminary assessment of track conditions in view of track maintenance:

\[
\frac{q_{\text{peak},f}}{q_{\text{peak},b}} = \frac{1}{1 + \beta \sqrt{\text{VCI}}} \quad (3)
\]

where \( q_{\text{peak},b} \) and \( q_{\text{peak},f} \) are the peak deviator stresses for fresh and fouled ballast respectively, \( \beta \) is an empirical parameter whose magnitudes are given in Table 5.4 for varying values of \( \sigma_3' \).
Figure 5.13: Variation of peak deviator stress ($q_{\text{peak,f}}$) with Void Contaminant Index for clay fouled ballast

Table 5.4: Values of $\beta$ based on Equation (3)

<table>
<thead>
<tr>
<th>Confining Pressure, $\sigma'_3$, kPa</th>
<th>Peak deviator stress $q_{\text{peak,b}}$, kPa</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>280</td>
<td>0.094</td>
</tr>
<tr>
<td>30</td>
<td>340</td>
<td>0.047</td>
</tr>
<tr>
<td>60</td>
<td>470</td>
<td>0.050</td>
</tr>
</tbody>
</table>
5.3.2 Shear Strength Envelopes

Figure 5.14 shows Mohr-Coulomb diagrams of the specimens of clay fouled ballast which exhibit a non-linear shear strength envelope similar to other rockfills (Indraratna et al., 1993; 1998). The apparent peak friction angle ($\phi_p$) corresponding to a given normal stress is readily determined by the gradient of the envelope (i.e. $\tan^{-1} d\tau/d\sigma'$). Figure 5.14-d shows the peak friction angle versus effective confining pressures where the peak friction angle rapidly decreases when the effective confining pressures are in the range of 10-35kPa. When VCI increases, the peak friction angle also decreases. In Figure 5.14-d, the peak friction angle for clean ballast at 10 kPa confinement is 59°, which is similar to the findings of Aursudkij et al. (2009), and Raymond (1978).
Figure 5.14: Mohr-Coulomb strength envelopes for clay fouled ballast and variation of peak friction angles with effective confining pressure

Figure 5.15-a plots the relationship between the peak friction angle ($\phi'_p$) and the normal stress ($\sigma'_n$) for VCI varying from 0 to 80%. Here the normal stress was determined on the plane where the mobilised friction angle is at its maximum and as expected, $\phi'_p$ decreases with an increasing $\sigma'_n$. Figure 5.15-b illustrates the typical variation of $\phi'_p$ with VCI at a relatively low normal stress of 30 kPa. In general, for any given $\sigma'_n$, the $\phi'_p$ is expected to decrease with an increase in VCI, but at high normal stress ($\sigma'_n > 200$ kPa), irrespective of the value of VCI, $\phi'_p$ converges to a value close to $30^0$. This implies that at high normal stresses, the role of soft plastic
clay is effectively suppressed as the coarse aggregates are forced to be in contact despite the voids being filled with clay.

Figure 5.15: Apparent peak friction angle ($\phi'_p$) with normal stress ($\sigma'_n$) for clay fouled ballast with: (a) different values of VCI and (b) the typical variation of $\phi'_p$ with VCI at normal stress of 30 kPa.
Figure 5.16 shows the normalised Mohr-Coulomb peak strength envelopes for clean and clay fouled ballast. Here, the shear stress ($\tau$) and the effective normal stress ($\sigma_n'$) are represented as dimensionless quantities, by dividing them with the uniaxial compressive strength ($\sigma_c$) of the parent rock. The significance of the UCS, as a normalizing parameter was recognized by Hoek & Brown (1980) for rock samples tested in triaxial compression, and this technique allows an evaluation of the strength characteristics appropriate for both intact and fractured rocks. Rockfill can be regarded as intensely fractured rock, and further breakage of individual fragments during shearing is a function of particle angularity, confining pressure and the point load index related to UCS of the parent rock. This was further explained by Indraratna (1990) and Indraratna et al. (1993). The value of $\sigma_c$ is 130 MPa based on the point load test (Indraratna et al., 1998). It is clear that the normalised shear strength ($\tau/\sigma_c$) decreases with an increasing VCI.
These dimensionless strength envelopes at all levels of fouling can be expressed by a non-linear equation proposed earlier by Indraratna et al. (1998):

\[
\frac{\tau_f}{\sigma_c} = m \left( \frac{\sigma_n}{\sigma_c} \right)^n
\]  

(4)

where \( m \) and \( n \) are empirical coefficients which vary with VCI. Figure 5.16 indicates that beyond 50% VCI, the subsequent drop in the shear strength envelope is marginal. The values of \( m \) and \( n \) in Equation (4) are established by best fit (regression) analysis with the following expressions:

\[
m = 0.07[1 + \tanh(VCI/21.5)]
\]  

(5)
With an increase in $VCI$ beyond 50%, the normalised shear strength envelopes become increasingly less curved as $m$ and $n$ converge to 0.14 and 0.72 respectively (see Figure 5.17).

![Empirical strength parameters $m$ and $n$ for clay fouled ballast at varying values of $VCI$](image)

Figure 5.17: Empirical strength parameters $m$ and $n$ for clay fouled ballast at varying values of $VCI$

5.3.3 Ballast Breakage and the effect of clay fouling

Particle breakage affects the shear strength and deformation characteristics of ballast, which in turn influences track stability (Raymond et al., 1976, Selig and Waters, 1994, Indraratna et al., 2011). Owing to the angular nature of quarried aggregates, ballast degradation is primarily attributed to breakage of the sharp corners or attrition of the asperities, while particle splitting across the body of grains may occur at high confining stresses (Indraratna et al., 1998, Lackenby et al., 2007). By sieving the
ballast before and after testing, the extent of breakage can be evaluated by quantifying the differences between the particle size distribution curves.

The extent of ballast breakage was analysed by carrying out pre-test and post-test PSD analyses. In this study the Ballast Breakage Index (BBI) was evaluated using the relation given below (Indraratna et al., 2005):

\[
BBI = \frac{A}{A+B}
\]

where \(A\) is the area between the PSD curves before and after testing, and \(B\) is the area between the arbitrary boundary of maximum breakage and the final PSD, as illustrated in Figure 2.5 (Chapter 2).

Intuitively, it seems that clay would have enough cushioning effect to thwart the harsh attrition between the rough and angular particles, thereby reducing the high contact stresses and associated degradation. Figure 5.18 illustrates the variation of \(BBI\) with \(VCI\), which confirms that the ballast particles experience less breakage at higher values of \(VCI\). Nevertheless, the benefits of reduced ballast degradation are offset by the drop in shear strength as the degree of fouling increases.
5.4 Behaviour of Clay Fouled Ballast under Partially-Drained Conditions

In this section the stress-strain and degradation of clay fouled ballast under partially drained conditions is discussed. In order to investigate the behaviour of clay-fouled ballast when the voids were completely fouled with clay (i.e. 100% VCI), a series of large-scale triaxial tests were conducted under a constant total confining pressure of 10, 30, and 60kPa. The specimens were saturated and consolidated under these confining pressures, as explained in section 5.2.1, and the tests were carried out while the drainage valves remained open (Figure 5.2).
Figure 5.19 shows the stress-strain, excess pore water pressure, and change in volume during isotropically consolidated drained tests at different initial effective confining pressures on totally fouled ballast (i.e. 100% VCI). As expected, the peak deviator stress $q$ increases with an increasing confining pressure. The dilation of clay-fouled ballast seems to reduce with an increase in confining pressure because the increased confining pressure holds the particles together. Also the maximum compression values increase with the increase in confining pressure. A consolidated drained test was carried out at the same strain rate (i.e. drainage valves were kept open during shearing). However, significant excess pore water pressure still developed during shearing as ballast completely fouled by clay had a very low permeability. Therefore this test could be classified as a consolidated partially drained test.

As shown in Figure 5.19 initially, the pore water pressure was raised to a certain value, but then it decreased to a negative value (suction) followed by a further reduction in suction. If the fully drained condition could somehow be achieved, the sample would then have undergone compression first, followed by dilation. However, because the drainage conditions were poor, the pore pressure increased to a level where maximum compression occurred. Subsequently, with increasing dilation, the excess pore water pressure decreased to a negative value (suction), which occurred due to the sample dilating along with a very low permeability that prevented water from immediately entering the sample. However, as the test proceeded, water slowly penetrated into the sample from the drainage valve thereby reducing this suction with time.
On similar lines, the development of excess pore pressure ($\Delta U$) for the other range of confining pressures can be explained. However, as the confining pressure is increased, the location of maximum suction moved to the left corresponding to the applied confining pressure (Figure 5.19). As the confining pressure increased, the maximum pore pressure also increased. Hence, it took longer for the pore pressure which developed in the initial stage, to dissipate. Moreover, the fact that the starting point of dilation moved to the left as a result of the increasing confining pressure, is another reason why the location of maximum suction shifted.
Figure 5.19: Stress-strain, excess pore water pressure, and change in volume during isotropically consolidated drained tests at different initial confining pressures ($p_0$) on 100% VCI fouled ballast
5.5 Comparison of Plastic and Non-Plastic Fine Fouled Ballast under Triaxial Conditions

An additional test has been conducted for 80% VCI beach sand-fouled (non-plastic fines) specimen under consolidated drained condition at 60kPa confining pressure, in order to see whether there is a different stress-strain behaviour than plastic fine (clay) fouled ballast.

Figure 5.20 shows the stress-strain response and the change in volume during isotropically consolidated drained tests of fresh ballast, sand-fouled ballast (80% VCI), and clay-fouled ballast (80% VCI) at a confining pressure of 60kPa. In contrast to the clay-fouled ballast, sand-fouled ballast had a higher deviator stress at all levels of strain. The bottom plot shows that the specimen fouled with sand underwent less initial compression than the fresh and clay-fouled specimen, and then dilated faster than the fresh ballast. The specimen fouled with sand began to dilate very early compared to the fresh ballast, whereas the specimen fouled with clay began to dilate much later.
Figure 5.20: Stress-strain, and change in volume during isotropically consolidated drained tests for fresh ballast, 80% VCI fouled ballast by sand, and clay at 60kPa confining pressure.
5.6 Critical State

The variation of deviator stress \( (q) \) with the mean effective stress \( (p') \), for fresh and fouled ballast (10% to 80% VCI) under triaxial drained testing are shown in Figure 5.21. As expected an increase in confining pressure increases the mean effective stress that is associated with a higher deviator stress. The figure also shows that at the end of the drained shearing tests the state of stresses \( (p', q) \) of all the ballast specimens corresponding to various confining pressures, lie approximately on a straight line. [Please note that the linear portion of CSL plotted here will not pass through the origin. This is because the CSL of ballast is highly non-linear at very low confining pressure (Indraratna et al., 1998; Indraratna et al., 2011). However, the line still should pass through the origin having non-linear portion at very low confining pressure (<10kPa)]. In other words, irrespective of the magnitude of confining pressures, the stress state of ballast moves during triaxial shearing towards unique (i.e. critical) states that can be represented linearly on the \( p'-q \) plane. The slopes of the critical state lines \( (M) \) for fresh and fouled ballast are shown in Table 5.5.

Table 5.5: Slopes of the critical state lines \( (M) \) for fresh and fouled ballast

<table>
<thead>
<tr>
<th>Void Contaminant Index (VCI), %</th>
<th>Slope of the critical state line (M)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.90</td>
</tr>
<tr>
<td>10</td>
<td>1.71</td>
</tr>
<tr>
<td>25</td>
<td>1.59</td>
</tr>
<tr>
<td>50</td>
<td>1.52</td>
</tr>
<tr>
<td>80</td>
<td>1.50</td>
</tr>
</tbody>
</table>
Figure 5.21: Variation of $p'$ and $q$ in drained triaxial shearing, (a) fresh ballast, and (b) 10% VCI fouled ballast (c) 25% VCI fouled ballast (d) 50% VCI fouled ballast (e) 80% VCI fouled ballast
The variations in the void ratio \((e)\) with the mean effective stress \((p')\), during drained shearing were plotted on a semi-logarithmic scale, as shown in Figure 5.22 (a) to (e) for the fresh and fouled ballast (10% to 80% VCI), respectively. These figures show that in drained shearing, the void ratio of fresh and fouled ballast changes such that the state of the specimens at high levels of shear strain relate to each other in a specific way. As explained earlier, irrespective of the confining stresses, all the ballast specimens move towards the critical state. The critical state lines for fresh and fouled ballast shown in Figure 5.22, also show the slopes of the critical state lines in \(e\)-\(\ln p'\) plane \((\lambda)\) for the fresh and fouled ballast.

Table 5.6: \(\Gamma\) and \(\lambda\) for fresh and fouled ballast

<table>
<thead>
<tr>
<th>VCI, %</th>
<th>(\Gamma)</th>
<th>(\lambda)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.901</td>
<td>0.1810</td>
</tr>
<tr>
<td>10</td>
<td>2.082</td>
<td>0.0579</td>
</tr>
<tr>
<td>25</td>
<td>2.062</td>
<td>0.0564</td>
</tr>
<tr>
<td>50</td>
<td>1.901</td>
<td>0.0561</td>
</tr>
<tr>
<td>80</td>
<td>1.681</td>
<td>0.0388</td>
</tr>
</tbody>
</table>
Figure 5.22: Variation of void ratio in drained shearing, (a) fresh ballast, and (b) 10% VCI fouled ballast (c) 25% VCI fouled ballast (d) 50% VCI fouled ballast (e) 80% VCI fouled ballast
5.7 Practical Implications

From the results reported earlier, it is clear that clay fouling has significant implications on the performance of railway ballast. At low effective confining pressures ($\sigma_3' = 10-30$ kPa: typical of the track environment in Australia), a relatively small amount of clay fouling (VCI of 10-20%) may improve track longevity somewhat through reduced particle degradation, however excessive fouling seriously affects the shear strength of ballast. Excessive fouling exceeding 50% VCI may also threaten the ‘free draining’ property of ballast (see Chapter 4).

The peak friction angle ($\phi_p'$) is the most important design parameter for ballast and it is influenced by the amount of fouling (VCI) and the applied normal stress ($\sigma_n'$). The Australian rail industry has been using the peak friction angle of ballast mainly obtained from direct shear tests that are often conducted on reduced particle sizes ($d_{100} @ 40-50$ mm), due to the capacity of conventional shear equipment. The range of peak friction angles used in design varies from $45^\circ$-$60^\circ$ depending on the ballast gradation and the test method used. Broadly graded particle size distributions tested using large-scale shear equipment give a higher friction angle than the highly uniform gradations stipulated in the Australian Standards (Indraratna et al., 2011).

The Authors recommend that where possible, the friction angle for design should be determined using large scale triaxial tests @ 300 mm diameter samples, thereby eliminating the need for reducing the particle sizes. Also, the range of confining pressure used in shear testing kept relatively low (10-70 kPa), as most rail tracks
have measured internal confining pressures between 15-60 kPa. At relatively high confining pressures, there was a diminishing friction angle apart from increased particle breakage (Lackenby et al., 2007), while at very low confining pressures (<15 kPa), the apparent friction angle of ballast tends to be maximum after exceeding 60°, approaching the natural angle of repose.

In low lying estuarine plains where ‘clay pumping’ may occur under wet conditions, the risk of clay fouling needs to be captured in design. A past study (Indraratna et al., 2010) indicated that track maintenance would be required as the VCI > 40%. Track performance may be substantially compromised when VCI >50%, and where undrained conditions prevail for VCI > 60%. Under these circumstances, a corresponding friction angle that is significantly lower than the fresh ballast should be considered in design, unless additional geotextile and/or sub-ballast designed as a filter is placed beneath the ballast bed to prevent fouling by intruding subgrade fines. For a given gradation of ballast, if the appropriate reduced friction angle is not carefully selected on the basis of the anticipated levels of fouling, this may over predict the bearing capacity and stability of the track. Therefore, an accurate assessment of the stress, strain, and degradation characteristics of fouled ballast is highly beneficial for carrying out better maintenance and operating schemes for existing tracks, and for the preliminary design of rehabilitated tracks.
5.8 Summery

Rail ballast becomes contaminated or fouled due to the infiltration of subgrade fines such as clay. Excessive fouling of ballast may cause differential settlements and rapid deterioration of the track, demanding regular maintenance. Therefore, a proper understanding of the stress-strain, and degradation characteristics of fouled ballast is pertinent for efficient maintenance and operation of tracks. In this chapter, a series of isotropically consolidated drained triaxial tests using a large scale cylindrical triaxial apparatus were conducted on clay fouled ballast. It was observed that as the fouling increased, the shear strength of the ballast decreased, which at a high normal stress ($\sigma'_n > 200$ kPa) remained relatively unaffected, irrespective of the VCI.

The overall volumetric response of these fouled specimens was initially compressive, but depending on the VCI, subsequent dilation bears a significant contrast to clean ballast. It was observed that excessive fouling ($VCI > 50\%$) decreased both the rate and magnitude of dilation at high axial strain, while an increase in the confining pressure $\sigma'_3$ from 10 to 60 kPa, suppressed dilation considerably. A relatively small amount of clay fouling ($VCI \leq 10\%$) may slightly increase the initial rate of compression, and this can be attributed to the ‘clay coated’ aggregates providing a lubrication effect at smaller axial strains.

Based on the laboratory findings, a novel empirical relationship between the peak deviator stress and VCI was proposed, with the aim of assisting the practitioner in preliminary track assessment. A non-linear shear strength envelope for fouled ballast was introduced in a non-dimensional form, where the relevant shear strength
coefficients could be conveniently evaluated as a function of $VCI$, based on the proposed empirical equations. With the increase in fouling, the ballast particles experienced less breakage, a situation attributed to the ‘cushioning’ effect of clay that would reduce the high internal contact stresses and associated attrition of rock particles. At a high $VCI > 50\%$, although breakage was reduced, a considerable drop in shear strength could also affect the stability of track and its carrying capacity. In the future, the more complex aspects of clay fouling needs to be carefully assessed through rigorous micro-mechanical studies to more accurately predict the longevity and performance of ballasted tracks.

The test series further confirmed that at unacceptable level of fouling ($>80\%$ $VCI$), excess pore water pressure could be induced in the clay-fouled ballast with a decrease in the effective confining pressure. At higher axial strains particle dilation would also occur, measuring small value of suction within the clay-fouled voids. Different stress and strain behaviour was also observed through additional tests conducted on sand-fouled ballast i.e. beach sand (non–plastic fines), as a fouling material. Based on these tests, critical state parameters were derived and would be beneficial in the formulation of a constitutive model for the clay-fouled ballast as described in Chapter 7.
6 BEHAVIOUR OF CLAY FOULED BALLAST UNDER CYCLIC LOADING

6.1 Introduction

A series of large scale cyclic drained triaxial tests has been conducted to investigate the effects of fouling, along with a number of loading cycles on settlement and ballast degradation. Details regarding the large scale equipment, testing material, specimen preparation methods are discussed in Chapter 5, hence, not repeated here. In this chapter the laboratory testing procedure and the test results and interpretations are presented.

The primary objectives of this investigation are the assessment of the effects of fouling on the permanent strain (axial and volumetric) and also the resilient strain as a function of the number of loading cycles N. The effect of fouling on particle breakage is also investigated. The results of the static triaxial tests of latite basalt discussed in Chapter 5 form the basis of a comparison between the static and cyclic loading response.

6.2 Experimental Procedure

A series of large scale triaxial tests under a confining pressure of 10 kPa has been conducted under different ranges of fouling (0, 10, 25, 50 and 80% VCI-(used kaolin as fouling material)). The samples were prepared and consolidated in the same manner described in Chapter 5.
After preparing the test specimen, the outer cell chamber was placed and connected to the axial loading actuator. The specimen was fully saturated by applying a back pressure of 100 kPa, followed by isotropic consolidation under a confining pressure ($\sigma_3$) of 10 kPa. During the application of $\sigma_3'$, changes in the specimen volume were recorded, with typical changes in volumetric strain ($\varepsilon_v$) during consolidation ranging from 0.01 to 0.02%.

6.2.1 Cyclic deviatoric load

The cyclic load was varied between the maximum deviatoric stress ($q_{\text{max}}$) and the minimum deviatoric stress ($q_{\text{min}}$). The value of $q_{\text{min}}$ was set equal to 45 kPa which represents the state of the unloaded track, such as the weight of the sleepers and rails (Lackenby et al., 2007, Salim, 2004). As expressed in Chapter 2, determining the magnitude of ‘typical’ sleeper-ballast contact pressure resulting from a ‘standard’ train axle load relies on some simplifications regarding the impact and velocity forces. $q_{\text{max}} = 230$ kPa was appropriate to represent a 25 tonne axle load with no vertical excitation ($\varphi' = 0$), based on a number of simplifying criteria. Figure 6.1 represents the cyclic loading applied during the test series.
6.2.2 Testing

An initial static load was applied to each specimen at a rate of 1 mm/sec to a stress equal to the cyclic mean deviator stress (Figure 6.1). A loading frequency of 20 Hz was used to simulate a train speed of 145 km/h, assuming an axle wheel spacing of 2.02 m. However, a reduced frequency conditioning phase (less than 5 Hz) was required at the commencement of cyclic loading (during rapid vertical deformation) to prevent any impact loading and loss of actuator contact with the specimen. Permanent deformations were recorded at regular intervals, whilst bursts of data (sampling frequency = 588 Hz) were recorded at specific cycles to determine the resilient modulus. Testing was continued for 500,000 cycles or until the vertical
deformation approached 28% axial strain (i.e. the limit of the actuator displacement). Sieving was carried out at the completion of each test to evaluate the extent of breakage. Membrane corrections were calculated using the ASTM (2002) procedure and by assuming an axial strain of 8.5% and a 7 mm thick membrane with an elastic stiffness of 4300 kPa. This resulted in a 35 kPa correction being required in the deviator stress.

6.3 Data Analysis Methods

6.3.1 Permanent Strain (Axial and Volumetric)

Permanent axial deformation was measured using a position transducer within the actuator, lined with a data logger. Whenever the actuator moved, it never lost contact with the specimen (unloads), and therefore this downward movement represents the deformation of the specimen.

A volume change device was used to measure the volumetric strain of the specimen. A piston inside the device moves up and down whenever water enters (dilating) or leaves (compresses) the specimen, and an LVDT and data logger records the movement of the piston. A piston movement of 1 mm corresponds to a 34.6 cm³ loss (or gain) in the specimen’s volume. The displacement of water due to movement of the actuator (1 mm of movement corresponds to 8 cm³ of water displacement) was corrected when calculating the volumetric strain. The radial strain $\varepsilon_r$ and shear strain
$\varepsilon_q$ can be calculated using equations 6.1 and 6.2, respectively, where $\varepsilon_a$ and $\varepsilon_v$ are the axial strain and volumetric strain (positive in compression), respectively.

$$
\varepsilon_t = \frac{(\varepsilon_v - \varepsilon_a)}{2}
$$

(6.1)

$$
\varepsilon_q = \frac{2}{3}(\varepsilon_a - \varepsilon_t)
$$

(6.2)

6.3.2 Quantification of Breakage

The ballast was washed thoroughly after each test to separate any clay fouling and was then dried in air. A sieve analysis was carried out to measure the extent of particle breakage. In this study, the Ballast Breakage Index (BBI) was evaluated using the relationship given below, as described in Chapter 5 (section 5.1)

$$
BBI = \frac{A}{A + B}
$$

(6.3)

6.3.3 Resilient Modulus

Bursts of data (1000 readings in 1.7 seconds at a sampling frequency of 590 Hz) were collected by the data logger control program GOUI (Figure 6.2) to evaluate the resilient modulus $M_R$. The bursts were sampled over two channels, in the load (later converted to stress) and actuator positions (recoverable or resilient strain). $M_R$ was calculated using Equation 6.4, where $\Delta_{q,\text{cy}}$ is the magnitude of deviator stress, as defined in Equation 6.5, and $\varepsilon_{a,\text{rec}}$ is the recoverable portion of axial strain (Figure 6.1).
\[ M_R = \frac{\Delta q_{\text{cyc}}}{\varepsilon_{a,\text{rec}}} \quad (6.4) \]

\[ \Delta q_{\text{cyc}} = q_{\text{max}} - q_{\text{min}} \quad (6.5) \]

Figure 6.2: GOUI screen used to control the movement of the dynamic actuator
(Adopted from Lackenby, 2006)

6.3.4 Specimen ‘Failure’ under cyclic loading

In the monotonic triaxial testing of ballast it is usual to continue loading until a
certain predetermined value of axial strain (\(\varepsilon_a\)) has been reached, for example 28 %
in this current study. Specimen failure is defined at the peak deviator stress \(q_{\text{peak,sta}} =
(\sigma_1' - \sigma_3')_{\text{peak}}\), and this value corresponds to the maximum load carrying capacity of
the specimen. The magnitude of $q_{\text{peak,sta}}$ depends on the degree of fouling, the loading conditions (effective confining pressure), the gradation of ballast and applied strain rate, and the drainage characteristics. Unlike fine-grained materials such as clay, large granular specimens do not have a distinctive failure plane or shear band. Instead, failure is usually accompanied by specimen ‘bulging’ (Indraratna et al., 1998).

Identifying the location of specimen ‘failure’ during cyclic loading, which may be indicated by a loss of bearing capacity or some other mechanism, is fundamental in assessing the strength of materials subjected to repeated loading environments. In drained, stress controlled, cyclic loading at high frequency and constant deviator stress magnitude, the onset of ‘failure’ is difficult to ascertain owing to the absence of a peak stress or a characteristic location where the material undergoes weakening or softening. Past research has identified that a prominent settlement can be observed in the specimen during the initial loading cycles (e.g. Alva-Hurtado and Selig, 1981; Jeffs and Marich, 1987; Anderson and Key, 2000; Dahlberg, 2001). If the strength of the material is exceeded during this period then a rapid axial strain ($\varepsilon_a$) and large expansive radial strains ($\varepsilon_r$) would result, and if this situation continues at each successive loading cycle, albeit temporarily, the accumulative axial strain becomes extensive and the specimen ‘fails’.

Failure of large ballast specimens under repeated loading appears to be more appropriately defined by an arbitrary level of accumulated axial strain, so an axial strain $\varepsilon_a$ that is greater than 25% has been chosen (Lackenby et al., 2007) as the
failure criterion for the ballast specimens in this study. An axial strain $\varepsilon_a$ that equals 25% represents a 75 mm vertical loss in track geometry for a standard 300 mm thickness of ballast.

6.4 Results and Discussion

The upper plot of Figure 6.3 illustrates the variation in axial strain of fouled ballast ($VCI=10\%-80\%$) compared to clean ballast ($VCI=0\%$), with the number of load cycles ($N$) at an initial effective confining pressure of 10kPa. With a $VCI$ between 0% and 50%, the axial strains gradually increase at a decreasing rate as the number of cycles ($N$) increases up to 50,000 cycles, but beyond this region the axial strains approach a constant level (shakedown region). However when $VCI$ exceed 50%, the axial strain seems to increase continuously.

It is clear that as fouling increases, the axial strains also increase significantly. This increase in axial strain can be attributed to cyclic deviator stresses exceeding the peak deviator stresses observed during the strain-controlled monotonic loading tests, as reported earlier by Indraratna et al. (2011). Suiker (2005) also observed a similar trend for clean ballast subjected to cyclic loading. This can be further explained by the relative cyclic stress ($n$) defined by Suiker (2005). As shown in Table 6.1, the value of $n$ increased with an increase in $VCI$, and the corresponding increase in axial strains also increased.
\[ n = \frac{\left( \frac{q}{p} \right)_{\text{cyc}}}{\left( \frac{q}{p} \right)_{\text{stat, max}}} \] (6.6)

Figure 6.3: Strain response under cyclic loading: (a) axial strain \( \varepsilon_a \), (b) volumetric strain \( \varepsilon_v \), as a function of the number of cycles \( N \)
Table 6.1: Value of relative cyclic stress $n$

<table>
<thead>
<tr>
<th>VCI, %</th>
<th>relative cyclic stress, $n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.98</td>
</tr>
<tr>
<td>10</td>
<td>1.00</td>
</tr>
<tr>
<td>25</td>
<td>1.02</td>
</tr>
<tr>
<td>50</td>
<td>1.04</td>
</tr>
<tr>
<td>80</td>
<td>1.06</td>
</tr>
</tbody>
</table>

The bottom plots of Figure 6.3 show the changes in volumetric strain with respect to $N$, for varying levels of fouling at a confining pressure of 10kPa. It was observed that all the fouled ballast specimens dilated at the initial stage of cyclic loading (indicated as a negative sign), and due to the subsequent densification, the specimens indicated overall compression as the loading cycles were increased.

With the increase of level of fouling, maximum dilation also increases. This is because in the absence of cyclic densification (which normally occurs after 10,000 cycles), the loss of inter-particle contacts between the ballast aggregates due to the increased fouling, resulted in higher dilation. Suiker (2005) reported that when their cyclic stress levels approached failure, fresh ballast showed dilation at the initial stage of loading, but after about a thousand load cycles, dilation transformed to compression, which signalled the onset of progressive densification.
The upper plot of Figure 6.4 illustrates the variation in axial strain of highly fouled ballast \((VCI=80\%)\) compared to clean ballast \((VCI=0\%)\), with the number of load cycles \((N)\) at an effective confining pressure of 10kPa. At a higher degree of fouling, such as 80% of VCI, segregation between ballast particles due to excessive amounts of clay resulted in a lower initial placement density compared to the other samples, and this was responsible for the rapid increase in axial strains as the number of load cycles increased. In addition, the bottom plot of Figure 6.4 shows the development of excess pore water pressure for 80% VCI fouled ballast, which leads to a decrease in the mean pressure \((p_{cyc})\), causing an increase of “n”. It was observed that at 80% VCI, the axial strains exceeded 25% at \(N=250,000\), which was considered failure by Lackenby (2007), and a stable level of strains was not reached even after 500,000 load cycles.
Figure 6.4: Variation of: (a) axial strain $\varepsilon_a$, (b) volumetric strain $\varepsilon_v$ (c) Excess pore water pressure measured at the middle of the specimen as a function of the number of cycles $N$. 

- $80\%$ VCI
- $q_{\text{min}} = 45\text{kPa}$
- $q_{\text{max}} = 230\text{kPa}$
- $\sigma' = 10\text{kPa}$

Axial Strain $\varepsilon_a$, %

Volumetric Strain $\varepsilon_v$, %

Excess Pore Water Pressure $\Delta U$, kPa

Number of cycles, N
Figure 6.5 illustrates the final (accumulated) permanent axial strain of all the ballast specimens as a function of the Voids Contaminant Index (VCI). Up to 50% VCI, the final permanent axial strains gradually increased, but at a declining rate. However, beyond this the axial strain appears to increase rapidly as the specimen exceeds its static shear strength and would have approached its cyclic shear strength. This is normally measured while conducting a static load test after a certain number of cycles (Suiker, 2005).

In Figure 6.5 there are 3 distinguished zones which can be explained as follows:

In Zone 1, the axial strains increase rapidly with an increase in the VCI. This contributed to the ‘lubrication’ effect of kaolin fines where the small amount of kaolin clay forms a thin coating around the ballast particles.

In Zone 2, with the further addition of fouling material, the lubrication effect does not seem to increase significantly and the increase in fouling material compared to Zone 1 contributes very little to the mechanical behaviour of ballast. Therefore, the increase in axial strains remains almost negligible.

In Zone 3, the significant amount of fouling material occupying the voids in the ballast reduces its drainage capability, which leads to the development of excess pore water pressure and a reduction in the effective stresses, as well as increased axial strains.
Figure 6.5: Final Axial strain $\varepsilon_a$ at 500,000 cycles as a function of the Voids Contaminant Index

Figure 6.6 illustrates the final (accumulated) permanent volumetric strain of all the ballast specimens as a function of $VCI$. With an increase in fouling ($VCI$) the tendency towards compression decreases, as increased clay fouling occupies the voids and acts as a filler adversely affecting the rearrangement of ballast particles.
Figure 6.6: Final Volumetric strain $\varepsilon_v$ at 500,000 cycles as a function of the Voids Contaminant Index

Figure 6.7 shows the variation of initial (at $N = 50$ cycles) and final recoverable axial strains (at $N = 500,000$ cycles). Both the initial and final recoverable strains increase with an increase in the $VCI$. However, there is a significant increase in the recoverable axial strain at the end of the cycles. This increase in recoverable strains makes train drivers think they are driving on a soft soil and after influences them to reduce the train speed according to rail practitioners.
Figure 6.8 shows the resilient modulus \( M_r \) as determined from Equation (6.4) as a function of \( N \) for different degrees of fouling. \( M_r \) increases with an increase of \( N \) at a diminishing rate, until it approaches to a constant. An increased level of fouling leads to a decrease in \( M_r \). For instance, this would lead to an enforcement of speed restrictions during the rainy season.

This lower \( M_r \) means that higher elastic axial strains compared to clean ballast as deviator stresses are maintained constant during loading. This implies that fouling increases both the elastic and plastic components of axial strains. More elastic strains lead to greater vibrations of the track structure which may further reduce the resiliency of ballast (low \( M_r \)).
Figure 6.8: Resilient Modulus $M_r$ responds as a function of $N$

Figure 6.9 shows the Ballast Breakage Index ($BBI$) as a function of $VCI$. The extent of ballast breakage was analysed by carrying out pre-test and post-test particle size distribution (PSD) analyses. In this study, the Ballast Breakage Index ($BBI$) was evaluated as described in section 6.3.2. Figure 6.9 confirms that the ballast particles experience less breakage at higher values of $VCI$. It seems that the clay would provide enough cushioning to prevent any harsh attrition between the rough and angular particles and thereby reduce the otherwise higher inter-particle contact stresses and associated ballast degradation.
6.5 Summary

In this chapter a laboratory investigation to study the permanent and resilient deformation of railway ballast under high speed cyclic loading of clay fouled ballast was presented. The detrimental effects of fouling on the performance of ballast were highlighted through a number of key parameters such as axial deformation, volumetric deformation, and the resilient modulus.

With a VCI of less than 50%, the axial strains gradually increase at a diminishing rate as the number of cycles (N) increases up to 50,000 cycles, and then approach a constant level (cyclic densification). However with a VCI greater than 50%, the
axial strain seems to increase continuously without approaching the cyclic densification. An increase in fouling always increases the permanent axial strain at every number of cycles.

At all levels of fouling, the clay fouled ballast exhibited dilation at the initial stage of cyclic loading (up to ~ 50 cycles) and then changed to compression due to subsequent densification, as the loading cycles continued. However, this increase in fouling lowered the final compressive strain because clay particles act like a void filler. With the increase in the level of fouling, maximum dilation also increased.

The magnitude of resilient strain was shown to decrease with an increasing degree of fouling. This implies that fouling increases the elastic component of axial strains. Increased elastic strains generate more vibration of the track structure which would further reduce the ballast resiliency.

Apart from the detrimental effects discussed earlier, fouling leads to a decrease in particle breakage as it provides a cushioning effect which can be considered as beneficial to some extent. However, the higher degree of fouling discussed in previous chapters causes significant track problems, such as poor drainage and poor load bearing capacity. These detrimental effects often dominate track performance even though ballast breakage may reduce due to fouling acting as a ‘cushioning’ fill between aggregates.
7 APPLICATION OF BOUNDING SURFACE PLASTICITY CONCEPT FOR CLAY FOULED BALLAST

7.1 Introduction
Using bounding surface plasticity, a constitutive model is formulated to simulate the nonlinear behaviour of ballast with varying degree of clay fouling. The model describes strain-softening and stress-dilatancy with 13 model parameters determined from the results of the large scale monotonic triaxial tests as presented in Chapter 5. This chapter describes the stepwise procedure of the constitutive model employing the bounding surface concept.

7.2 Bounding surface plasticity models
The bounding surface plasticity has attracted a great deal of interest due to its simplicity and the ease of use. The concept of bounding surface was first introduced by Dafalias and Popov (1976) and Krieg (1975) in metal plasticity with a simple isotropic rule. This new concept was applied to clays by Dafalias and Herrmann (1982) to sands by Hashigushi and Ueno (1977), Aboim and Roth (1982), and Bardet (1983), and to pavement base materials by McVay and Taesiri (1985). In contrast to the above bounding surface models, the model proposed by Bardet (1986) takes into account of strain softening and stress-dilatancy. Dafalias (1982) introduced the concept of “Bounding Surface” in stress space within the framework of critical state soil plasticity.

The salient feature of this bounding surface concept is that plastic deformation may occur when the stress state lies on or within the bounding surface, by allowing the plastic modulus to be a decreasing function of the stress state from a corresponding
point on a bounding surface (Figure 7.1). The essential elements of the bounding surface plasticity are (Dafalias and Herrmann, 1980):

(i) bounding surface separating admissible from inadmissible states of stress,

(ii) loading surface on which the current stress state lies,

(iii) plastic potential describing the mode and component magnitudes of plastic deformation, and

(iv) hardening rules controlling the movement of the current stress state towards the image point on the bounding surface, as well as the size and locations of the loading and bounding surfaces.

---

**Figure 7.1.** Schematic illustration of bounding surface (after Dafalias and Herrmann, 1982)
The approach is geometrical in nature, and makes no appeal to physical reasoning of the problem. Furthermore, it lends itself to a number of general and versatile formulations, each removing the inherent restrictions in the conventional theory of plasticity (Khalili et al., 2005). Bounding surface framework introduced by Dafalias and Herrmann, (1980) is applied to the clay fouled ballast.

7.3 Preliminaries

For simplicity, triaxial notations (p’-q) are adopted throughout and time-dependent phenomena are ignored. A non-associate plastic flow rule is adopted. The model is validated with a series of monotonically loaded drained triaxial tests.

7.3.1 General equations

\[ p' = \left( \frac{\sigma'_1 + 2\sigma'_3}{3} \right) \]  
(7.1)

\[ q = \sigma'_1 - \sigma'_3 \]  
(7.2)

\[ \varepsilon_v = \varepsilon_1 + 2\varepsilon_3 \]  
(7.3)

\[ \varepsilon_q = \frac{2}{3} (\varepsilon_1 - \varepsilon_3) \]  
(7.4)

\[ \eta = \frac{q}{p'} \]  
(7.5)

7.3.2 Bounding Surface

The concept of a bounding surface introduced by Dafalias and Popov (1975) is applied here. Here, the plastic deformation occurs when the stress state lies on or
within the bounding surface. This is achieved by defining the plastic modulus to be a
decreasing function of the distance between $\sigma_i$ on the loading surface and $\sigma_i$ (image
point) on the bounding surface, which is denoted by $\delta$ (Figure 7.1). Stress conditions
on the bounding surface are denoted using an overbar (i.e. $\sigma_i$).

Khalili (2004) and Yu (1998) used following equation for the bounding surface to
model the sand behaviour;

$$F = \bar{q} - M_{cs} \bar{p} \left[ \frac{\ln(\bar{p} / \bar{p}_{\sigma})}{\ln R} \right]^{\gamma_N} = 0$$  \hspace{1cm} (7.6)

Bounding Surface in this study is defined by the function below (Figure 7.2) to
accompany the highly nonlinear behaviour of ballast as reported by Indraratna et
al.(1998). The inclusion of $p'_{o}$ into the bounding surface equation is very important
because granular material behaves differently at different confining pressure. In this
bounding surface inclusion of $p'_{o}$ is such a way that increase of $p'_{o}$ increases the size
of the bounding surface so that the increased deviator stress response of the ballast at
the elevated confining pressures is adequately captured by $p'_{o}$. This implies that at
higher confining pressure, shear strength of material is higher. However, the shape of
the bounding surface still remains same as Equation 7.6 with this inclusion.

$$F = \bar{q} - a \left( \sqrt{p_{\sigma} / p_{o}} \right) M \bar{p} \left[ \frac{\ln(\bar{p} / \bar{p}_{\sigma})}{\ln R} \right]^{\gamma_N} = 0$$  \hspace{1cm} (7.7)

where, M is the slope of the critical state line (CSL) in the $p'-$q plane. $\bar{p}_{\sigma}$ controls the
size of the bounding surface and it is a function of $\varepsilon_{\sigma}^p$. 
\[ \bar{p}_c = \exp \left[ \frac{\Gamma - v_o - \kappa \ln p'}{\lambda - \kappa} \right] \] (7.8)

where, the slope of the line connecting the critical state points in \( p-q \) plane gives the value of \( M \), and that in \( e-lnp \) plane gives \( \lambda \). The specific volume \((e+1)\) of the critical state line at \( p = 1 \) kPa is the value of \( \Gamma \). \( v_o \) is the initial specific volume \((e_o+1)\). \( R \) is a material constant representing the ratio between \( \bar{p}'_c \) and the value of \( \bar{p}' \) at the intercept of bounding surface with the CSL. The material constant \( N \) controls the curvature and varies with \( VCI \). \( \alpha \) is a material constant and \( p_a \) is arbitrarily selected to normalise pressure and its value equals to 1kPa.

Figure 7.2. Schematic illustration of bounding surface, loading surface and mapping rule
7.3.3 Loading surface

The loading and bounding surfaces are assumed to be of the same shape and homologous about the origin in the \( q - p' \) plane. The function of the loading surface is;

\[
f = q - a \left( \frac{p'_c}{p'_{o}} \right) M_p \left[ \ln \left( \frac{p'_c}{p'} \right)^{1/N} \right] = 0
\]

(7.9)

where, \( p'_c \) is an isotropic hardening parameter controlling its size as illustrated in Figure 7.2.

7.4 Constitutive Relations

In bounding surface plasticity, as in classical plasticity, the incremental strains \((d\varepsilon_v, d\varepsilon_q)\) resulting from a stress increments \((dp, dq)\) are the sum of elastic and plastic incremental strains, hence,

\[
d\varepsilon_v = d\varepsilon_v^e + d\varepsilon_v^p
\]

(7.10)

\[
d\varepsilon_q = d\varepsilon_q^e + d\varepsilon_q^p
\]

(7.11)

7.4.1 Elastic constitutive relations

The increment of elastic strain \((d\varepsilon_v^e, d\varepsilon_q^e)\) is given by isotropic elasticity using the bulk modulus, \(B\), and the shear modulus, \(G\).

\[
d\varepsilon_v^e = \frac{dp}{K}
\]

(7.12)

\[
d\varepsilon_q^e = \frac{dq}{3G}
\]

(7.13)
where,

\[
K = \frac{(1+e_o)p'}{\kappa} \tag{7.14}
\]

\[
G = \frac{3(1-2\nu)(1+e_o)p'}{2(1+\nu)\kappa} \tag{7.15}
\]

In the above, \(e_o\) is the initial void ratio and \(\nu\) is the Poisson’s ratio.

It is now possible to obtain the plastic constitutive relations as follows:

\[
d\varepsilon_p^p = \frac{1}{h}[n_p dp + n_q dq]m_p \tag{7.16}
\]

\[
d\varepsilon_q^p = \frac{1}{h}[n_p dp + n_q dq]m_q \tag{7.17}
\]

where, \(n_p, n_q\) are loading direction unit vectors (i.e. unit vector normal to the loading or bounding surface) and \(m_p, m_q\) are plastic flow directional unit vectors (i.e. unit vectors normal to the plastic potential) along \(p'\) and \(q\) axes. The parameter \(h\) is the plastic modulus.

\[
n_p = \frac{\left( \frac{\partial f}{\partial p'} \right)}{\sqrt{\left( \frac{\partial f}{\partial p'} \right)^2 + \left( \frac{\partial f}{\partial q} \right)^2}} \tag{7.18}
\]

\[
n_q = \frac{\left( \frac{\partial f}{\partial q} \right)}{\sqrt{\left( \frac{\partial f}{\partial p'} \right)^2 + \left( \frac{\partial f}{\partial q} \right)^2}} \tag{7.19}
\]
Using partial derivatives of equation (7.9) and substituting in equations 7.18 and 7.19 respectively, gives:

\[ n_p = \frac{\eta \left[ \frac{1}{N \ln \left( \frac{p_p}{p'} \right)} - 1 \right]}{\sqrt{\left\{ \eta \left[ 1 - \frac{1}{N \ln \left( \frac{p_p}{p'} \right)} \right] \right\}^2 + 1}} \]  
(7.20)

\[ n_q = \frac{1}{\sqrt{\left\{ \eta \left[ 1 - \frac{1}{N \ln \left( \frac{p_p}{p'} \right)} \right] \right\}^2 + 1}} \]  
(7.21)

### 7.4.2 Plastic potential

The plastic potential defines the ratio between the incremental plastic volumetric strains to the plastic shear strain. The flow rule used by Khalili et al. (2005) and Gajo and Muir Wood (1999) is modified here to capture the effect of fouling. They expressed the flow rule as:

\[ \frac{d \varepsilon^p_v}{d \varepsilon^p_q} = M_f - \eta \]  
(7.22)

where,

\[ M_f = (1 + k_d \xi)M \]  
(7.23)

where the state parameter (\( \xi \)) is given as

\[ \xi = e - e_c \]  
(7.24)

In the above, \( e \) is the void ratio at the current stress (\( p' \)), \( e_c \) is the void ratio at the critical state corresponding to \( p' \) and \( \eta \) is the stress ratio. In this study, \( k_d \) is a material
parameter which is varying with the confining pressure as well as the degree of fouling as described in section 7.5.

Therefore, plastic flow directional unit vectors $m_p$ and $m_q$ can be written as:

$$m_p = \frac{M_f - \eta}{\sqrt{1 + (M_f - \eta)^2}}$$  \hspace{1cm} (7.25)

$$m_p = \frac{1}{\sqrt{1 + (M_f - \eta)^2}}$$  \hspace{1cm} (7.26)

7.4.3 Hardening modulus

In bounding surface plasticity, the hardening modulus $h$ is usually divided into two components:

$$h = h_b + h_f$$  \hspace{1cm} (7.27)

where, $h_b$ is the plastic modulus at $(\bar{p}', \bar{q}')$ on the bounding surface, and $h_f$ is some arbitrary modulus at $(p', q)$ defined as a function of the distance between $(\bar{p}', \bar{q}')$ on the bounding surface and $(p', q)$ on the loading surface (i.e. $\delta$ in Figure 7.2).

Applying the consistency condition at the bounding surface, and assuming isotropic hardening of the bounding surface with plastic volumetric compression, $h_b$ can be determined as:

$$h_b = -\frac{\partial F}{\partial \bar{p}_c} \times \frac{\partial \bar{p}_c}{\partial \bar{e}_c} \times \frac{m_p}{\|\partial F/\partial \bar{e}\|}$$  \hspace{1cm} (7.28)

From the bounding surface, differentiating Equation (7.7) with respect to $\bar{p}_c$ gives:
From isotropic volumetric hardening, the evolution of $P'_e$ with $\delta e^p_\kappa$ is given as by:

$$\frac{\partial P'_e}{\partial \delta e^p_\kappa} = \frac{(1+e_o)}{\lambda - \kappa} P'_e$$  \hspace{1cm} (7.30)$$

$$\frac{\partial \| F \|}{\partial \sigma} = \sqrt{1+\left(1\frac{\eta}{N\ln(P'_e/P)}\right)^2}$$ \hspace{1cm} (7.31)$$

Substituting equations 7.29 and 7.30 into equation 7.28 gives:

$$h_b = \frac{M_\alpha(\sqrt{p'/p'_o})}{\lambda \kappa \ln(P'_e/P)} \times \frac{(1+e_o)}{(\lambda - \kappa)} \frac{m_p}{\partial F/\partial \sigma}$$ \hspace{1cm} (7.32)$$

The hardening modulus $h_f$ is assumed as given below to capture the effect of fouling:

$$h_f = \frac{t}{P'_e} \frac{\partial P'_e}{\partial \delta e^p_\kappa} \left(\frac{\delta}{\delta_{\max} - \delta}\right)$$ \hspace{1cm} (7.33)$$

where,

$$\delta = \sqrt{\bar{q}^2 + (\bar{p}' - \bar{p})^2}$$ \hspace{1cm} (7.34)$$

$$\delta_{\max} = \sqrt{\bar{q}^2 + (\bar{p})^2}$$ \hspace{1cm} (7.35)$$

where, $t$ is a function of VCI as well as initial confining pressure ($p'_o$) as described in section 7.5.
7.4.4 Image stress on bounding surface

For simplicity, the radial mapping rule is selected to define an image point on the bounding surface as given below:

\[ \bar{p} = x \bar{p}_c \]  \hspace{1cm} (7.36)

\[ \bar{q} = \eta x \bar{p}_c \]  \hspace{1cm} (7.37)

where, \( x \) is a function of \( \eta \) and can be found substituting equation 7.36 and 7.37 to equation 7.7, given by:

\[ x = \frac{1}{\exp \left[ \ln \left( M^\gamma \right) \right]} \]  \hspace{1cm} (7.38)

7.5 Evaluation of Model Parameters

To calibrate the model quantitatively, the numerical model predictions were compared with the selected laboratory experimental results, i.e. 0%, 50% and 80% VCI at various confining pressure values. \( M, \lambda \) and \( \Gamma \) were presented in Chapter 5 (Section 5.6). The parameter \( \kappa \) can be determined from an isotropic (hydrostatic) loading-unloading test with the measurements of volume change. The slope of the unloading part of isotropic test data plotted in e-lnp plane gives the value of \( \kappa, \nu, \kappa, N \) and \( R \) as shown in Table 7.1. The \( \alpha \) is assumed to equal 7.8.
Table 7.1. υ, κ, N and R values for different degree of fouling

<table>
<thead>
<tr>
<th>VCI, %</th>
<th>υ</th>
<th>κ</th>
<th>N</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.3</td>
<td>0.0070</td>
<td>2.0</td>
<td>2.2</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>0.0068</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>50</td>
<td></td>
<td>0.0059</td>
<td>3.1</td>
<td>2.6</td>
</tr>
<tr>
<td>80</td>
<td></td>
<td>0.0053</td>
<td>3.7</td>
<td>3.0</td>
</tr>
</tbody>
</table>

The values of \( k_d \) are determined by applying equations 7.22 to 7.24 into the experimental data ignoring the elastic strain components as this fraction is negligible. The values of \( k_d \) at different confining pressure values are plotted against VCI (Figure 7.3). \( k_d \) decreases significantly when VCI is less than 40%.

![Figure 7.3. Variation of \( k_d \) with VCI at different confining pressures](image)

General equation for \( k_d \) can be proposed from the regression analysis:

\[
k_d = \left[ 0.094 \ln \left( \frac{P^*}{P_a} \right) - 0.418 \right] \times \left[ 1 + \left( 1.07 \exp \left( \frac{0.03P^*}{P_a} \right) \right) \right] \tan \left[ \frac{VCI}{k_1 \ln \left( \frac{P^*}{P_a} \right) + k_2} \right]
\] (7.39)
where, \(k_1\) and \(k_2\) are model constants.

Equation 7.39 can be expressed as;

\[
k_d = \left[0.094 \ln \left( \frac{p'_o}{p_a} \right) - 0.418 \right] \times \left[ 1 + \left\{ 1.07 \exp \left( \frac{0.03 p'_o}{p_a} \right) \right\} \tan \left( \frac{VCI}{A} \right) \right]
\]  

(7.40)

where,

\[
A = k_1 \ln \left( \frac{p'_o}{p_a} \right) + k_2
\]  

(7.41)

‘A’ values corresponding to different confining pressures (\(p'_o\)) can be found by fitting the Equation 7.40 to predetermine \(k_d\) as shown in Figure 7.3. Then ‘A’ values can be plotted against \(\ln(p'_o/p_a)\) as shown in Figure 7.4 to determine \(k_1\) which is the gradient of the best fit line and \(k_2\) which is the intercept (Figure 7.4).

![Figure 7.4. Variation of ‘A’ against \(\ln(p'_o/p_a)\)](image-url)
Hardening parameter \( t \) is determined by applying Equations 7.16 to 7.38 to the experimental data. Then the values of \( t \) at different confining pressure values were plotted against \( VCI \) as shown in Figure 7.5.

![Figure 7.5. Variation of ‘t’ with VCI at different confining pressures](image)

General equation for parameter, \( t \) can be proposed from the regression analysis:

\[
\frac{t}{t_1 + \left( \frac{p'_o}{p_a} \right) + t_2 \text{VCI}} = \left[ 0.2 \left( \frac{p'_o}{p_a} \right) + 43 \right]
\]

(7.42)

where, \( t_1 \) and \( t_2 \) are model constants.

Equation 7.42 can be expressed as;

\[
t = \frac{t_1 \left( \frac{p'_o}{p_a} \right) + t_2 \text{VCI}}{1 + \left( 0.2 \left( \frac{p'_o}{p_a} \right) + 43 \right)}
\]

(7.43)

where,
\[
B = t_1 \left( \frac{p'_o}{p_a} \right) + t_2
\]  
(7.44)

‘B’ values corresponding to different confining pressure \((p'_o)\) can be found by fitting Equation 7.43 to the above \(t\) values (Figure 7.5) Then ‘B’ values can be plotted against \((p'_o/p_a)\) as shown in Figure 7.6, where, \(t_1\) is the gradient of the best fit line and \(t_2\) is the intercept (Figure 7.6).

![Graph of B vs. \((p'_o/p_a)\)](image)

Figure 7.6. Variation of ‘B’ against \((p'_o/p_a)\)

### 7.6 Model Validation and Predictions

To validate the model, triaxial test series conducted for 25% \(VCI\) is used and presented in Figure 7.7 to Figure 7.9 together with other experimental results used for the calibration. Initial conditions for each test are shown in Table 7.2. As shown in Figure 7.7, the constitutive model is able to correctly simulate strain softening and stress-dilatancy of clay-fouled ballast at different ranges of fouling \((0 \leq VCI \leq 80\%)\) and at low confining pressure of 10kPa. The volumetric strain response of fouled ballast as predicted by this model is in good agreement with the experimental test.
data. The model captures the increased deviator stress and an overall volumetric decrease (reduced dilation) with an increase in the confining pressure to 30 kPa and 60 kPa, as observed in the laboratory data (Figure 7.8 and 7.9). Thus, the constitutive model, presented in this chapter, can accurately capture stress-strain response of clay-fouled ballast for varying VCI \((0 \leq \text{VCI} \leq 80\%\)) and for given range of confining pressures.

As described in Chapter 5, the overall volumetric response of the clay-fouled ballast was initially compressive, but depending on the VCI, subsequent dilation was observed which bears a significant contrast to the behaviour of clean ballast. It was also observed that increased level of fouling \((\text{VCI} \geq 25\%\)) decreased both the rate and magnitude of dilation at high axial strain. Therefore, it is evident that this model can predict reduced maximum compression and suppressed dilation at large strains with increase of VCI (See Figure 7.10).
Table 7.2. Initial conditions for each test

<table>
<thead>
<tr>
<th>VCI, %</th>
<th>P’₀, kPa</th>
<th>e₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>10</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.7</td>
</tr>
<tr>
<td>25</td>
<td>10</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.66</td>
</tr>
<tr>
<td>50</td>
<td>10</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.51</td>
</tr>
<tr>
<td>80</td>
<td>10</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.34</td>
</tr>
</tbody>
</table>
Figure 7.7. Analytical prediction of stress-strain of ballast with different VCI, compared to test data at 10kPa confining pressure
Figure 7.8. Analytical prediction of stress-strain of ballast with different VCI, compared to test data at 30kPa confining pressure.
Figure 7.9. Analytical prediction of stress-strain of ballast with different VCI, compared to test data at 60kPa confining pressure.
Figure 7.10. Model Prediction for volumetric strains with different degree of fouling at 10kPa confining pressure

7.7 Practical Applications of the model

The model can be used to determine the peak strength of ballast at different degree of fouling as shown in Table 7.3.

Table 7.3. Peak strength of clay fouled ballast (kPa)

<table>
<thead>
<tr>
<th>VCI, %</th>
<th>10kPa</th>
<th>30kPa</th>
<th>60kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>268</td>
<td>338</td>
<td>468</td>
</tr>
<tr>
<td>25</td>
<td>196</td>
<td>274</td>
<td>378</td>
</tr>
<tr>
<td>50</td>
<td>170</td>
<td>254</td>
<td>341</td>
</tr>
<tr>
<td>80</td>
<td>153</td>
<td>245</td>
<td>331</td>
</tr>
</tbody>
</table>
In this study, three axle loads of 25, 30 and 35 tonnes were considered. The corresponding dynamic load is calculated using the equation proposed by Selig and Waters (1994) and adopted by American Railway Engineering Association (AREA), i.e.,

$$P_d = \left[ 1 + \frac{0.0052 \times V}{D_w} \right] \times P_s$$

(45)

where, $P_d$ is the design wheel load, $P_s$ is the static wheel load, $D_w$ is the diameter of the wheel (m), and $V$ is the velocity of the train (km/h).

$$P_s = \frac{\text{Axle load}}{2}$$

(46)

Atalar et al. (2001) reported that part of this design wheel load is transferred to the adjacent sleepers, and 40-60% of the wheel load is supported by the sleeper directly below the wheel. Therefore 50% of the design wheel load is assumed to be the rail seat load. To determine the sleeper-ballast contact pressure ($q_d$), the following equation proposed in Japanese standards (Atalar et al., 2001) as illustrated in Figure 7.11 is used, hence,

$$q_d = \left( \frac{q_r}{2aB} \right) F_2$$

(47)

where, $q_r$ is rail seat load, $B$ is the width of the sleeper and $a$ is the distance between rail head centre and edge of the sleeper. $F_2$ is a factor depending on the type of sleeper.
In this study, from Singleton (NSW) case study track data is used to determine the allowable train speed. as follow and three different train speeds (V= 40, 60 and 100 km/h) were considered, for which:

\[ D_w = 0.97m \]
\[ B = 0.25m \]
\[ a = 0.5m \]
\[ F_2 = 1 \]

The above parameters are defined through Equations (45) – (47).

Using the peak strength obtained from the model predictions and Equations 45-47 the allowable train speed, \( V \) in Equation (45) can be determined and plotted against VCI as shown in Figures 7.12 to 7.14 for axle loads 25-35 tonnes.
Figure 7.12. Allowable train speeds for 25 tonnes axle load at different confining pressures

Figure 7.13. Allowable train speeds for 30 tonnes axle load at different confining pressures
Figure 7.12 shows that at 10kPa confining pressure, the rail track can tolerate fouling up to 20% \( VCI \) for trains with 25t axle load passing at an average speed of 70km/h. However, if the confining pressure is more than 30kPa, the track is considered to be stable at all levels of fouling in terms of the strength criteria. In contrast, as described in Chapter 4, the ‘poor drainage’ can govern the track serviceability when \( VCI \) is more than 50%. Figure 7.13 shows that even though the confining pressure increases to 30kPa fouling cannot exceed 50% \( VCI \) for 30t axle trains moving at an average speed of 70km/h. Nevertheless, increasing of confining pressure beyond 60kPa will make the track stable enough against all levels of fouling. However, achieving a confining pressure of 60kPa in the field is often impractical as this involves the use of sheet plies on either side of track or overly close sleeper spacing. Confining
pressures exceeding 60kPa for 35t axle load (Figure 7.14) also indicate speeds higher than the average train speed.

Therefore considering both drainage and strength aspects, 20% VCI can be considered critical for axle loads below 25t with 10kPa of confining pressure. If the train axle load is 30t, the confining pressure should increase to at least 30kPa where fouling level should not exceed 50% VCI. The results presented in Figures 7.12, 7.13 and 7.14 can be used as guidelines for track maintenance together with the maintenance chart discussed earlier in Chapter 4.

7.8 Summary

A new constitutive model based on bounding surface framework for clay fouled ballast is proposed under monotonic loading. A non-associate plastic flow rule is adopted to include effects of clay fouling. The model describes important features such as strain softening and stress-dilatancy. The constitutive model needs 13 parameters to be evaluated from large scale monotonic drained triaxial tests. Variation of model parameters with void contaminant index demonstrate that the size and shape of bounding surface is directly influenced by ballast fouling. The calibration and validation is carried out at low confining pressures, as in real track scenarios, ballast is often subjected to less than 50kPa confining pressure as described in Chapter 5. Model predictions are in decent agreement with the experimental data. Using the model predictions, recommended maximum train speeds for fouled ballast is proposed considering both strength and drainage characteristics.
8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Introduction

Rail tracks are usually positioned on a coarse granular medium (i.e., ballast) for several reasons. These include essential requirements of economy (availability of material and abundance), rapid drainage, acceptable resiliency, and sufficient load bearing capacity. However as ballast ages the progressive intrusion of fines into the inter-granular voids (fouling) causes deterioration of its major functions, triggering high track maintenance costs. Since the fouling materials occupy the ballast voids, the drainage capacity of the track is compromised. Also, the lubrication effects of the finer fouling materials result in the reduction of the load carrying capacity as well as inducing differential settlements of the track.

To date, no Australian standards or technical specifications are available to guide track engineers when to maintain these fouled tracks in relation to its performance. If fouling could be quantified and captured, there would be significant savings on the cost of maintenance. To investigate these problems, a series of large scale laboratory experiments were carried out using the unique testing equipments designed and built at the University of Wollongong.

The following sections provide an outline of the major conclusions obtained with regard to the fouling indices, the effect of fouling on drainage, and the stress-strain
behaviour under monotonic loading, permanent strain, resilient modulus, and breakage under cyclic loading. Recommendations for future study are also provided.

8.2 Fouling Indices

In this thesis, the fouling indices which have been used conventionally in rail practices were introduced in Chapter 2. Apart from reviewing commonly used methods of assessing ballast fouling such as the Percentage of Fouling, Fouling Index (Selig and Waters, 1994), and Percentage Void Contamination (Feldman and Nissen, 2002), a new parameter called Void Contaminant Index (\( VCI \)) was introduced to quantify the extent of fouling.

The Void Contaminant Index (\( VCI \)) can capture the effects of void ratios, specific gravities, and the gradations of both fouling material and ballast. It is shown that the \( VCI \) accurately captures ballast fouling and can be adopted as a more realistic fouling index, especially when the fouling material has a specific gravity that is significantly different to the rock aggregates. Laboratory tests were conducted to measure the \( FI \) (Selig and Waters, 1994), \( PVC \) (Feldman and Nissen, 2002), and \( VCI \) values on clay-fouled ballast (simulated with kaolin as the fouling material), sand-fouled ballast (simulated with fine clayey sand as the fouling material), and coal-fouled ballast. A comparison among \( FI \), \( PVC \), and \( VCI \) for various percentages of fouling was presented. The method for determining the \( VCI \) in the field was also discussed.
8.3 Characterising the Drainage of Fouled Ballast

A series of large-scale constant head hydraulic conductivity tests were conducted with different levels of fouling to establish the relationship between the void contamination index and the corresponding hydraulic conductivity. Two types of fouling distributions (uniformly distributed and non-uniformly distributed) were simulated in the laboratory. Although a one-dimensional flow was maintained during laboratory studies, the test results were essential for investigating how the levels of fouling influenced the overall hydraulic conductivity of fouled ballast. The results confirmed that the hydraulic conductivity decreased with the increase in $VCI$. Initially, even a small increase in $VCI$ caused a significant decrease in the hydraulic conductivity of ballast, but beyond a certain limit of $VCI$ (50% for coal and 90% for clay) the hydraulic conductivity of fouled ballast converged to that of the fouling material itself.

An analytical model based on the concept of dual layer permeability was presented to predict the hydraulic conductivity of fouled ballast while considering non-uniform distribution of fouling material with depth. This analytical model agreed well with the laboratory test data.

Considering the fact that the flow is two-dimensional in the actual rail track while the laboratory measured hydraulic conductivity values were one-dimensional, a two-dimensional finite element seepage analysis was conducted to simulate actual track geometry using SEEP-W (GeoStudio, 2007a. The variation of hydraulic conductivity of ballast with different $VCI$ established through laboratory testing was incorporated in the FEM analysis. The drainage capacity of the track, which has different levels of
fouling in different layers, including the shoulder ballast, was estimated using this numerical model. The results of this analysis revealed that both the location and extent of fouling play a vital role when assessing the overall drainage capacity of the track. In this study the drainage condition of the track is proposed based on a typical high rainfall intensity in Australia and the associated track drainage capacity. The analysis shows that cleaning the ballast with the undercutting method should commence when the $VCI$ of the top 100mm of the ballast exceeds 50%. When the shoulder ballast is fouled to more than 50% $VCI$, it should be cleaned or replaced in order to maintain an acceptable drainage capacity. If the shoulder ballast is highly fouled (i.e. $VCI > 50\%$), ‘poor drainage’ occurs even if the other ballast layers are relatively clean.

8.4 Behaviour of Clay Fouled Ballast under Monotonic Loading

In order to understand the effect of clay fouling on the stress-strain behaviour of ballast for different proportions of fouling, a series of large scale consolidated drained triaxial tests under monotonic loading conditions were carried out at low confining pressures. The test results corroborate that fresh ballast always showed a higher deviator stress than fouled ballast at any given axial strain, for all the confining pressures tested, but this difference decreased at higher axial strains. The test results revealed that as fouling increases, the shear strength of the ballast decreases, but when the $VCI$ exceeds 25%, the subsequent decrease in shear strength becomes gradual.
An increasing $VCI$ generally shows a reduced compression, except when a lower level of fouling ($VCI < 10\%$) acts as a lubricant rather than a void filler for the fouled specimen, because, the voids between the ballast grains are occupied by fouling acting as a ‘filler’. At higher axial strains all the specimens indicate dilation but the binding effect generated by clay fouling suppresses the tendency to dilate.

In this study, a novel empirical relationship between the peak deviator stress and $VCI$ was proposed based on laboratory investigations which may be a useful tool when practicing engineers are making preliminary track assessment. A non-linear shear strength envelope for fouled ballast was proposed in a non-dimensional form, where the relevant shear strength coefficients could easily be estimated as a function of $VCI$, based on the proposed empirical equations.

The additional tests conducted at very high levels of fouling ($VCI > 80\%$) showed that the generation of excess pore water pressure in clay fouled ballast caused the effective confining pressure to decrease.

### 8.5 Behaviour of Clay Fouled Ballast under Cyclic Loading

To study the effect of fouling on the permanent and resilient deformation of railway ballast under high speed train loads, a series of large scale cyclic loading tests was executed at 20Hz frequency with different proportions of clay fouling. An increase in the $VCI$ always increased the permanent and recoverable axial strains, but when the $VCI$ exceeded 50\%, the axial strain seemed to increase continuously without approaching cyclic densification (constant level).
Unlike the behaviour of fouled ballast under static loading, as the number of loading cycles increased, fouled ballast initially dilated and then became compressed. In contrast under static loading, all the ballast specimens initially showed compression, followed by dilation as the axial strain increased. With the increase in fouling, the final compressive strains shows a reduction, because, the clay particles act as a ‘filler’.

The investigations also showed that fouling significantly decreases the resilient modulus ($M_r$), which implies that fouling increases the recoverable axial strains. More recoverable strains lead to greater track vibration which may induce train drivers to reduce the speed.

Apart from the unfavourable effects discussed earlier, fouling leads to a decrease in particle breakage as it provides a cushioning effect which is considered beneficial to some extent. However, the higher degree of fouling discussed in previous chapters causes significant problems in the track, such as poor drainage and reduced load bearing capacity, associated with higher differential settlements with time. These detrimental effects often dominate track performance even though ballast breakage may decrease due to fouling.

### 8.6 Constitutive model for clay fouled ballast

A new constitutive model using bounding surface framework was proposed for clay fouled ballast under monotonic loading. A non-associate plastic flow rule is adopted. The model describes important features such as strain softening and stress-dilatancy. The size and shape of bounding surface was influenced by degree of fouling. Model predictions were in decent agreement with the experimental data.
8.7 Recommendations

The following are recommendations for future studies.

- Large scale permeability tests have been conducted using one-dimensional flow, but to obtain both horizontal and vertical coefficients of hydraulic conductivity, a more rigorous two-dimensional flow should be simulated in the laboratory. This will provide insight to a more rigorous analysis of drainage conditions using the numerical (FEM) model.

- To evaluate the drainage capacity of the track more accurately, a more sophisticated finite element model with a number of different ballast zones and varied VCI will be required.

- Large scale triaxial experiments were limited to pure clay (kaolin) fouling to simulate the worst case scenario, but in reality, fouling can be a mixture of clay and sand/gravel. Therefore, if fouling with different proportions of sand or gravel is considered, the adverse effects of fouling can be quantified more realistically and accurately.

- The ballast tests were conducted under fully saturated conditions to simulate the worst possible track conditions, however as the moisture content of fouling can with time (climate condition), it is worthwhile to conduct laboratory investigations where the moisture content of the fouling materials is varied to capture the role of suction.
• In the laboratory studies, the ballast used to distinguish the effect of fouling at every level was fresh quarried aggregates. Past studies at the University of Wollongong (Salim, 2004) have shown that degraded ballast has less shear strength and more dilation. With more intrusion of fouling over time, ballast particles also degrade. Thus further laboratory investigations should be carried out using degraded ballast rather than fresh ballast. It may be argued that the combination of degraded ballast and fouling will further reduce the shear strength obtained for fresh ballast.

• The large scale triaxial cyclic loading program was limited to only one frequency (20 Hz) and one confining pressure (10 kPa). Future studies should examine the influence of frequency on fouled ballast behaviour in order to simulate high speed trains (>40Hz). Investigating the effect of confining pressure would also be a useful way to assess the performance of the track where ballast experiences different confining pressures.

• In the future, the more complex aspects of clay fouling needs to be carefully assessed through rigorous micro-mechanical studies in order to understand the role of cohesive fines interacting with coarse granular media, to predict the longevity and time dependent performance of ballasted tracks more accurately.
REFERENCES


*Granular Matter, 7*, No. 1, 19-29.


*Geotechnique, 56*, No. 9, 651-655.


A.1. Introduction

The high frequency triaxial test apparatus was designed and built at the University of Wollongong and its salient features are summarised below;

- An existing 4-post/ bridge test structure was re-used.
- A new system consists of a servo-actuator, transducers, load cell, and servo valves, and an electronic control system installed to assist the project.
- Hydraulic oil for the apparatus was supplied by the existing central power pack, operating at 20000 kPa. Pipework supplying oil to the test apparatus was designed and installed by commencing at the end terminations already installed at the main existing ring pipework.
- Operational and basic test level data is provided via a serial connection to a computer that operates software specifically configured for this application. This machine cannot be operated without an external computer.

It should be noted that the controller for this machine is a digital, programmable device that can be re-programmed to provide a number of altered operations. The basic design specifications for the test apparatus are shown in Table A.1
Table A.1. Test apparatus design specification

<table>
<thead>
<tr>
<th>Hydraulic Power Pack</th>
<th>• Existing 20 MPa up to 140 l/min fixed supply pressure.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Servo Actuator</td>
<td>• 20 MPa maximum safe working pressure.</td>
</tr>
<tr>
<td></td>
<td>• Design force range to 150 kN.</td>
</tr>
<tr>
<td></td>
<td>• 100mm Bore x 50mm rod x 200mm stroke.</td>
</tr>
<tr>
<td></td>
<td>• Custom designed to suit the application with low friction seals, and built in hydraulic services in the blind endcap.</td>
</tr>
<tr>
<td>Operating Modes</td>
<td>• Ramped position control (velocity control), with minimum set speed .01 mm/sec, maximum set speed 100 mm/sec.</td>
</tr>
<tr>
<td></td>
<td>• Force override when in position control.</td>
</tr>
<tr>
<td></td>
<td>• Static force control (totally independent of position), with a position override if the cylinder moves past a predetermined position.</td>
</tr>
<tr>
<td></td>
<td>• Dynamic force control, where the applied force varies sinusoidally between 2 set levels at a set frequency - control is independent of position, however a position override will occur if the cylinder moves past a pre-determined position.</td>
</tr>
<tr>
<td>Feedback devices</td>
<td>• 50,000lb rated Load cell at 4 mV/V signal sensitivity, amplified to provide 0-10VDC over 0-200kN range.</td>
</tr>
<tr>
<td></td>
<td>• 0-200mm magnetostrictive position transducer, providing 0-10V output signal.</td>
</tr>
<tr>
<td>Servo valves</td>
<td>• 2 off 60 l/min rated, very high response type, mechanical feedback.</td>
</tr>
</tbody>
</table>

A.2. Description of the test apparatus control system

The test apparatus electronic controller is a device that was specifically designed and built for high performance hydraulic control systems. It has been programmed to suit the test apparatus and contains a number of key features such as those described below. To operate this machine, a basic understanding of the control system is imperative.

A.2.1. Closed loop position control loop

Position control loop has 3 main functions;

• Hold the cylinder at a fixed position during setup, and non-operating times.
• Provide constant speed extension and retraction of the actuator when the changing position SP (setpoint) is entered by the operator. This varies from 0.6 mm/min to 100 mm/sec.

• Switch the cylinder from Force control to Position control if it exceeds the setpoint during Force control. This mechanism allows a test to be completed and the cylinder to be retracted to a desired home position.

A.2.2. Closed loop force control loop

The force control loop has 4 main functions;

• Switch the cylinder from position control to Force control if the force sensed at the load cell exceeds the force setpoint, and if the cylinder has not yet reached the setpoint position.

• Control the cylinder to ensure that the force sensed at the load cell follows the setpoint, which comprises a static level and a sinusoidal component if the wave generator is switched on.

• Ramp the basic setpoint force levels to provide smooth loading and unloading of the test piece at the beginning and end of a test.

• Generate a sinusoidal force profile at the operator requested frequency and magnitude for the servo actuator to follow.
A.2.3. Force peak detector

This software looks at the levels of force at each cycle of oscillation from the load cell feedback, and captures the maximum and minimum peaks. These values are continuously updated on the operator’s computer screen, and used by the Attenuation and DC Offset compensation functions.

A.2.4. Fault detection system

The software drives a warning light on the front of the controller assembly, and provides a similar red warning on the operator’s screen when either the force or position control systems are unable to control the actuator. In addition, an emergency stop button allows the operator to instantly stop the controller driving the servo valves.

A.3. Interfacing with the controller via a computer

The controller dedicated to control the test apparatus is a high speed digital controller, specifically designed and marketed for use with high performance hydraulic equipment. The commissioned software should never need to be altered unless the operation of the machine needs to be changed.

The only way to operate the test apparatus is via a computer connected to the controller, and via the serial connector on the front door of the controller enclosure. The computer can also be directly connected to the controller when the door of the enclosure is open by disconnecting the ribbon cable and connector on the front panel of the controller. The following information describes how the computer is setup to enable it to communicate with the controller.
A.3.1. Graphical operator user interface (‘GOUI’).

GOUI is a windows based programme that enables adjustments such as gain or other motion related variables to be made to the controller, monitors the system in operation, enables software to be reloaded into the controller, and provides a high speed, limited time data acquisition system for troubleshooting the various control loops.

A view of the GOUI screen is shown in Figure A.1, and a description of all of the functions on the screen are shown in Table A.2.

![Figure A.1. A view of the GOUI screen](image-url)
Table A.2. Description of all of the functions on ‘GOUI’ screen

<table>
<thead>
<tr>
<th>Function</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAVE button</td>
<td>This button is disabled, and cannot be used (Its function is to save software changes to the controller).</td>
</tr>
<tr>
<td>Position SP (mm)</td>
<td>An adjustment to determine where the actuator should be positioned. When altered, the actuator will move to its new position at the speed set at the ‘Cylinder Speed SP (mm/sec)’ field.</td>
</tr>
<tr>
<td><strong>NOTE:</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>To ensure the actuator transitions to Force control, the Position SP must be set to a point further than the contact point between the cylinder and the test piece.</td>
</tr>
<tr>
<td></td>
<td>1. If the test piece ‘creeps’ during a test such that the cylinder gradually extends past the Position SP, it will drop out of force control and back to position control. This feature ensures that a test piece is not totally destroyed if it starts to fail.</td>
</tr>
<tr>
<td>Cylinder Pos FB (mm)</td>
<td>The actual position of the cylinder at any time.</td>
</tr>
<tr>
<td>Cylinder Speed SP (mm/sec)</td>
<td>An adjustment to determine how fast the cylinder moves during position control only.</td>
</tr>
<tr>
<td><strong>NOTE.</strong></td>
<td>Minimum speed, and minimum increment size is 0.01 mm/sec.</td>
</tr>
<tr>
<td>Force Control</td>
<td>When force control has been achieved (i.e. the actuator has moved up against an obstruction and transitioned to Force control), the indicator light changes to green.</td>
</tr>
<tr>
<td>--------------</td>
<td>--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Load Cell FB (kN)</td>
<td>The actual sensed Force at the Load cell at any time.</td>
</tr>
</tbody>
</table>
| Lowest Force SP (kN) and PP Force Range SP (kN) | These 2 fields together set the Force control levels.  
- When the wave generator is turned off, the force SP is the Lowest Force SP, + 0.5 x the PP range. E.g. if lowest =60kN, and PP range =40 kN, then static force SP = 80 kN.  
- When the wave generator is turned on the force level minimum value is the Lowest Force, and the maximum level is the Lowest Force + the PP range. |
<p>| Min peak Force FB (kN) and Max peak Force FB (kN) | Actual Force Feedback values from the minimum and maximum peak detectors. |
| Control Saturated | When the controller is unable to control the cylinder, this indicator changes to red and the red light on the front of the controller turns on. This indicates either a fault in the system or that the performance specification requested is higher than the test apparatus can deliver (a combination of frequency and cylinder stroke to meet the load force variation) |
| Sine Wave Frequency (Hz) | The desired frequency of the force wave generator (sinusoidal wave shape). |</p>
<table>
<thead>
<tr>
<th><strong>Start Wave Generator</strong></th>
<th>An on/off set of buttons to start and stop the wave generator - only works when the cylinder is already in force control.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Force Loop Kp.</strong></td>
<td>The only adjustment that should be necessary for the force loop is the Kp parameter - this gain applies to a number of the actual mathematical blocks in the controller, and can be used to de-tune the controller if a test piece with low stiffness or high moving mass is to be tested. Commissioned value was 1.2</td>
</tr>
<tr>
<td><strong>Force Loop Ki.</strong></td>
<td>Force loop Ki, should never need to be adjusted (integral gain - affects overshoot on transition from position to force control).</td>
</tr>
<tr>
<td><strong>Position loop Kp</strong></td>
<td>Position loop proportional gain. The position loop should be left as commissioned in a highly de-tuned state to avoid excessive reaction to noise on the transducer signal, and to ensure smooth low speed operation.</td>
</tr>
<tr>
<td><strong>Actual SP Range (kN)</strong></td>
<td>This field indicates the actual PP range used by the controller when in force control with waves above 6Hz. (Attenuation Compensator). If the value is significantly below the PP range SP (kN) set on the LH side of the screen, then the test apparatus is approaching its dynamic limits. For example, during commissioning the system was run at 80Hz with ~ 0.5mm rod movement from 110 to 150 kN, and the actual PP range had increased from 40 to 75 kN.</td>
</tr>
<tr>
<td>Actual Lowest Force SP (kN)</td>
<td>Similar indication for how hard the offset compensator is working.</td>
</tr>
<tr>
<td>----------------------------</td>
<td>------------------------------------------------------------------</td>
</tr>
<tr>
<td>Valve Command (%)</td>
<td>The current valve command signal (to both valves in parallel)</td>
</tr>
</tbody>
</table>

The right hand side of the GOUI screen provides the data acquisition function when a relevant definition file has been opened from the scope menu. The oscilloscope provides a dynamic display of 2 channels of data, as determined by the various .SCP files. 3 off files have been created for troubleshooting or tuning, and are summarised as follows:-

‘Forcepos.scp’ - logs the Force and position FB values.
‘Forctune.scp’ - logs the Force FB and valve command signals.
‘Postune.scp’ - logs the position FB and valve command signals.

The controller allows 1000 samples worth of data to be recorded on each channel, with the sampling rate preset to every 1.7 msec, providing 1.7 seconds worth of data. Logging is initiated by clicking the ‘Start’ button under the logger display, and as soon as the data has been collected, it will be drawn onto the display. It should be noted that whenever a new data run is initiated any previously displayed data will be lost unless it has been saved under the scope menu first.
A.4. Load cell calibration

The load cell can be quickly and easily calibrated as follows:

1. Move the load cell and controller assembly to a universal tensile load testing machine.

2. Connect the controller to 240VAC, and open the front door. Locate the small green coloured load cell amplifier in the centre of the enclosure.

3. Connect a good quality voltmeter to the ‘Force FB’ test points on the front of the enclosure, and set the meter, if not autoranging, to a scale that can accurately read < 200mV.

4. With no load applied, adjust the ‘zero’ pot on the front of the load cell amplifier until the voltmeter reads 0.0 mV.

5. Reset the voltmeter to a > 10V scale, and apply a 200kN load. Adjust the span pot so that the testpoint voltage is 10.00V.

6. If necessary, check increasing and decreasing intermediate levels to verify linearity and hysteresis.

REFERENCES
Some field investigation carried out during the research work to study the characteristics of the fouling material and the preliminary test results are shown as follows.

**B.1. COAL FOULED BALLAST**

To investigate the permeability of coal fouling, samples were obtained from Rockhampton (QLD) and Bellambi (NSW). Figure B.1 shows the particle size distribution curves of the coal fouled samples.

![Particle Size distribution curves for samples of coal fouled ballast](image-url)

Figure B.1. Particle Size distribution curves for samples of coal fouled ballast
Laboratory investigation to determine the fouling indices were conducted and presented in Table B.1.

Table B.1. Fouling Indices for samples of coal fouled ballast obtained from Rockhampton (QLD) and Bellambi (NSW)

<table>
<thead>
<tr>
<th>Fouling Index</th>
<th>Rockhampton (QLD)</th>
<th>Bellambi (NSW)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F.I (Selig and Waters, 1994)</td>
<td>14</td>
<td>9</td>
</tr>
<tr>
<td>Percentage of the fouling/(%)</td>
<td>17</td>
<td>11</td>
</tr>
<tr>
<td>F.I_D (Ionescu, 2004)</td>
<td>20</td>
<td>6</td>
</tr>
<tr>
<td>PVC /(%)- (Feldman and Nissen, 2002)</td>
<td>55</td>
<td>26</td>
</tr>
<tr>
<td>VCI /(%)- (UOW method)</td>
<td>72</td>
<td>33</td>
</tr>
</tbody>
</table>

B.1.2. Small scale permeability test

Small scale falling head permeability tests were conducted on coal fouling material obtained from Rockhampton and Bellambi, NSW using the permeability apparatus (95mm diameter x 200mm height) shown in Figure B.2. The permeability values are $5 \times 10^{-4}$ mm/s and $2.6 \times 10^{-4}$ mm/s respectively. Gradations of the above fouling materials are shown in Figure B.3.
Figure B.2: Small scale permeability apparatus

Figure B.3. Particle size distribution of the fouling material obtained from Rockhampton (QLD) and Bellambi (NSW)
B.2. CLAY FOULED BALLAST

Fouling materials obtained from Sydenham and Thirroul, NSW were used for sieve analysis and Atterberg limit tests.

B.2.1 Atterberg Limits

Fouling material was separated from the samples of clay fouled ballast obtained from Sydenham and Thirroul, NSW and then the liquid limit and plastic limit values were determined as shown in Table B.2.

Table B.2. Atterberg Limits and classification of fouling material

<table>
<thead>
<tr>
<th>Clay fouling material</th>
<th>L.L</th>
<th>P.L</th>
<th>P.I</th>
<th>Fouling Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1-From Sydenham</td>
<td>51.5</td>
<td>20</td>
<td>31.5</td>
<td>CH</td>
</tr>
<tr>
<td>Sample 2-From Sydenham</td>
<td>51</td>
<td>22</td>
<td>29</td>
<td>CH</td>
</tr>
<tr>
<td>Sample 3-From Sydenham</td>
<td>50.5</td>
<td>21</td>
<td>29.5</td>
<td>CH</td>
</tr>
<tr>
<td>Sample -From Thirroul</td>
<td>48</td>
<td>23</td>
<td>25</td>
<td>CL</td>
</tr>
<tr>
<td>Mixture of 25%kaolin + 75% beach sand</td>
<td>19.1</td>
<td>15</td>
<td>4.1</td>
<td>ML</td>
</tr>
<tr>
<td>Mixture of 50%kaolin +50% beach sand</td>
<td>27.5</td>
<td>20.4</td>
<td>7.1</td>
<td>ML</td>
</tr>
<tr>
<td>Mixture of 75%kaolin + 25% beach sand</td>
<td>40</td>
<td>20</td>
<td>20</td>
<td>CL</td>
</tr>
<tr>
<td>100%kaolin</td>
<td>52.1</td>
<td>26.4</td>
<td>25.7</td>
<td>CH</td>
</tr>
</tbody>
</table>

Plasticity chart in Figure B.4 was used to classify the fouling material.
Figure B.4 Plasticity chart: USCS based on Wagner, A. A. (1957), Proceedings of the 4th ICSMFE, by permission of Butterworth & Co. (ref.: Holtz & Kovacs 1981)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Specific gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1-From Sydenham</td>
<td>2.44</td>
</tr>
<tr>
<td>Sample 2-From Sydenham</td>
<td>2.34</td>
</tr>
<tr>
<td>Sample 3-From Sydenham</td>
<td>2.6</td>
</tr>
<tr>
<td>kaolin</td>
<td>2.51</td>
</tr>
<tr>
<td>beach sand</td>
<td>2.602</td>
</tr>
</tbody>
</table>

Gradation of the fouling materials obtained from the above sites are shown in Figure B.5.
Figure B.5. Particle size distribution of the fouling material obtained from Sydenham and Thirroul (NSW)
## APPENDIX-C

RISK ASSESSMENT AND SAFE WORK PROCEDURE FOR LARGE SCALE TRIAXIAL TEST APPARATUS

### C.1. RISK ASSESSMENT

<table>
<thead>
<tr>
<th>Risk Assessment Task/ Location</th>
<th>Large Scale- triaxial apparatus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Person Conducting the Risk Assessment</td>
<td>Nayoma Tennakoon</td>
</tr>
<tr>
<td>Supervisor of the Area</td>
<td>Dr.Cholachat Rujikiathmajorn</td>
</tr>
</tbody>
</table>

Referenced UOW Guidelines, Legislation, Australian Standards, Code of Practice:

<table>
<thead>
<tr>
<th>No.</th>
<th>1.1.1.1.1 Hazard Identification</th>
<th>Risk Assessment</th>
<th>Risk Control</th>
<th>Review</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>What harm can happen to people or equipment</td>
<td>Risk 1.1.1.1.1</td>
<td>List any Control Measures already implemented</td>
<td>Describe what can be done to reduce the harm</td>
</tr>
</tbody>
</table>

---

222
<table>
<thead>
<tr>
<th>No.</th>
<th>Hazard Identification</th>
<th>Risk Assessment</th>
<th>Risk Control</th>
<th>Review</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>What harm can happen to people or equipment</td>
<td>List any Control Measures already implemented</td>
<td>Describe what can be done to reduce the harm</td>
<td>Whom Responsible</td>
</tr>
<tr>
<td>01</td>
<td>Dropping down heavy parts of the cell and lifting heavy weights</td>
<td>M Use of a lifting device and laboratory safety guidelines. Training</td>
<td>Proper coupling before lifting. Not to lift heavy weights manually and use of protective equipment</td>
<td>User</td>
</tr>
<tr>
<td>02</td>
<td>Injuries due to moving parts when applying the load</td>
<td>M Keep clear of the equipment when it is switched on and well documented operational procedures. Proper training on use of the equipment</td>
<td>Follow the guidelines in operational procedures. Handle the equipment with care</td>
<td>User</td>
</tr>
<tr>
<td>No.</td>
<td>What harm can happen to people or equipment</td>
<td>Risk</td>
<td>List any Control Measures already implemented</td>
<td>Describe what can be done to reduce the harm</td>
</tr>
<tr>
<td>-----</td>
<td>------------------------------------------</td>
<td>------</td>
<td>-----------------------------------------------</td>
<td>--------------------------------------------</td>
</tr>
<tr>
<td>03</td>
<td>Oil and water spills, pump leakages</td>
<td>L</td>
<td>Routine inspection procedures and availability of cleaning equipment</td>
<td>Regular checkups before operating the equipment. If something goes wrong instant switching off of the equipment</td>
</tr>
<tr>
<td>04</td>
<td>Tightening and removing screws, dismantalling parts of the cell</td>
<td>M</td>
<td>Equipment handling guide lines. Training</td>
<td>Follow the operational procedures and use correct tools for removal</td>
</tr>
<tr>
<td>No.</td>
<td>What harm can happen to people or equipment</td>
<td>Risk 1.1.1.1</td>
<td>List any Control Measures already implemented</td>
<td>Describe what can be done to reduce the harm</td>
</tr>
<tr>
<td>-----</td>
<td>------------------------------------------</td>
<td>-------------</td>
<td>-----------------------------------------------</td>
<td>---------------------------------------------</td>
</tr>
<tr>
<td>05</td>
<td>Injuries due to sharp objectives</td>
<td>M</td>
<td>Use of gloves and proper clothing.</td>
<td>Correct training.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ask for help when requires.</td>
</tr>
<tr>
<td>06</td>
<td>Eye injuries due to air/water hose</td>
<td>L</td>
<td>Safety guidelines</td>
<td>Use protective glasses.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Inspect the equipment</td>
<td>Correct training</td>
</tr>
<tr>
<td>07</td>
<td>hand/foot injury</td>
<td>N</td>
<td>Adequate space when using the lifting equipment.</td>
<td>Training in the use of lifting equipment</td>
</tr>
</tbody>
</table>
C.1.1. What is a hazard?

A  Could people be injured or made sick by things such as:
- Noise
- Light
- Radiation
- Toxicity
- Infection
- High or low temperatures
- Electricity
- Moving or falling things (or people)
- Flammable or explosive materials
- Things under tension or pressure (compressed gas or liquid; springs)
- Any other energy sources or stresses
- Biohazardous material
- Laser

B  What could go wrong?
- What if equipment is misused?
- What might people do that they shouldn’t
- How could someone be killed?
- How could people be injured?
- What may make people ill?
- Are there any special emergency procedures required?

C  Can workplace practices cause injury or sickness?
- Are there heavy or awkward lifting jobs?
- Can people work in a comfortable posture?
- If the work is repetitive, can people take breaks?
- Are people properly trained?
- Do people follow correct work practices?
- Are there adequate facilities for the work being performed?

D  How might these injuries happen to people?
- Broken bones
- Eye damage
- Hearing problems
- Strains or sprains
- Cuts or abrasions
- Bruises
<table>
<thead>
<tr>
<th>E</th>
<th>Imagine that a child was to enter your work area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>What would you warn them to be extra careful of?</td>
</tr>
<tr>
<td></td>
<td>What would do to reduce the harm to them?</td>
</tr>
<tr>
<td>F</td>
<td>What are the special hazards?</td>
</tr>
<tr>
<td></td>
<td>What occurs only occasionally—for example during maintenance and other irregular work?</td>
</tr>
</tbody>
</table>

- Are universal safety precautions for biohazards followed?
- Is there poor housekeeping? Look out for clutter
- Torn or slippery flooring
- Sharp objects sticking out
- Obstacles

- Burns
- Lung problems including inhalation injury/ infection
- Skin contact
- Poisoning
- Needle-stick injury
C.1.2. How to Assess Risk

Step 1 – Consider the Consequences
What are the consequences of this incident occurring? Consider what could reasonably have happened as well as what actually happened. Look at the descriptions and choose the most suitable Consequence.

<table>
<thead>
<tr>
<th>Consequence</th>
<th>Description</th>
<th>Likelihood</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major</td>
<td>Death or extensive injuries</td>
<td>A</td>
<td>The event is expected to occur in most circumstances</td>
</tr>
<tr>
<td>Moderate</td>
<td>Medical treatment</td>
<td>B</td>
<td>The event could occur at some time</td>
</tr>
<tr>
<td>Minor</td>
<td>First aid treatment</td>
<td>C</td>
<td>The event could occur, but only rarely</td>
</tr>
<tr>
<td>Insignificant</td>
<td>No treatment</td>
<td>D</td>
<td>The event may occur, but probably never will.</td>
</tr>
</tbody>
</table>

Step 2 – Consider the Likelihood
What is the likelihood of the consequence identified in step 1 happening? Consider this without new or interim controls in place. Look at the descriptions and choose the most suitable Likelihood.

<table>
<thead>
<tr>
<th>Description</th>
<th>Consequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>E</td>
</tr>
<tr>
<td>B</td>
<td>H</td>
</tr>
<tr>
<td>C</td>
<td>M</td>
</tr>
<tr>
<td>D</td>
<td>L</td>
</tr>
<tr>
<td>E</td>
<td>N</td>
</tr>
</tbody>
</table>

Step 3 – Calculate the Risk
1. Take step 1 rating and select the correct column
2. Take step 2 rating and select the correct line
3. Circle the risk score where the two ratings cross on the matrix below.

<table>
<thead>
<tr>
<th>Risk Score</th>
<th>E</th>
<th>H</th>
<th>M</th>
<th>L</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maj</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mod</td>
<td>A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Insignif.</td>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
</tr>
</tbody>
</table>

E = Extreme, H = High, M = Medium, L = Low, N = Negligible

Risk Score = .................
C.1.3. Risk Control

Risk control is a method of managing the risk with the primary emphasis on controlling the hazards at source. For a risk that is assessed as “high”, steps should be taken immediately to minimize risk of injury. The method of ensuring that risks are controlled effectively is by using the “hierarchy of controls”. The Hierarchy of Controls are:

<table>
<thead>
<tr>
<th>Order No.</th>
<th>Control</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>Firstly</td>
<td>Eliminate</td>
<td>Removing the hazard, eg taking a hazardous piece of equipment out of service.</td>
</tr>
<tr>
<td>Secondly</td>
<td>Substitute</td>
<td>Replacing a hazardous substance or process with a less hazardous one, eg substituting a hazardous substance with a non-hazardous substance.</td>
</tr>
<tr>
<td>Thirdly</td>
<td>Isolation</td>
<td>Isolating the hazard from the person at risk, eg using a guard or barrier.</td>
</tr>
<tr>
<td>Fourthly</td>
<td>Engineering</td>
<td>Redesign a process or piece of equipment to make it less hazardous.</td>
</tr>
<tr>
<td>Fifthly</td>
<td>Administrative</td>
<td>Adopting safe work practices or providing appropriate training, instruction or information.</td>
</tr>
<tr>
<td>Sixthly</td>
<td>Personal Protective Equipment</td>
<td>The use of personal protective equipment could include using gloves, glasses, earmuffs, aprons, safety footwear, dust masks.</td>
</tr>
</tbody>
</table>
C.2. SAFE WORK PROCEDURE

<table>
<thead>
<tr>
<th>Process/Equipment: Large-Scale Triaxial</th>
<th>Location: Rocks Lab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Procedure Developed by: Promod Thakur, Nayoma Tennakoon, Mousumi Mukherjee</td>
<td>Approved by: Dr. Cholachat Rijikiathmajorn</td>
</tr>
<tr>
<td>Date: 12/08/2008</td>
<td>Referenced UOW Guidelines, legislation, codes of practice, Australian Standards etc:</td>
</tr>
<tr>
<td></td>
<td>UOW OH&amp;S Policy</td>
</tr>
<tr>
<td></td>
<td>Occupational Health &amp; Safety Act 2000</td>
</tr>
<tr>
<td></td>
<td>Occupational Health &amp; Safety Regulation 2001</td>
</tr>
<tr>
<td></td>
<td>AS/NZS 1336: 1997 Occupational Eye Protection</td>
</tr>
<tr>
<td></td>
<td>AS/NZS 2210.1: 1994 Occupational Protective Footwear</td>
</tr>
<tr>
<td></td>
<td>AS/NZS 2161.1: 2000 Occupational Protective Gloves</td>
</tr>
<tr>
<td></td>
<td>AS/NZS 1269.3: 2005 Occupational Noise Management</td>
</tr>
<tr>
<td></td>
<td>AS/NZS 3760 Electrical Inspection &amp; Testing</td>
</tr>
<tr>
<td></td>
<td>Personal Protective Equipment Required (Check the box for required PPE):</td>
</tr>
</tbody>
</table>

**Personal Protective Equipment Required**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step Number</th>
<th>Activity (Steps in the process/task)</th>
<th>Hazards Identified (What could cause an injury)</th>
<th>Risk Score (How harmful is it)</th>
<th>Controls (What can be done to minimise the risk of injury)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ensure the work area is safe</td>
<td>-Tools</td>
<td>M</td>
<td>-Ensure Tidy/clean work area</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-Incorrectly kept heavy parts of triaxial cell and electric cables</td>
<td></td>
<td>-Make sure cell parts are securely positioned, and watch for electric cables</td>
</tr>
<tr>
<td>Step Number</td>
<td>Activity</td>
<td>Hazards Identified</td>
<td>Risk Score</td>
<td>Controls</td>
</tr>
<tr>
<td>------------</td>
<td>----------------</td>
<td>---------------------------------------------</td>
<td>------------</td>
<td>--------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>2</td>
<td>Preparation</td>
<td>- back injury</td>
<td>M</td>
<td>- Ask two or more people to help and handle properly</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- hand injury</td>
<td></td>
<td>- Use hard hat, gloves and steal capped shoes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- slip/fall down</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- head injury</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Sample Preparation</td>
<td>- Finger/Head injury</td>
<td>M</td>
<td>- Look actuator position</td>
</tr>
<tr>
<td></td>
<td>Remove/insert the outer lid</td>
<td></td>
<td></td>
<td>- Ask the trained person to use the forklift to remove/insert the outer lid</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- Use hard hat</td>
</tr>
<tr>
<td>4</td>
<td>Sample Preparation</td>
<td>- Back injury</td>
<td>M</td>
<td>- Ask somebody in the lab to help</td>
</tr>
<tr>
<td></td>
<td>Remove/insert the steel cap on top of the sample</td>
<td></td>
<td></td>
<td>- Use gloves and steal capped shoes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Hand injury</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Foot injury</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Testing</td>
<td>- Hand injury</td>
<td>M</td>
<td>- Use gloves, dust mask, safety glass, hearing protector</td>
</tr>
<tr>
<td></td>
<td>Pouring &amp; compacting of ballast</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Eye injury</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Hearing loss</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Dust inhalation</td>
<td></td>
<td></td>
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<td>Step Number</td>
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<td>-------------</td>
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<td>-------------------------------------------------</td>
<td>------------------------------</td>
<td>---------------------------------------------------------</td>
</tr>
<tr>
<td>6</td>
<td>Testing Starting of hydraulic machine</td>
<td>-Death</td>
<td>E</td>
<td>-Ask lab technician to teach how to start</td>
</tr>
<tr>
<td>7</td>
<td>Testing Loading/Unloading actuator</td>
<td>-Hand injury, -Head injury, -Hearing loss</td>
<td>M</td>
<td>-Use gloves, safety glass, hearing protector -Ensure all the switches are on/off</td>
</tr>
<tr>
<td>8</td>
<td>Data collection</td>
<td>-Hearing loss</td>
<td>M</td>
<td>-Use hearing protector</td>
</tr>
<tr>
<td>9</td>
<td>Testing complete Remove the water, take all the devices out of the triaxial equipment</td>
<td>-Fall/Slip</td>
<td>M</td>
<td>-Look carefully -Stop the actuator</td>
</tr>
<tr>
<td>10</td>
<td>Stop Hydraulics</td>
<td>-Death</td>
<td>E</td>
<td>-Ask lab technician to teach how to stop</td>
</tr>
</tbody>
</table>