AN ENGINEERING GEOLOGICAL INVESTIGATION

OF

FOOTWALL TOE-BUCKLE INSTABILITY

AT THE MALVERN HILLS OPENCAST COAL MINE,

INLAND CANTERBURY

A thesis

submitted in partial fulfilment

of the requirements for the Degree

of

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by

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Two spades and two supervisors, what would you do?

Use a bigger shovel!

“Do you think he is showing off for us?”
Joc Campbell, Supervisor
Abstract

A small opencast coal mine has been developed over previous underground workings in the Malvern Hills, inland Canterbury, New Zealand. The coal measure strata dip at ~45° to the southeast, and consist of finely laminated mudrocks with multiple coal seams of varying thickness. Production is in the range 10,000 to 15,000 tonnes per annum from two principal seams with an aggregate thickness of ~4.5m. The open pit has been designed with footwall batters parallel to bedding, vertical bench separation of 15m, and the highwall formed to a nominal 4V:1H.

Preliminary examination of the open pit mine site in 2003 indicated that footwall failures involved de-lamination due to drying out on exposure, and buckling and/or shearing along bedding surfaces. During mine development it became apparent that the batters formed easily where thin (less than 0.3m thick) coal seams were present in the sequence. In the 2004 campaign the pit floor was lowered, with a new batter and bench formed to expose the 3m thick Main Seam coal. The day after completion of this batter, a large buckle failure occurred involving the entire length of the pit (85m along strike), and a 2m thick intact slab with a total volume of ~3700m³ translated down dip 6.2m on the base of a thin coal seam to form a pronounced buckle at the toe. Even though footwall batters are cut to the angle of dip, which is entirely realistic geotechnically, the de-coupling and buckling that occurred compromised the safety and economics of the whole operation.

Buckling failure in moderately dipping soft rock sequences has been identified in footwall slopes of coal mining operations. Models used in the literature to simulate similar footwall failures include: the Euler solution using column and beam buckling theory to calculate the kinematic feasibility of a slab-buckle, conceptual modelling using a base friction table, and numerical modelling using distinct element analysis. Back analysis of the Malvern Hills failure was necessary to investigate the controls on the footwall stability, and for future mine design. Engineering geological description of the pit and slab materials was done, and an engineering geological model created. Samples of the slab material and failure surface were collected by coring and
trenching, with testing of these materials to establish the required parameters for use in the Euler solution. Back analysis using three different forms of the Euler solution provided unrealistic results that overestimated the overall length of a stable slope by more than 10 times. An engineering geology reassessment was undertaken, and a number of inadequacies in the Euler solution methodology were identified particularly in relation to pore pressure and elasticity considerations.

Given that the Malvern Hills toe-buckle slab failure displays both elastic and plastic deformation components in the soft mudrocks, and the slab itself cannot be considered as homogenous, reservations must exist about conventional predictive analytical techniques for pit slope failures of this type. No further large scale slab-buckle failures have developed at the mine site, in part because of the slow rate of coal extraction, but precautionary drainage of the footwall slopes has been undertaken to improve overall batter stability. The location of the slab-buckle failure on a critically positioned pre-sheared thin coal seam with full hydrostatic head is considered the most probable cause, rather than inherent instability of the generic bench and batter arrangement adopted. The adoption of a precedent based engineering geology approach to future mine design is considered the most appropriate solution in the circumstances.
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And to my son Brent, thank you for being you.
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1 Introduction

1.1 Project Background

This thesis examines a large, slab toe-buckle footwall failure of laminated mudrocks in the Broken River Formation at the Malvern Hills Opencast Mine, inland Canterbury (Figure 1.1). The toe-buckle failure was discovered on 1 July 2004 when the excavator operator arrived to extract the three metre thick coal seam that was exposed the day before. Sometime during the night a two meter thick slab had translated down dip 6.2m, and buckled at the toe partially covering the Main Seam coal.

Figure 1.1 Photo of the large toe-buckle, footwall failure that occurred on 1 July 2004 at the Malvern Hills Opencast Mine, inland Canterbury indicating bench levels and toe-buckle slab dimensions.
Chapter 1  Introduction

The Malvern Hills Coalfield, part of the Canterbury Coal Region, is situated along the foothills of the Southern Alps at the western edge of the Canterbury Plains between the Rakaia and Waimakariri Rivers, with the main strike of the coal measures extending northeast from Glenroy to Sheffield (Figure 1.2). The Late Cretaceous to Palaeocene coal measures, overlying the Torlesse basement, dip to the southeast due to back thrusting along the Hororata Fault, and terminate where they dip beneath the Harper Hills. Underground mining operations have been centred in the eastern half of the coalfield for the past 125 years, but only recently has it been economically viable to use opencast methods for coal extraction on a large scale. Two of the most important underground mines were the Homebush (1872 – 1938) and the Klondyke (1929 – 1972), with outputs of approximately 350,000 tonnes of coal each. Numerous smaller underground mines exist in the area, with small opencast mines developed over outcropping coal (Sara, 1972).

Figure 1.2  A location map of Malvern Hills Coalfield which is part of the Canterbury Coal Region on the South Island of New Zealand (after Duff & Barry, 1989). The Malvern Hills Coalfield overlies the Torlesse basement, strikes to the northeast between Glenroy and Sheffield, and dips beneath the Harper Hills to the northwest of the Hororata Fault (Duff and Barry, 1989).
The Malvern Hills Opencast Mine development began in April 2003 and is the first large scale opencast mine in the area. Two economic coal seams (Engine Seam and Main Seam) with a combined thickness of 4.5m are being worked. To achieve an economical pit design and minimize waste in the coal bearing units that are moderately dipping (45 ± 5°), the footwall batters are cut to parallel the dip of the bedding. The footwall design has 5m wide benches cut at 15m vertical intervals, creating batter dip slopes of 21m in length as seen in Figure 1.3 and Figure 1.4. Down-dip development of the pit exposed previous underground workings, a tight fold, and associated fault. The start up stripping ratios have been high, resulting in marginal economic feasibility.

Figure 1.3 Schematic cross section showing the mid-pit profile of the batter and bench configuration of the Malvern Hills Opencast Mine shortly after the 1 July 2004 large toe-buckle failure. The two main coal units (Engine Seam and Main Seam) dip at 45° in the footwall, but folding and corresponding thrust faulting of the seams occur in the highwall. The footwall pit design minimises waste by cutting the batters parallel to bedding at 15m vertical intervals separated by 5m wide benches. This thesis examines the large toe buckle failure that translated a 2m thick slab, 6.2m down dip, and buckled at the toe on the 21m long dip slope of the batter above the 355 RL bench the day after the batter was completely formed.
The majority of the coal falls in the New Zealand classification of sub-bituminous C with low sulphur content of in-ground coal (less than 1%) (Duff and Barry, 1989). Acid Mine Drainage (AMD) was not a problem during previous underground mining operations, but AMD has been identified in the new opencast mine and attributed to high intensity storm runoff over the long footwall dip slopes that have desiccated during dry summer months (Bell and Seale, 2004). Additional research has been done to further establish the source of AMD and remediate the problem (de Boer, 2005); (Alipate, 2005).

Figure 1.4  Photo taken on 4 Feb 2006, almost two years after the large toe-buckle failure, showing footwall benches, and long dip slope batters cut parallel to bedding. The smooth batter face shown between the marks is the base of the toe-buckle slide surface (after removal of the slab) only marginally altered by minor surface slaking and apron of debris at the base. The Main Seam has been excavated along strike in the trend of the base of the lowest batter. Note contrast with slope deposits on crest on top right indicative of small slab failures parallel to bedding.
By June 2004 the pit had been excavated ~100m along strike and 40m deep. When the operator left the site on 30 June 2004 the Main Seam coal was exposed to 350 RL and the footwall batter stratigraphically below the Main Seam was formed, completing the 21m dip slope batter to the 355 RL bench. On 1 July 2004 it was not possible to extract the exposed Main Seam coal as it was partially covered by a large toe-buckle failure that extended almost the entire strike of the pit. The footwall batter had failed and a slab (2.0m true thickness and 85m in length) translated down dip ~6m and buckled at the toe.

Buckling of beams and columns is described in structural engineering textbooks and the term has been used to describe buckling of geotechnical materials by a number of authors (Cavers, 1981; Goodman, 1976; Piteau and Martin, 1981). Bray and Goodman (1981) used a base friction table to physically model the kinematics of buckling failures and Stead and Eberhardt (1997) used numerical modelling to explain buckling failures in footwall slopes. This thesis examines the Malvern Hills Opencast Mine failure in the context of such failures and presents data relevant to geotechnical analysis.

1.2 Specific Aims and Objectives

This thesis analyses the mechanisms that allowed a large buckling footwall failure to occur on 1 July 2004 at the Malvern Hills Opencast Mine site, and applies this knowledge to the forward design of the pit slope.

Specific objectives of the study have been to:

- Describe the site engineering geology and develop a suitably detailed model of the 1 July 2004 failure.

- Characterise both the physical and mechanical properties of the materials involved using direct and indirect test methods.
Apply the Euler solution for column and beam buckling to assess the validity and sensitivity of this analytical method to account for the observed buckle failure geometry and dimensions.

Use the above techniques to develop an engineering geological model for the toe-buckle failure, and to recommend forward mine design and practice.

1.3 Regional Geological Setting

The Canterbury Coal Region is made up of small localised coal deposits in the foothills along the eastern margins of the Southern Alps from the Hurunui River in North Canterbury to the Opihi River in South Canterbury. This study of buckling footwall failures is located at the Malvern Hills Opencast Mine in the Malvern Hills Coalfield, one of several identified coal fields in the Canterbury Coal Region (Figure 1.5). The inferred in-ground coal resource for the Canterbury Coal Region is 3.54Mt (discounting coal less than 1 m thick), of which the Malvern Hills Coalfield is estimated to contain 3.20Mt (Barry et al., 1994).

![Figure 1.5](Image)
1.3.1 Stratigraphic Sequence

The coal resource in the Canterbury Coal Region is from the terrestrial Broken River Formation, which was deposited in the Late Cretaceous to Early Paleocene at the beginning of the Tertiary transgressive-regressive cycle between the Rangitata and Kaikoura Orogenys. The Broken River Formation is of variable thickness (up to 1500 m) and overlies the Munro Conglomerate, which is of Late Cretaceous age (Figure 1.6). This conglomerate is of local extent and extremely variable thickness and is derived from local erosion. It is deposited onto a Late Cretaceous unconformity, eroded into Late Triassic greywacke and argillite, Jurassic Wakaepa Plant Beds, and Early Cretaceous Mt Misery Volcanics. The Broken River Formation is overlain by the Late Cretaceous to Early Paleocene Conway Formation, dominated by micaceous and quartz rich fine sandstones and mudstones indicative of littoral to shallow marine settings (Field and Browne, 1989).

In the Malvern Hills Coalfield, Duff and Barry (1989) divided the stratigraphy into three units. The lowest unit (Munro Conglomerate) was 800 to 1000m thick, described as a fining upward sequence of conglomerate, grit, sandstone and claystone with thin interbedded and uneconomic coal seams that represents terrestrial deposition. The middle unit (Broken River Formation) was up to 200m thick, consisting of coal seams, thick carbonaceous mudstone and very fine to fine quartz sandstone. The upper unit (Conway Formation) comprises of micaceous fine sandstones and mudstones. The Malvern Hills Opencast Mine is located entirely within the Broken River Formation. Coal seam geometry is complex, with overlapping lensoidal coal seams of limited extent deposited insitu, and transported or drift peat accumulations (Duff, 1986). Coal seams are highly variable in thickness with a maximum thickness found in the Malvern Hills Coalfield of 10 metres (Duff and Barry, 1989) with ~3m thick seams common.
Figure 1.6 Generalised stratigraphy of Malvern Hills Coalfield Region (Carlson et al., 1980; Duff, 1986; Duff and Barry, 1989; Field and Browne, 1989; Mathews, 1989; Tappenden, 2003). Note that Malvern Hills Opencast Mine is located solely within the Broken River Coal Measures.
1.3.2 Geology and Tectonics

Subsequent to the deposition of the Broken River Coal Measures the succession was affected by local volcanic activity in the Eocene (deposition of the View Hill Volcanics) and faulting and uplift in the Eocene and later in the Pliocene. Further volcanic activity also occurred in the Miocene, with the deposition of the Harper Hills Basalt and associated tuffs and bentonite clay deposits (Figure 1.7).

![Figure 1.7 Generalised map of Malvern Hills Coalfield showing geological units and faults with locations of mines noted in the text. Modified after Carlson et al., 1980; Duff, 1986; Duff and Barry, 1989; Field and Browne, 1989; Mathews, 1989; Tappenden, 2003. Refer to Figure 1.6 for details of the stratigraphic units.](image)

Late Tertiary deformation was localised on the previously weakened rock mass and created zones of intense shearing and faulting. Therefore the local stresses resulted in a series of non uniform folds and associated faulting of the covering sedimentary succession. Faulting and uplift occurred during the Late Eocene, and later uplift in the Pliocene resulted in the development of the range front system that has a pronounced north-east alignment. Distinct tilted fault blocks were created (Duff,
1986; Duff and Barry, 1989; Field and Browne, 1989). As a result of the tectonic uplift and igneous intrusions the Cretaceous to Tertiary sediments in the region generally dip to the southeast.

Figure 1.8  Map showing partial traces of principal faults in the area of the Malvern Hills Coalfield from literature review. Modified after Bradshaw, 1975; Mathews, 1989; McMorran, 2002.

There are several known faults with a northeast strike within 30 km of the Malvern Hills Coalfield. The Porters Pass Fault, an extension of the Porters Pass- Amberly Fault Zone, lies 27 km to the north of the Coalfield. The Springfield Fault strikes northeast from the upper Selwyn River to Springfield (14 km north of the coal field) and has had repeated movements in the last 10,000 years. The Hororata Fault is located 3km to the south of the coalfield on the Canterbury Plains and has deformed the fluvio-glacial gravels suggesting a similar formation history to the Springfield Fault (Figure 1.8) (McMorran, 2002). The coal measures in the Malvern Hills Opencast Mine have been interpreted to be on the downthrow side of the Cairn Hill-
Mt Misery Fault and localised warping was caused by igneous intrusions during the Oligocene (Bradshaw, 1975; Gage, 1970; Mathews, 1989).

Recently several more faults were found whilst investigating the area for a regional landfill and down dip excavation of the Malvern Hills Opencast Mine. During the landfill investigations a west dipping thrust fault (referred to as the Western Gully Fault) was found at the contact between the coal measures and basement (McMorran, 2002). East dipping reverse faults were also found outcropping to the south of the Western Gully Fault and are inferred to be active due to displaced Holocene colluvium. During the same investigation drill core examinations revealed slickensided joints that were interpreted to represent small amounts of displacement along bedding within the Tertiary sediments (McMorran, 2002). While excavating the highwall in the Malvern Hills Opencast Mine in May 2004 an anticlinal fold and related fault dipping to the south were discovered, in what were previously thought to be continuous southeast dipping beds. The presence of folding within the mine confirmed that investigations were needed to explain bedding plane shearing in the footwall that may contribute to slope toe buckling.

1.4 Climate and Land Use

The Malvern Hills Coalfield is geographically located in the rolling hills 250 – 400m a.s.l. at the interface of the Southern Alps and the western edge of the Canterbury Plains. The nor’west “arch”, a gap of clear sky against which the ranges are clearly outlined, signals the arrival of a weather system from the west with heavy rain in the upper reaches of the Waimakariri and Raikaia rivers and strong winds funnelling down the valleys onto the Canterbury Plains. This results in higher rainfalls closer to the Southern Alps that decrease to the southeast. Rainfall averages per year are: Malvern Hills 1000+mm/year, Darfield 790mm/year and Christchurch 640mm/year (Figure 1.9) (Mabin et al., 2005).
The Malvern Hills area was settled by the Deans family in the late 1800s and used for sheep grazing. Homebush, the family homestead and historic wool shed, were built from bricks fired by the coal and made from the clay of the Broken River coal measures (Figure 1.10). The township of Glentunnel was established in the 1870s with the opening of the first underground coal mine (Homebush Mine), and the establishment of the Glentunnel Tile Brick and Pottery Works. A branch railway line from Darfield connected the area to Christchurch in November 1875, and transported the coal, sand and other products from the “Works”. These included 16 types of fine bricks (eg chimney pots, tiles, and drain pipes) and some 43 varieties of moulded bricks (Knowles, 1990).
Figure 1.10  Photo of Homebush, the original Deans Family homestead, established in the late 1800s. The bricks were made from locally sourced clay and fired by coal mined in the area.

The growth of Christchurch in the late 1800s and early 1900s relied heavily on the brick and coal sourced from Malvern Hills, and on the building stone from the quarries in the Port Hills. The Malvern Hills can therefore be seen as a primary contributor to the early development of the city, and the area remains a potential source of non-metallic mineral products. Today, the brick and coal are used locally (i.e. in Canterbury), and the bentonite quarried from the nearby Harper Hills is shipped world-wide.

Pastoral farming of sheep, cattle, deer, and ostrich now occupy the foothills, and the Selwyn Plantation board manages an established radiata pine plantation whose annual overall growth rates are significantly higher than on the Canterbury Plains.

The controversial and privately funded Central Plains Water Enhancement Scheme is in the planning stages of building a dam (2000m long, 55m maximum height) at the mouth of the Waianiwaniwa Valley (immediately to the southeast of the Malvern Hills Opencast Mine) to store water from the Rakaia and Waimakariri Rivers for irrigation of the Canterbury Plains. This storage reservoir would flood the most fertile
grazing land, cover the northern projected extent of coal, impede access to existing mining activities, sterilise some coal resources, and create new AMD issues.

1.5 Mining History

Coal mining in the Canterbury Coal Region began in 1864 with the development of the Mt Somers Coal Mine, which until its recent closure was the oldest working coal mine in New Zealand. Mining has occurred continuously in both the Mt Somers Coalfield and the Malvern Hills Coalfield and of the 120+ mines in the region approximately 77 are located in the Malvern Hills Coalfield. The greatest producers were the underground mines Homebush (1872-1938) and the Klondyke (1925-1972), with ~350,000 tonnes of coal each. Other notable mines of over 40,000 tonnes each are the Springfield, Steventon 2, Victory 2, and Avenue (Figure 1.7) (Duff and Barry, 1989).

The stacked coal seams in the Malvern Hills Coalfield tend to be elongated to the northeast and southwest, dip steeply, and thin up and down dip. Coal was won easily by shallow opencast methods where coal seams crop out near the surface. The underground mines had relatively short working lives (i.e. a few years) as it was easier to develop another seam along strike than to work up or down the dip. Seam thinning, dirty coal, faulting, high sulphur content of overburden and interburden, flooding, and decreasing rank away from igneous intrusions also limited the extent of mining.

In the late 1980s there were several mines in operation in the Malvern Hills Coalfield. The Klondyke Extension Opencast mined four steep seams with a maximum individual seam thickness of 3m (Duff and Barry, 1989). In the immediate vicinity of the present Malvern Hills Opencast Mine there were three underground mines operated by Nimmo Colleries: the Hydro Mine, Rolly’s 86, and Malvern Hills. Workings from small opencast and underground mines have been exposed in the Malvern Hills Opencast Mine pit, and the three Nimmo mines will be exposed as the pit progresses down dip and along strike.
1.6 Thesis Methodology

1.6.1 Background to Project

Prior to formally establishing the topic of this thesis research and commencing the research project there had been the opportunity to follow the establishment phase and history of the pit.

My observation of the Malvern Hills Opencast Mine area began at start-up (January 2003) working as a field assistant for David Bell. The task was to investigate the footprint for a settling pond to control the runoff from the mine footprint and evolved to the first excavation of the pit in April 2003.

In July 2003 two benches were established at 385 RL and 370 RL with the pit floor at 360 RL. In October 2003 the ENGE 471 and ENGE 474 courses that I was enrolled in used the site for class exercises. At the end of the year I wrote a Masters thesis proposal to investigate the possibility of footwall buckle failures of the batters on the basis of a relatively smaller failure between 385 RL and 370. RL. As part of my thesis research survey pegs were inserted in the footwall (December 2003) and monitored over several months with negligible movement. No further excavation or mining occurred from July 2003 to June 2004.

In June 2004 I took a study leave to work as the Engineering Geology technician in the department. On 1 July 2004 the large toe-buckle footwall failure occurred and became the focus of investigation. During this period of the mining campaign further excavation had taken place to reveal a fold and thrust fault in the highwall.

I returned to study in July 2005 and have been fortunate now to have observed the mine for three years. This long-term observation of the pit development is useful for back analysing the footwall failure, and in understanding the need for a forward plan for footwall stability of long dip slopes for the mine life.
1.6.2 Toe-Buckle Investigation

Footwall stability is more important than highwall stability for this mine, as the latter failures are small in volume ($\geq 1000 \text{m}^3$) and rarely disrupt extraction. Footwall failures will result in disruption of mining while the debris is removed from the pit floor and the batter re-established. Large footwall slide failures may result in partial closure of the pit or cause safety concerns whereas highwall failures are likely to be joint controlled and of small volume.

To investigate and back analyse the large toe-buckle footwall failure that occurred on 1 July 2004, and to establish a forward plan for stable batter design for the future mine life, the following methods have been used:

- Engineering geological mapping of the Malvern Hills Opencast Mine area to establish the structure of the pit and the relationship of folding and faulting to shear surface produced by flexural bedding slippage. Site plans and cross sections of the pit before and after the large 1 July 2004 buckle failure are produced to provide the engineering geology for geotechnical analysis.

- Hand digging, coring, trenching, and in-situ testing were undertaken to log and characterise the slab structure and failure surface of the toe buckling footwall failure.

- Test samples were collected in the field from hand digging, coring, and trenching in order to characterise their physical properties, and to obtain necessary input parameters.

- Input physical properties of the slab material and slab failure parameters into the structural engineering concept of the Euler solution for column and beam buckling. Use the above results to formulate a forward plan for stable footwall slopes for the mine life.
1.7 Thesis Format

Chapter 2 describes the engineering geology of the mine along with plans and cross sections incorporating the geology, tectonics, and pit structure relationships. The pit before and after the 1 July 2004 failure is modelled in suitable detail, and the pit development since the failure is presented.

Chapter 3 describes the engineering geology of the toe-buckle slab and the role the footwall stratigraphy plays in the stability and configuration of the footwall in respect to the large toe-buckle failure on 1 July 2004. The toe-buckle failure is described in detail and an engineering geological model of the toe-buckle failure is created.

Chapter 4 characterises the footwall materials detailed in the 1 July 2004 failure model using both direct and indirect testing methods. As these materials are classified as “soft” rocks both soil and rock mechanics tests were conducted. Soil tests such as particle size, moisture content, particle density, soil shear box and ring shear were done and where appropriate rock tests such as rock shear box, slake durability, Schmidt hammer and sonic velocity. Testing was done to establish the material parameters for use in the Euler solution.

Chapter 5 presents the structural engineering concept of the Euler solution for column and beam buckling, and how it has been used to model footwall buckling. The solution is applied to the material parameters and slope geometry at the Malvern Hills mine site and then back analysed and sensitivity tested to model the footwall failure of 1 July 2004.

Chapter 6 is an engineering geology reassessment of the toe-buckle failure following on from the findings of Euler analysis.

Chapter 7 provides a summary and the conclusions of the study.
Chapter 2 Mine Engineering Geology

2.1 Introduction and Site Overview

This chapter examines the Malvern Hills Opencast Mine site with the primary objective of formulating, in suitable detail, an engineering geological interpretation of the pit before, and after, the large toe-buckle failure. The mine stratigraphy, structural setting, and stability of the pit walls are illustrated using plans and cross sections.

An aerial photo of the mine area taken in 2005 by Selwyn District Council shows the pit configuration as it was after the large toe-buckle failure material was removed (Figure 2.1). On the left (western) side of the photo, the opencast pit is visible with the footwall to the northwest, and the highwall to the southeast. It is possible to see all the benches in the footwall and the Main Seam coal on the pit floor. In the centre of the photo is the settling pond where the water from the pit and access roads is directed. The largest spoil heap is on the right (eastern) side of the photo, and reflects the volumes of overburden and interburden that were excavated to win coal from the 1.5m thick Engine Seam and the 3m thick Main Seam up to that time (85,000m$^3$).

The Engineering Geological Sketch Plan of the Malvern Hills Opencast Mine (Figure 2.2) is the basis for much of the information presented in this chapter and it also provides a good overview of the mine area when compared with the aerial photo in Figure 2.1. In the sketch plan the base topography and the outlined Pit Area were provided by the Connell Wagner survey in December 2004 (Connell-Wagner, 2004), although the topography in the plan has been adjusted to correspond with the pit configuration on 3 July 04 immediately following the removal of the material involved in the toe-buckle failure. At that time the pit floor was developed to 352 RL and the Main Seam coal was exposed down dip from the 355 RL bench to 352 RL pit floor along the entire length of the pit. The toe-buckle footwall failure (1 July 2004) occurred on the batter between the 370 and 355 RL benches (shown in red in Figure 2.2), and the topography of the batter in the sketch plan correlates to the failure surface of the toe-buckle failure after the failed slab material removal.
Figure 2.1 Aerial photo taken in 2005 of the Malvern Hills Opencast Mine. On the left is the pit showing three footwall benches, the pit floor with exposed Main Seam coal, and the highwall. The pit configuration is as it was after removal of the toe-buckling material and Main Seam coal. Note the settling pond in the centre of the photo where runoff water from the access tracks and pit is directed. Overburden and interburden are moved to the spoil heap on the right side of the photo (Aerial photo courtesy of Selwyn District Council).
Figure 2.2 Engineering Geological Sketch Plan of Malvern Hills Opencast Mine site showing strike and dip of bedding, folding, faulting and three cross section lines.
The format for the remainder of the chapter is as follows:

- Section 2.2 describes the mine stratigraphy: the overburden, interburden and footwall materials found in the open cast mine.

- Section 2.3 presents the engineering geological descriptions of the materials in the pit footprint and the implications of using soil and/or hard rock descriptions for the soft rock material.

- Section 2.4 develops the mine footprint structural setting and relates the mine stratigraphy to the structure of the pit using several cross sections positioned perpendicular to the toe-buckle failure plane.

- Section 2.5 generalises the hydrogeology of the area.

- Section 2.6 examines issues of the highwall stability and subsequent pit development.

- Section 2.7 is a synthesis of the mine engineering geology.

### 2.2 Mine Stratigraphy

#### 2.2.1 Mapped Units

The geology exposed within the “Pit Area” shown on Figure 2.2 is presented in Figure 2.3 as a stratigraphic column divided into three sections. The **Highwall** or overburden includes a thin coal seam that is uneconomic to mine and sandy (as well as silty) materials. The **Pit floor** contains the mineable seams, specifically the 3m thick Main Seam and the overlying 1.5m thick Engine Seam #1, with laminated mudstone/siltstone interburden. And the **Footwall** materials are predominately laminated mudstone/siltstone but can be locally carbonaceous and contain thin (<0.5m thick) coal that is uneconomic to mine.
Figure 2.3 Stratigraphic column of the Broken River Formation exposed in the mine footprint.
The sequence exposed in the Malvern Hills Opencast Mine is part of the Broken River Formation deposited in the Late Cretaceous to Early Palaeocene, as noted in Chapter 1, Figure 1.6. In the pit the bedding is dipping at ~40 to 45° to the southeast, and the pit footwall, as shown in cross section in Figure 1.3, has been formed parallel to the bedding. The units in the opencast pit are categorised by where they are exposed in the pit (highwall, pit floor, or footwall) because their material properties and bedding orientations have an impact on the slope stability which is analysed in following chapters.

2.2.2 Pit Stratigraphic Column

As shown in Figure 2.3 the highwall and overburden immediately above the 1.5m thick Engine Seam in the pit floor is a laminated mudstone/siltstone, covered by a green sand, a thin coal seam with lignite lenses, laminated mudstone/siltstone, and capped with Late Pleistocene loess colluvium. The pit floor exposes the 3m thick Main Coal Seam, 4m of laminated mudstone/siltstone, and the 1.5m thick Engine Seam. The footwall exposes lengths of dip slope surfaces of 21m between benches and is comprised of laminated mudstone/siltstone interbedded with thin carbonaceous layers and coal seams. A sandstone unit is exposed on the batter above the 385 RL bench.

In the stratigraphic column (Figure 2.3) seven coal seams are represented (one in the highwall, two mineable seams in the pit floor, and four seams in the footwall), yet it is important to note that these are not the only seams recognised as thin coal seams are present throughout the pit sequence. Frequently the laminated mudstone/siltstone grades into more carbonaceous mudstone with thin coal lenses (0.1 – 0.5m). The thin coal seams shown in the column can be traced along the entire strike length of the pit, and therefore have been singled out and represented in the column.

As shown in Figure 2.3, the overburden forming the highwall lies immediately above Engine Seam #1. It comprises the following sub units:

- A basal greensand including laminated mudstone/siltstones, totalling ~2.5m in thickness.
• Engine Seam #2 coal seam and laminated carbonaceous mudstones/siltstones, ~0.5m thick with 5-10cm thick muddy lignite lensing at a scale of several metres laterally.

• A laminated mudstone/siltstone with silicified channel fill of variable thickness but typically in the range of 5-10m thick.

• Higher in the sequence, but not shown in Figure 2.3 as it was not exposed at the time of the toe-buckle failure, is a yellowish, quartz rich, medium sand, at least 5m thick.

• A loess colluvium cap of ~2m thick unconformably blankets the landscape and the underlying Broken River Coal Formation.

The pit floor and interburden consists of three sub-units, two of which are the producing coal seams and the third is the ~4m thick interburden. Key features of these three sub-units from the base up are:

• The Main Seam is typically 3m thick over the 100m strike length of the exposure, and is characterised by its massive nature with little or no silicified lenses or non-coal partings.

• The Interburden is a laminated creamy white and light grey mudstone/siltstone, with laminations on the scale of 5-10mm and is locally carbonaceous.

• The Engine Seam #1 is approximately 1.5m thick and contains ~20cm thick silicified sandstone lenses/channels.

The footwall, which by definition commences immediately below the Main Seam, consists of alternating carbonaceous laminated mudstone/siltstones and thin coal seams with sandstone below. Key sub-units are as follows:
• A bottom mapped unit of weak, greyish white, massive sandstone of unknown thickness underlies the laminated mudstone/siltstone and is exposed on the batter above the 385 RL bench.

• The laminated mudstone/siltstone above the massive sandstone is more carbonaceous than the interburden. The laminations are greyish white and greyish black and the units contain four thin interbedded coal seams (~0.1 to 0.5m thick). It is important to note that these seams contain broken coal, and that extend over the full strike and dip length of the mine footprint.

2.2.3 Laminated Mudstone/Siltstone

The mudstone/siltstone characteristics change throughout the sequence at a scale of 5-10cm. A generalised scaled photo of the laminations is shown in Figure 2.4. In this photo of sample taken from below the Main Seam coal it is possible to see the waviness of the lamination and continuity of a bedding surface across the specimen.

![Figure 2.4 Close up scaled colour photo of laminated mudstone/siltstone showing bedding relationships of the alternating layers, where the lighter siltstone forms discontinuous stringers and flaser bedded lenses.](image-url)
As seen in the photo (Figure 2.4) and in hand specimen this material has a visibly dominant fraction of darker muds laminated with a subordinate fraction of lighter coloured silts and therefore has been labelled as a mudstone/siltstone in this thesis.

As mentioned earlier, the mudstone/siltstones vary in both their carbonaceous content and degree of channelling. Another variation is in colour. The highwall laminations are greyish white and greyish black, the interburden is creamy white and light grey (i.e. not as carbonaceous), and the footwall is greyish white to greyish black with more numerous carbonaceous muds and thin coal seams.

The strength of the laminated mudstone/siltstones varies considerably. When cool and wet to moist a hand size specimen of the mudstone/siltstone will need an implement to break a piece off. Yet as the specimen dries it loses strength, a moist to dry mudstone/siltstone will part on a bedding plane, and a dry sample will sheet apart on the bedding plane (on a scale of 3-8mm) as easily as spreading a deck of cards. This characteristic is discussed further in Chapter 4 and has a strong implication for small nuisance failures of the drying footwall slopes, giving them a tendency to slake and slab when desiccated.

2.3 Engineering Geological Descriptions and Implications

In Figure 2.3 the units are described in geological descriptive terms. Table 2.1 has referenced both the geological descriptions and the engineering geological descriptions (rock or soil, as appropriate) of the material illustrated in the stratigraphic column of the mine footprint. Engineering geological descriptions of soil and rock material are especially useful for characterising the material geotechnically as the descriptions include weathering and strength terms and in the case of soils a water content and Unified Soil Classification System term (Appendix B). Table 2.1 is divided into three sections: Highwall and Overburden, Pit Floor and Interburden, and Footwall. Soil material descriptions were used for the loess colluvium and green sand of the highwall, whereas all the other materials in the mine footprint were described using rock material descriptions.
<table>
<thead>
<tr>
<th>Material</th>
<th>Geological Description</th>
<th>Engineering Geological Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loess Colluvium</td>
<td>Glacial-derived loess (SILT) and colluvium containing 2-3cm sandstone cobbles and up to 0.5m ferricrete (iron cemented) blocks.</td>
<td>Slightly weathered, dry, compact, yellowish brown, massive SILT containing ferricrete blocks up to 0.5m and cobbles of sandstone colluvium. USCS classification ML.</td>
</tr>
<tr>
<td>Laminated</td>
<td>Alternating 1-2mm thick, laminated and locally flaser bedded siltstone and mudstone with channels of silicified sandstone. Siltstone is light grey, poorly indurated, laminated quartzose siltstone. Mudstone is dark grey, poorly indurated, laminated, carbonaceous mudstone. Channels are grey, well indurated lenses, of bedded very fine to fine sandstone, quartz arenite.</td>
<td>Unweathered, moderately weak to weak, greyish white and greyish black, very finely layered MUDSTONE/SILTSTONE. Contains lens shaped channels of silicified sandstone up to 0.2 m thick.</td>
</tr>
<tr>
<td>Mudstone/Siltstone</td>
<td>Engine Seam #2 Coal seam of sub-bituminous C rank coal containing 5-10cm thick muddy lignite lens.</td>
<td>Unweathered, weak, black, layered, COAL and carbonaceous muds containing 5-10cm thick muddy lignite lens.</td>
</tr>
<tr>
<td></td>
<td>Green Sand Green, loose finely bedded, well sorted very fine to fine sandstone, quartz arenite. Fines upwards and is interbedded with laminated mudstone/sandstone and muddy coal lens.</td>
<td>Unweathered, moist, loose, greenish grey, finely layered, medium to fine SAND with some interbedding of laminated mudstone/siltstone.</td>
</tr>
<tr>
<td>Pit Floor &amp; Interburden</td>
<td>Engine Seam #1 Coal seam of sub-bituminous C rank coal. Fresh face has concoidal fracture pattern. Interbedded lens (20cm thick) of dark to medium grey, well indurated, silicified, very fine sandstone quartz arenite.</td>
<td>Unweathered, weak, black, layered, sub-bituminous C rank COAL with interbedded lens (20cm thick) of strong, dark to medium grey, finely layered silicified sandstone.</td>
</tr>
<tr>
<td>Laminated</td>
<td>Alternating 1-2mm thick, laminated locally flaser bedded siltstone and mudstone. Siltstone is creamy white, poorly indurated laminated quartzose siltstone. Mudstone is light grey, poorly indurated, laminated carbonaceous mudstone.</td>
<td>Unweathered, moderately weak to weak, creamy white and light grey, very finely layered MUDSTONE/SILTSTONE.</td>
</tr>
<tr>
<td>Mudstone/Siltstone</td>
<td>Main Seam Coal seam of sub bituminous C rank coal. Massive, no visible mud or sand.</td>
<td>Unweathered, moderately weak (breaks down to chips), black, massive, sub-bituminous C rank COAL.</td>
</tr>
<tr>
<td>Footwall</td>
<td>Alternating 1-2mm thick, laminated locally flaser bedded siltstone and mudstone. Siltstone is medium grey, poorly indurated laminated quartzose siltstone. Mudstone is very dark grey, poorly indurated laminated carbonaceous mudstone. Sequence contains several thin (3-10mm) thick coal seams/carbonaceous muds.</td>
<td>Unweathered, moderately weak to weak, greyish white and greyish black, very finely layered MUDSTONE/SILTSTONE.</td>
</tr>
<tr>
<td>Sand</td>
<td>Greyish white, friable, massive, fine sand, quartz arenite.</td>
<td>Unweathered, weak, greyish white, massive SANDSTONE.</td>
</tr>
</tbody>
</table>
Most of the material in the pit can be classified as soft rock, with the exception of some silicified sandstone “channels” within the Engine Seam #1 and the paleo-channels in the mudstone/siltstone unit above Engine Seam #2. Even the Main Seam coal is loose or breaks into chip sized pieces easily. In later chapters soil material descriptions are applied to the footwall materials as these more appropriately apply to the soft rock material involved in the toe-buckle failure.

It is important to note that the materials in the mine footprint, when described in engineering geological terms, are weak to moderately weak rocks or loose soils and therefore a hand specimen can be broken by hand and/or finger pressure or a sample can be removed from the exposure in a disaggregated form by hand. The hardness of the laminated mudstone/siltstones units are variable (from weak rock to loose soil), and some of these can be described as engineering soils because they are not volumetrically stable when immersed in water. Hence the stability of the material when saturated has implications that will be examined further in Chapter 4.

Since there is difficulty in adequately describing the pit materials as hard rock or soil, it is assumed they are “soft” rock materials. This assumption has implications for actual testing of the material, as most tests are designed for either “hard” rock or soil. The repercussions for soft rock testing are seen in Chapter 4, and include the following:

- Undisturbed sampling and testing techniques are designed for hard rock or soils, and it is difficult to collect undisturbed or intact samples of weak mud rocks.

- Coring is extremely difficult as the mudstone/siltstone fails on the bedding laminations, therefore preventing the retrieval of continuous core and samples large enough for testing. Also water is used to cool the drill bit and the mudstone/siltstone is not volumetrically stable in water, causing it to be washed away, swell, and/or become lodged in the drill core casing.

- The material is too stiff for insitu testing with a shear vane, as would be appropriate for engineering soils.
Laboratory tests are hard rock or engineering soil-specific. Triaxial testing of hard rock involves placing a specified length of core in a confining cell and applying normal pressure, and in this test the laminated mudstones/siltstones deform rather than fail. Whilst for soil triaxial testing the nature of the laminated structure of the mudstone/siltstone makes preparing an undisturbed sample by cutting it from a larger sample, preserving the laminated structure extremely difficult. It is likely that the sample would separate on a lamination and therefore no longer be undisturbed or intact.

2.4 Mine Footprint Structure

2.4.1 Pre-Mining Assumptions

The Engineering Geology Plan (Figure 2.2) shows the relatively small “window’ of bedrock exposed beneath the loess colluvium cover by the mining operations. Prior to development of the present opencast mine the structural data was inferred from past underground mining activity and very limited drill hole information, most of the latter being carried out well beyond the mine site. There is difficulty inferring any structural elements from the drill hole data due to the lensoidal nature of the coal resource and the absence of any retained core, but past underground mining in the area resulted in development along continuously dipping beds. The photographs in Figure 2.5A-D show stages in the “unroofing” of the coal measures from which the present structural “story” was developed.

During the initial development stage of the Malvern Hills Opencast Mine it was assumed that the bedding in the area dipped at ~45°, and that the strike and dip were both relatively consistent. The observed strike (055°) and dip (45° SE) were thought to correlate with the Nimmo Colleries plans from the 1980s of the same area, and so continue down dip without change. Trenching confirmed this orientation, yet investigations prior to the Malvern Hills Opencast pit excavation revealed vertical dipping beds at the portal of the Roly Mine immediately to the SE and down dip from the proposed pit (see location on Figure 2.2). This change in dip was assumed to be the result of slow near surface creep (Field, 2002) and was disregarded.
Figure 2.5  A. 21 April 2003 - First excavation for the pit was made immediately to the southeast of survey mark ITII and revealed coal (055/45SE).  B. 6 May 2003 - Old underground workings in Main Seam coal (055/45SE) were exposed in the pit at 373 RL.  C. 3 October 2003 - Looking to the northeast along strike at the pit developed to 360 RL.  Note the long dip slopes of the footwall.  D. 31 May 2004 - A tight fold is revealed in the southwest end of the highwall.

2.4.2 Interpretation Changes with Mine Development

Excavation of the Malvern Hills Opencast Mine began on 21 April 2003 and exposed a coal seam at 055/45SE (Figure 2.5A).  Further pit development revealed previous underground workings at 373 RL that were not on the plan of the Roly mine from Nimmo Colleries (Figure 2.5B), nor were these workings in the 45° up dip projection of the Roly mine coal seam.  To explain this situation, a survey error of Nimmo Colleries mine plan map-grid reference was assumed, although it was also known that easily won coal was not always recorded.  The structural implications of the offset in the up-dip projection of the Main Seam coal were not understood at this time (April-May 2003).
By 3 October 2003 the pit had been excavated to 360 RL and 2,500 tonnes of coal and 30,000 m$^3$ of soft rock overburden removed. The pit extended for 80m along strike and to a depth of ~30m as part of initial development (Figure 2.5C). Cross cutting faults were noted in the footwall and were the side release points of several small slab failures. Operations resumed in early May 2004, with 80,000m$^3$ of overburden removed to waste by 9 July. During the campaign the highwall was cut back in order to have access to the Engine Seam and Main Seam coal down dip, and on 31 May 2004 a tight anticlinal fold of what is interpreted to be the upper part of the Engine Seam was exposed (Figure 2.5D).

As the highwall was cut back and the pit floor lowered, a syncline was exposed in the pit floor on 1 June 2004 and a trench oriented perpendicular to the strike of the bedding was dug from the top of the Engine Seam to the highwall where the Engine Seam repeated (Figure 2.6). In the trench the normal sequence above the Engine Seam #1 was seen to the southeast (green sand, Engine Seam #2, laminated mudstone/siltstone as in Figure 2.3, yet the strikes and dips measured for the units above the green sand changed in dip direction from southeast to northwest. The strikes and dips taken in this trench located the axis of the syncline in the green sand unit and a shear zone adjacent to the highwall where the repetition of Engine Seam #1 dipping to the southeast was uncovered.

The photo in Figure 2.7 taken on 7 June 2004 shows the exposed highwall sequence, with the swing in strike, change in dip, and several faults annotated. The southwestern end of the highwall where the anticlinal fold was first exposed showed southeast dipping beds and Engine Seam #1 coal. The northeastern end shows the units above the Engine Seam #1 dipping north and inferred to be the northern limb of the anticline. Two cross cutting faults were identified in the highwall. The information from the pit floor trench and highwall was used to complete the three cross sections in Section 2.4.3.
Figure 2.6 Trench dug on 7 June 2004 to expose the syncline on the pit floor and fold related faulting at the base of the highwall.
Figure 2.7 Photo of highwall taken on 7 June 2004 showing measured strikes and dips of bedding along with two partial fault traces.
2.4.3 Interpreted Mine Structure

In light of the information collected from photos, personal observation and measurements described in Section 2.4.2 the pit subsurface structure was interpreted. Figures 2.8 to 2.10 provide three cross sections from within the mine footprint showing the progressive structural geology changes from southwest to northeast along the axis of the pit floor. These three sections, labelled SW, Mid, and NE, indicate the respective locations as indicated on the Engineering Geology Sketch Plan in Figure 2.2, and show the pit geology as exposed immediately after the toe-buckle failure removal in early July 2004. The locations of the cross sections were selected to best illustrate the observed changes in the structure of the pit and the toe-buckle failure geometry. The structure of the fold hinges, limbs, and fault trace beyond the pit walls are extended from measurements taken and are inferred only.

The three pit cross sections are presented from the southwest end of the pit to the northeast. Each of the cross sections show the original ground profile with relevant coal seams (Engine #1, Main and four thin footwall seams) plotted in their stratigraphic position. The pit profile shown is as the pit was immediately following removal of the toe-buckle failure material. The pre-failure batter is shown as a blue dashed line and the toe-buckle slab profile after failure is shown as a black dashed line.

The southwestern cross section (Figure 2.8) shows the offset of the Engine #1 and Main Seam coal at the thrust fault with the Main Seam on the highwall side thrust upwards and meeting the Engine Seam on the footwall side of the fault. It is also important to note that the toe-buckle slab profile at this end of the pit is a true toe buckle with the slab translating down dip as a coherent flexible unit, buckling at the toe.

The mid pit cross section (Figure 2.9) indicates approximately the same offset along the fault as the southwestern end of the pit, but here the syncline is more shallow with the hinge appearing to be closer to the fault. The toe-buckle slab has partially slid behind the toe of the failure.
Figure 2.8 Cross section at southwestern end of the pit (see Figure 2.2 for location). Note large offset along fault of the Engine Seam and Main Seam and the “perfect” nature of the toe-buckle slab failure.

Figure 2.9 Cross section at mid pit (see Figure 2.2 for location). The fault offset of the Engine Seam and Main Seam coal is approximately the same as the southwestern cross section. The toe-buckle failure slab has partially slid behind the buckled toe.
Figure 2.10  Cross section at the northeastern end of the pit (see Figure 2.2 for location). Here the fault has a negligible offset to the coal seams and the toe buckle slab has completely slid behind the buckled toe material.

The northeastern pit cross section (Figure 2.10) shows the fault offset to be minimal. Note location of anticline hinge close to the fault trace. The toe-buckle failure slab has completely bottomed out on the bench behind the buckled toe.

2.4.4 Engineering Geology Implications

The Engineering Geological Sketch Plan (Figure 2.2) and cross sectional views (Figures 2.8, 2.9 and 2.10) are further discussed in relation to the toe-buckle failure in Chapter 3. This section compares the three cross sections to establish the structure of the pit area including the folding and faulting relationships. The cross sections imply that:

- The anticlinal and synclinal folds plunge to the northeast.
- The fold axial planes are parallel.
• The fault trace caused by the folding offsets the Engine Seam and Main Seam completely at the southwestern end of the pit, but there is only a small offset at the northeastern end.

These three implications have an impact on slope stability in the pit, and future down dip development due to kinematically feasible failures in the highwall and the long dip slopes of the footwall. Also it is important to realise that the inferred coal resource must be re-evaluated, along with the economics of extracting the coal, because seam repetition may increase the resource yet the quality of the resource in the folded sections may decrease. The folding, and faults related to the folding have allowed bedding plane flexural slip in the footwall, and this implies low shear strength surfaces within the rock mass. The unpredictable presence of thrust faults sub parallel to the dip will also have implications for future extension of the pit along strike.

Several cross faults striking northwest further disrupt the highwall and coal seam stratigraphy, causing small nuisance failures of the highwall where the defects and bedding surfaces provide kinematically feasible wedge slab failures.

### 2.5 Pit Hydrogeology

Surface water from rainfall in the pit is directed from the pit floor to the settling pond, where sediment is allowed to settle before the water is released into the catchment. As noted in Chapter 1 the settling pond has also become a water treatment pond after the discovery that the runoff from the pit was acidic after high intensity rainfall especially in summer and autumn. The acidity is sourced from the dessication and then slaking of the laminated mudstones/siltstones of the footwall during the dry summer months providing loose material and greater surface area for the high intensity rainfall to circulate through.

Below the surface water may be stored in the cleat of the coal seams as the exposed coal seams in the pit have access to water via rainfall and runoff. Coal seams often act as an aquifer (Fetter, 2001). Coal seam storage of water may be the simple explanation for the marshy areas at both ends of the pit which correspond to the near surface outcrop of the coal seams.
Permeability of the units in the pit footprint and the affect of defects in the pit vary. The units in the mine footprint are listed below with their respective inferred hydraulic properties (order of magnitude) (Fetter, 2001):

- The **highwall** loess colluvium has very low permeability with \( K \) values of less than \( 10^{-6} \) m/s.

- The **highwall** sands probably have \( K \) values of \( 10^{-2} \) m/s and have the ability to be water bearing if there is access to water.

- The **pit floor** interburden is effectively impermeable with \( K \) values of greater than \( 10^{-10} \) m/s.

- The **pit floor** coal seams can store water in their cleat if recharge is available, and thus act as aquifers.

- The **footwall** laminated mudstones/siltstones act as an aquiclude with the thin coal seams (some broken and sheared) acting as aquifers. Batter development tends to follow thin coal seams in the footwall, and the batter drains thus effectively recharging the coal aquifer at each bench.

It is important to note that fracture permeability in mudstone/siltstone is expected to be low as the defects are not persistent, yet cross cutting faults act as aquicludes as they are filled with gouge and no longer appear to be zones of preferred permeability.

### 2.6 Highwall Stability and Later Pit Development

#### 2.6.1 Highwall Design and Construction

The highwall was designed at a batter angle of \( \sim 76^\circ \) (4V:1H), with 15m vertical separation of the benches (the same separation as the footwall) if required. This batter and bench configuration was considered acceptable as the bedding of the highwall was expected to dip into the face at approximately 45\(^\circ\), and therefore realistic as the material was sufficiently coherent to stand at such a steep angle. Minor wedge or slab
failures could be expected to be released on joints or other structures, and some slaking from desiccation would occur on the exposed face.

As the pit was deepened below 360 RL and the complex structure was discovered in late May 2004, it became apparent that the highwall could be compromised to some extent due to the rapid changes in dip of the bedding. Figure 2.11 shows the highwall on 18 Oct 2005, and an extensive failure at the north end of the pit. This slide, which has an estimated volume of 800 m$^3$, involves rotation on a carbonaceous bed resting on green sand above the syncline in Engine Seam #1. Movement continues to deform the pit floor above 350 RL and the rate of displacement is sensitive to rainfall with several cm observed in a 24 hour period before internal drainage resulted in stabilisation.

Figure 2.11  Photo of the highwall showing an extensive failure at the northeast end. This rotational slide has an estimated volume of 800 m$^3$. At the base of the slide is a carbonaceous unit resting on green sand above the syncline in Engine Seam #1. Movement continued to deform the pit floor above 350 RL and the rate of the displacement was sensitive to rainfall.
2.6.2 Future Pit Development

Figure 2.12 shows the pit profile changes in cross section at mid pit. The original ground, November 2003 survey, July 2004 after toe-buckle failure removal, April 2006 survey, and the proposed stripping for May-June 2006 pit profiles are colour coded in order to easily track the pit development.

As the pit deepens minor toppling block failure (volumes \( \bullet 1 m^3 \)) are expected to continue to occur from the highwall, yet are not considered a significant safety hazard. The present highwall will be removed and reformed when the pit floor is lowered to 340 RL. At the same time the footwall dip slopes will increase in length. There is the potential for further toe-buckle failures to occur on the long dip slopes with thin coal seams as failures surfaces. To prevent the amount of water in the footwall slope drainage measures were implemented in March 2006. Four drains were placed in four coal exploration drill holes at the base and along strike of the Main Seam coal. Each drain extends 15m into the footwall and drains the thin coal seams in the footwall.

![Figure 2.12 Cross section at mid pit showing sequential pit development and location of drill hole for drainage of thin coal seam “aquifers” in the footwall.](image)
2.7 Synthesis

This chapter provides the engineering geology of the pit known at the time of the toe-buckle failure. The area was mapped and an Engineering Geological Sketch Plan produced on a Connell Wagner base map survey. The mapped units were divided into three sections, the highwall, pit floor, and footwall and described using geological terminology in a stratigraphic column. The units were then described in engineering geological terms to represent their strength and water content. All of the material exposed in the pit is “soft” rock (weak to moderately weak in engineering geological terms) and therefore assumptions are made as to characterise them as soil or rock along with the implications for testing.

The structure of the units in the pit is inferred from observation, photographs and direct measurements and three cross sections produced to reflect the 3D structure. Early in pit development the beds were thought to be dipping at ~45° yet as the pit developed down dip folding and related faulting were revealed which has an effect on the future development of the pit and the quality of the resource.

The pit hydrogeology plays an important role in the footwall stability. Water is stored in the coal cleat and holes have been drilled into the footwall and piped to drain the stored cleat water. The fact that the laminated mudstones/siltstones are of low permeability means that water cannot be drained from the coal naturally.

The highwall cut at 4V:1H will have nuisance toppling failures and the possibility of large rotational slides. But these can be mitigated with safe practices.

Future pit development will reduce the height of the highwall but lengthen the dip slope of the footwall. Therefore, the potential for future large toe-buckle failures exists. The toe-buckle engineering geological model is developed in Chapter 3.
3 Toe-Buckle Slab Engineering Geology

3.1 Introduction

In Chapter 2 the mine footprint engineering geology was presented, and Chapter 3 focuses in on the footwall stratigraphy and the role it played in the stability and configuration of the large slab, toe-buckle failure on 1 July 2004. The toe-buckle failure is described in detail, including the failure event, the pre-failure pit development, and the geometry of the failure. A closer examination of the 355 RL bench surface has been carried out to create an engineering geological model by logging, describing the material involved in the slab and failure surface of the toe-buckle failure, and collecting samples for testing. The test methods and results of the sampling are presented in Chapter 4.

The specific aims of the present chapter are:

- To characterise the footwall stratigraphy, materials, and structure in relation to the pit footwall.
- To describe the toe-buckle failure in detail, including the failure event, pre-failure pit development, and geometry.
- To provide a detailed log of the slab and failure zone.
- To reference locations of collected samples for parameter testing that are described in Chapter 4.
- To create an engineering geological model of the footwall, failed slab, and failure zone for subsequent analysis.

The chapter format is:

Section 3.2 details the footwall stratigraphy and the relationships of the batter and bench configuration of the footwall to thin coal seams.
Section 3.3 describes the toe-buckle failure of 1 July 2004 including the failure event, the pre-failure pit development and the failure geometry using three cross sections perpendicular to the failure plane.

Section 3.4 develops an engineering geological model of the toe-buckle slab and its footwall and hanging wall based on a trench through the 355 RL bench which was logged along with the engineering geological descriptions of the materials involved.

Section 3.5 provides an engineering geological model as a basis for further down dip pit development.

Section 3.6 is a synthesis of the footwall slab/toe-buckle engineering geology.

### 3.2 Detailed Footwall Stratigraphy

The mine footprint stratigraphy was presented in Section 2.2.2 and Figure 2.3, but a closer examination of the footwall stratigraphy and materials is necessary to establish the controls and mechanical feasibility of the large toe-buckle failure and the potential for more of the same type of failure. Section 2.2.2 described the footwall as comprised of laminated mudstone/siltstones interbedded with thin carbonaceous layers and coal seams. The footwall section of Figure 2.3 is reproduced in Figure 3.1 as a conventional stratigraphic column, with the location of the failed slab and failure surface of the 1 July 2004 toe-buckle failure bracketed. Several thin coal seams are seen in the footwall, yet those that are shown on the footwall column in Figure 3.1 represent seams that have also played a role in the pit development.

While forming the 15m vertical height batters between the 5m wide benches to the design specifications, the on-site excavator operator observed that the footwall kept failing back to the most easily parted thin coal seam horizons and formed a batter surface parallel to bedding pers com Ching, (2005). As the locations of these thin coal seams were close to the design parameters, instead of forming the batter in the laminated mudstone/siltstone, the thin coal seams were used. This fact is easy to confirm as each batter slope coincides with a thin coal seam and each batter drain on
the footwall side of the batter is cut into a thin seam that formed the batter above (Figures 3.1 and 3.2).

Figure 3.1 A stratigraphic column showing the sequence in the pit footwall immediately below the Main Seam coal. The relationship of four thin coal seams to the batter and bench configuration are shown along with the stratigraphic location of the toe-buckle slab and failure surface.
Figure 3.2  Cross section at mid pit showing the relationship of the four thin coal seams in the footwall to the batter and bench configuration. Note that the bench drains are at the foot of each batter, and therefore cut into a thin coal seam. Refer to Figure 3.1 for the footwall stratigraphic column.

The stratigraphic column in Figure 3.1 shows the location of the four thin coal seams in the footwall laminated mudstone/siltstone sequence and their relationship to the pit benches and batters is noted. Figure 3.2 illustrates the 45º dip of the units and the footwall batter and bench configuration in cross section in the middle of the pit. From this cross section it is easy to see the relationship of the batters to the presence of thin coal seams especially those involved in the large toe-buckle failure. For instance:

- The first coal seam below the Main Seam correlates to the batter surface formed below the 370 RL bench before the failure (see Figures 3.1 and 3.2 Note a).

- The thin seam ~2m stratigraphically below is recognised as the large toe-buckle failure surface (see Figures 3.1 and 3.2 Note b).
• The next thin coal seam down in the sequence corresponds to the batter between the 370 RL and 385 RL benches (see Figures 3.1 and 3.2 Note c).

• The lowest thin coal seam shown correlates to the batter above the 385 RL bench (see Figures 3.1 and 3.2 Note d).

• It is important to also note that the bench drain at the foot of each batter is placed in a coal seam. Unfortunately this configuration leads to easy recharge of the thin coal seams with water, as mentioned in Section 2.5.2.

3.3 Toe-Buckle Failure of 01 July 2004

3.3.1 Failure Event

The toe-buckle failure of 1 July 2004 occurred overnight on 30 June 2004, immediately following the completion of the excavation of the batter below the 370 RL bench. No one was present at the time and no machinery was affected by the collapse of some 3,700m$^3$ of footwall materials. The failure extended over 85m along the length of the 355 RL bench, as shown in Figure 3.3. The ~2m thick slab displaced 6.2m down dip and was accompanied by the development of a broadly cylindrical fold structure at the toe of the slab on the newly formed 355 RL bench. Complete removal of the failed slab debris from the 355 RL bench was easily accomplished in one day by a digger placed on the bench excavating the toe, with the loose slab sliding down dip on the basal failure surface.

The three photos in Figure 3.3 were taken on the morning of 1 July 2004 just prior to the digger removing the toe-buckle failure. Unfortunately I did not see the toe-buckle failure, only 20 “early morning into the sun” photos record the failure, along with two pages of notes in David Bell’s field notebook pers com Bell, (2004) (See Appendix A).

The panoramic series of photos in Figure 3.3 shows the entire footwall along strike and the portion that failed as a toe-buckle failure with pertinent dimensions. The photo in the bottom left corner of Figure 3.3 shows the shiny and still very wet failure
Figure 3.3  Composite photo taken on 1 July 2004 of the footwall showing the toe-buckle slab failure before the failure was removed. Dimensions of the failure are shown along with a description of the toe-buckle structural changes from a true buckle at the southwestern end to the slab driving behind the buckle at the northeastern end. Note the wet failure surface under the slab in the bottom left photo. The bottom right photo is taken looking down from the 370 RL bench and shows the changes in the buckle along the footwall.
surface on the morning of 1 July 2004, and the part of the 370 RL bench that slid down dip 6.2m. The bottom right photo is taken from the part of the 370 RL bench that remains, and looks down on the slab and toe-buckle. It is important to note that the toe-buckle fold structure varies from SW, where the slab is coherent and buckled at the toe, to the NE where the slab has driven in behind the buckling at the toe. This structure is further examined and reconstructed in Section 3.3.3.

3.3.2 Pre-Failure Pit Development

Figure 3.4 is a series of six photos showing the excavation sequence to deepen the pit in order to access more down dip coal during June 2004. Figure 3.4A is a photo of the pit looking to the northeast along strike on 11 June 2004. The 385 RL bench has been narrowed and a drain cut into the thin coal seam against the batter (Figure 3.4B). Figure 3.4C (13 June 2004) shows the digger removing overburden that was pushed onto the 370 RL bench from the narrowing of the 385 RL bench and cutting back of the batter between the two. The batter below the 370 RL bench is formed on a smooth planar surface that is dry, orange stained, and carbonaceous (Figures 3.4D and E). This surface is the pre-failure batter between the 355 RL and 370 RL benches, and was formed on the dry, orange stained, and carbonaceous surface below a coal seam as the rock mass naturally broke out along this surface. Figure 3.4F (26 June 2004) shows the excavator completing the formation of the batter below the 370 RL bench down to 355 RL.

Key geotechnical issues leading up to the toe-buckle slab failure were:

- Batter formation controlled by location of thin coal seams in footwall sequence.
- Dry nature of face and orange staining on batter below the 370 bench.
- Loading of batter between the 370 RL and 355 RL benches.
- Final rapid toe excavation of batter above 355 RL bench on the afternoon of 30 June 2004.
Figure 3.4  A) 11 June 2004 - The pit looking along strike to the northeast. Note the highest (385 RL) bench has been narrowed. B) 11 June 2004 - The narrowed 385 RL bench with bench drain cut into the thin coal seam at base of batter. C) 13 June 2004 - The digger removing overburden that was pushed onto the 370 RL bench from the narrowing of the 385 RL bench and cutting back of the batter. D) 13 June 2004 - The batter below the 370 RL bench (tape is sitting on bench) is formed on a smooth planar surface that is dry, orange stained, and carbonaceous. E) 13 June 2004 - Close up view of batter and orange staining on bedding plane. F) 26 June 2004 - The excavator completing the formation of the batter below the 370 RL bench.
3.3.3 Toe-Buckle Failure Geometry and Reconstruction

Removal of the toe-buckle failure debris was easily accomplished in one day and the slab parameters were measured on 1 July 2004 as the debris material was being removed. The following is a list of the measured parameters and assumed parameters that were estimated from photographs and on-site observations.

Measured slab parameters:

- Length = 84.9m
- Apparent Slab Thickness = 2.8m at SW end, 3.10 at NE end
- Slab True Thickness = 1.94m at SW end, 1.99m at NE end
- Batter Angle = 44º at SW end, 42º at MID, 40º at NE end
- Dip Slope Length = 22.5m
- Bench Vertical Separation = 15m
- Calculated Slab Volume = ~3744m³
- Down dip translation of slab = 6.20m

Assumed parameters:

- Down dip length of slab above toe-buckle = ~10m (estimated)
- Down dip length of toe buckle area = after failure ~5m (estimated) and before failure = 12.5m

As listed above, the plane of the failure surface varies from 44º at the SW end to 40º at the NE end, and therefore affects the apparent thickness of the slab, yet the true slab thickness using geometrical relationships varies by less than 0.05m which is assumed to be measurement error. From interpretation of the above observations Figure 3.5 shows three sections through the buckled slab at the positions indicated in Figure 3.3.
• SW – At this end of the failure the dip is the steepest (44°) and the apparent thickness of the slab the thinnest (2.8m). The toe of the slab at this end of the failure has buckled and the outer surface of the slab has fractured to accommodate the outside curve of the buckle. However, it continues to appear as a smooth surface with the slab showing no sharp break in slope.

• MID – The middle of the slab is dipping at 42° to the southeast. The toe of the slab at this point has buckled and the slab has slid partially behind the toe. The outer surface of the toe has fractured to accommodate the outside curve of the buckle, but does not continue as a smooth surface and the slab drives behind the toe debris to some extent.

• NE – This end of the failure has the shallowest dip (40°) and the greatest apparent slab thickness (3.10m). The toe of the slab has buckled and the slab has slid to the bench below the toe. The outer surface of the slab has fractured to accommodate the outside curve of the buckle, but is more fractured than at the middle of the failure, and it is apparent that the slab has driven behind the toe debris to the 355 RL bench surface.

Figure 3.5 Three schematic cross sections of the toe-buckle slab showing the changes in characteristics of the buckle along the length of the footwall.
Changes in character of the toe cylindrical fold from SW to NE possibly reflect local buttressing, bedding attitude changes, and/or side release mechanisms. My interpretations are:

**Local buttressing** – The failure occurred immediately following the rapid excavation of the toe (355 RL bench formation). A portion of the 355 RL bench was not completely formed and a “mound” of material can be seen in Figure 3.3 at the MID cross section line. At this point the buckle fold at the toe changes in character, possibly as a result of the influence of the mound on the slab toe.

**Bedding attitude changes** – The change in dip along the strike of the failure surface affects the apparent slab thickness, together with the driving forces and friction angle that needs to be overcome. These parameters affect the changes in character of the toe-buckle fold across the slab.

**Side release mechanisms** – The side release mechanisms may also contribute to the character of the toe-buckle fold. Both ends of the failure terminate at a joint perpendicular to bedding that intersects the 370 RL bench. Jointing perpendicular to bedding occurs frequently in the pit, with several joints separated by up to 3cm of orange gouge but persistence is unknown. The pinning effects of the SW and NE side release joints in my opinion vary, and affected the rapid movement of the slab down dip and the toe-buckle formation.

- The SW end of the slab is closer to the natural surface of the slope and thus in closer contact with the colluvium and soil. The slab has parted at this juncture and has parted again approximately 1.5m along strike on a joint perpendicular to bedding. This would result in the slab moving unhindered down dip, and therefore resulted in a smooth slab toe-buckle.

- The NE end of the slab has parted on a joint perpendicular to bedding, but has deviated from the joint (perhaps due to lack of persistence of the joint) and released diagonally to the NE and down dip. This may have resulted in pinning of the slab, not allowing it to translate down dip smoothly, and
therefore affecting the formation of the toe-buckle, allowing the slab to break up at the toe and drive behind the toe debris.

Figure 3.6 shows close-up views of the side release areas of the slab, and provides before and after photos of the 370 RL bench. These photos show the basis for the observations and assumptions made for the side release mechanisms of the slab.

The SW side releases of the slab failure are shown in photos Figure 3.6A, B, and C. Photo A taken on 1 July 2004 is a frontal view of the two slab release joints at the SW end, with one joint following the near surface contour of the natural ground and the release joint further in is a defect of enough persistence to allow easy release of the translating slab. Photo B is a view looking along the 370 RL bench, where it is possible to see the slight offset of the two joints, with the one further to the SW being slightly pinned. Photo C shows the same view as B but after the slab was removed. Here it is possible to see evidence of down dip movement of the slab and folding beyond the end joint which is assumed to have occurred prior to the release of the side joint and subsequent toe-buckle failure.

The NE side release of the slab is shown in Figure 3.6 D and E. Photo D was taken on 1 July 2004 and shows the side release occurring in a non-persistent joint translating into a diagonal down dip side release fracture. Photo E taken on 6 July 2004 shows the side view along the 370 RL bench of the NE slab release, and a pinned part of the slab remaining suspended high on the batter, still attached to the slope. Photo F compares the “before failure” 370 RL bench to the same view of the “after failure” bench in Photo E. In Photo F a “line” is visible ~2.0m out from the up dip batter and ~3m in from the edge of the bench, and is the future failure surface of the toe-buckle failure.
Figure 3.6  A) 1 July 2004 - Slab release at SW end of failure showing two joints, end release follows the near surface contour of the natural ground and the release joint further in is along a joint of enough persistence to allow easy release of translating slab. B) 1 July 2004 - Side on view from 370 RL bench of SW release joints and translated slab. Note the slight offset of the slabs two joints. C) 6 July 2004 - Same view as B but after slab debris removal. Note there is evidence of down dip movement and buckling of SW end beyond the failure. D) 1 July 2004 - Slab release at NE end of failure showing non persistent joint and diagonal down dip release of slab. E) 6 July 2004 - The 370 RL bench after the toe-buckle failure was removed. This photo shows the remaining 370 RL bench. Note the batter below the bench corresponds with the “line” in figure A. F) 13 June 2004 - The 370 RL bench before the toe-buckle failure. The batter above the 370 RL bench has been cut back to a thin coal seam with toe drain cut into same. This photo shows the 370 RL bench with visible “line” showing ~2.0m out from batter.
3.3.4 Contributing Factors

In order to back analyse the toe-buckle failure it is important to include two other contributing factors, rainfall history and earthquake activity, together with the geotechnical issues listed and observed structural relationships in Section 3.3.2 and Section 3.3.3.

Rainfall activity prior to the failure would add to the groundwater build-up because the 370 RL bench had its inside drain formed in a thin sheared coal seam of high permeability. Combined with the lack of drainage for the footwall thin coal seams, and rapid unloading during batter and bench formation, this would decrease the stability of the slope. Rainfall history prior to the failure is shown in Figure 3.7. The left hand chart shows that June 2004 had the lowest rainfall accumulations of all the months in 2004. Daily June rainfalls never reached more than 8mm, as shown in the right hand chart, and there was no measurable rainfall from 26 June to the time of the toe-buckle failure. Therefore rainfall may not be a direct contributing factor, yet it is important to note that water is retained in the permeable units and stored in the cleat of coal seams and broken coal confined by clayey layers of laminated mudstone/siltstone.

![Figure 3.7](image)

Figure 3.7 The rainfall data for these charts was recorded for the Malvern Hills Opencast Mine by a local resident that lives near the entrance to the mine. The left hand chart shows that June 2004 was a very dry month during that 2004, with rainfall accumulations of 31.25mm. The right hand chart indicates the daily rainfall amounts with no accumulation from 26 June onwards.

An earthquake in the area during the rapid excavation of the toe during the day of 30 June 2004 may have caused enough ground acceleration to break the cohesion and
separate the toe of the slope from the failure surface, allowing the slab to follow down
dip. Two earthquakes occurred just prior to the large toe-buckle failure being
discovered on the morning of 1 July 2004 (Figure 3.8). On 30 June 2004 at 6:01AM
an earthquake of magnitude 3.1 at a depth of 9km was centred within the immediate
area of the Malvern Hills Opencast Mine. A later earthquake that probably post dates
the failure occurred at 7:21 AM on 1 July 2004 (3.7) ~70km to the northwest
(GeoNet, 2006).

Figure 3.8  Map of central South Island showing the location of two earthquakes that occurred
prior to the 1 July 2004 toe-buckle failure at the Malvern Hills Opencast Mine. The 30 June 2004
earthquake in the immediate vicinity of the mine may have been a contributing factor to the
failure (GeoNet).

3.4 Engineering Geology of Toe-Buckle Slab

Three campaigns of sample collection and logging were undertaken in order to collect
samples for testing, to investigate the engineering geological controls and material
properties of the toe-buckle slab and failure surface, and most importantly to create a
model for analysis.

Firstly, holes were hand dug with spades to intercept the slab buckle failure surface on
the 355 RL bench at the base of the batter formed by the failure surface. The debris
fans of slaked material from the batter had to be removed and the failure surface identified. Two holes were dug, with the more southwest excavation exposing displaced blocks of the slab and free draining water in the coal layer. The northeastern hole intercepted the basal failure surface, and samples of a clayey layer with fine interbedded coal fines were cut from the batter for laboratory testing.

The second investigation involved the hire of a concrete corer and operator for the purpose of coring into the bench, through the slab material, and intercepting the failure surface and substrate. Unfortunately coring in laminated soft rock is difficult as the material is not volumetrically stable in water. The water-cooled corer introduced too much liquid, the cored material when wetted swelled and became stuck in the corer, or parted on the bedding surface when spun. Very little of the core drilled was useful for logging or testing, but with the information gained by the hand dug holes a correlation could be made and some samples for testing were retrieved.

The most successful investigation of the slab and failure surface was by trenching the 355 RL bench and cutting back into the batter. The location of this trench is shown on Figure 2.2. The operator manoeuvred his large 30 tonne digger to balance the bucket and dig a trench perpendicular to bedding through the 355 RL bench while the cab was sitting inclined on the 45º batter (Figure 3.9). The trench cut into the bench and cleared debris to expose the undisturbed material below the 355 RL bench surface, the toe-buckle slab material, the failure surface, and the footwall below the failure.

Figure 3.9  Photo of the 30 tonne digger cutting a trench perpendicular to strike on the 355 RL bench in order to expose the toe-buckle failure slab material.
Figure 3.10 is a composite model including a photo of the trench, an engineering geological cross section/log, engineering geological descriptions of the materials in the trench, and Schmidt hammer rebound numbers which are discussed in Chapter 4. The log shows a correlation of the thin coal layers with free water stored in the cleat and/or voids of broken coal (in the case of the failure surface) to the more weathered or clayey zones of laminated mudstone/siltstone. The fact that free draining water was found in the hand dug hole that intercepted the thin coal seam infers that coal seams act as aquifers, storing water in the cleat or in the voids of coal broken up by flexural slip during folding. The toe-buckle failure surface consists of a thin broken up coal seam sandwiched between oxidised clayey laminated mudstone/siltstone.

The material logged and sampled in the trench on the 355 RL bench can be divided into four units, as shown in the table in Figure 3.10. Stratigraphically from bottom to top these are: Footwall, Failure Surface, Toe-Buckle Slab, and Hanging Wall. (Also see Figure 3.1 for location of same in the footwall sequence.)

- The **footwall** is made up of dry to moist, compact, laminated mudstone/siltstone that grades into more weathered and softer, reddish mudstone/siltstone.

- The **failure surface** is divided into three units. The middle unit is a thin coal layer that in places is intact, and in others broken into granule size pieces. The coal layer is wet and free water was found in one of the hand dug holes. On both the top and bottom of the thin broken coal seam is weathered, clayey laminated mudstone/siltstone that is moist, soft to firm, and reddish in colour. The unit below the coal seam has interbedded coal fines in the weathered and clayey mudstone/siltstone.

- The **toe-buckle slab** is divided into four units. The bottom **Unit 1** is further subdivided into three sub-units having slightly different engineering geology properties. All are laminated mudstone/siltstones and white to light grey in colour. 1A and 1C are more compact and drier, with joints of low persistence (<300mm) visible perpendicular to bedding whereas 1B is not as compact, and
Figure 3.10 Trench log of toe-buckle slab materials including engineering geological soil descriptions of units, and Schmidt hammer rebound numbers.
wetter with no visible jointing. **Unit 2** is a carbonaceous and dry laminated mudstone/siltstone (A), grading into a coal layer which is dry (B), and then to a dry laminated mudstone/siltstone with orange staining between the layers which part easily when disturbed (C). **Unit 3** is a dry, not compact, lighter coloured laminated mudstone/siltstone that parts easily when disturbed and grades into **Unit 4**, which has orange staining on the bedding planes and joints, and parts very easily when disturbed. These orange stained layers corresponds to the original very planar and smooth batter formed between the 355 RL and 370 RL benches, and are shown at the 370 RL bench level in Figure 3.4 D & E.

- The **hanging wall** material corresponds with the batter formed below the 355 RL bench after failure. As seen throughout pit development this batter is formed on the upper part of a coal seam which is comprised of stony coal, and is interbedded with carbonaceous mudstone with orange staining. The material is dry yet stratigraphically below is a layer of moist, soft, orangish grey laminated mudstone/siltstone.

In summary Figure 3.10 shows the following:

- A basal failure surface in weak clay-rich mudstone/siltstone at the base of a 100mm thick thin sheared coal “aquifer”.

- A failed slab ~2m thick that has slid over the basal shear surface and at the same time incorporating broken coal fragments into the low-strength clay materials.

- The slab itself is not homogeneous but consisted of four distinct lithological units of mudstone/siltstone displaying variable weathering and a 0.1m thick coal seam.

- The upper surface of the failed slab had been formed on an orange pug in moderately weathered to highly weathered mudstone/siltstone.
• The existence of several thin coal seams in the footwall means that further slab-buckle failures are possible if the geometrical relationships (slab thickness:height ratio) are met, along with a failure surface and high pore water pressures.

3.5 Engineering Geology Model

Figure 3.11 is an engineering geological model of the mine as of April 2006, including the proposed deepening of the pit floor to 340 RL during the winter of 2006. It shows the following elements:

Footwall batters controlled by bedding on which small scale slide failures ($\leq 10^3$ m) can develop due to exfoliation and related slaking processes. More importantly, the presence of the thin sheared coal seams in the footwall act as aquifers and recharge from the bench drains, resulting in the potential for large scales ($>2000m^3$) toe-buckle slab failures.

The controls of the toe-buckle slab failure are both geometric (length to true thickness ratio) and geological (presence of thin sheared coal seams with a very low frictional strength). In the absence of batter drainage, significant pore pressures (heads of 20m) can develop at the base of the slab, resulting in failures once batter slopes exceed 20m.

The presence of a thrust fault in the centre of the pit floor not only controls resources recovery, but will pose a significant geotechnical concern once development progresses below 350 RL. Both toppling and shear failures are possible on the thin weak zone (~1.0m wide) that dips steeply (60-85º) to the southeast when coal recovery takes place in the southeastern part of the pit.

A steep (~4V:1H) highwall face developed across bedding, and with a general absence of penetrative rock mass defect. Minor wedge failures (typically $\leq 5m^3$ volume) are possible locally, and both toppling and shear failures were noted adjacent to cross fault. Apart from these defect controlled failure modes, only long term
Figure 3.11 Engineering geological model of the pit identifying the key issues for the highwall and footwall stability.
slaking and fretting of dried mudstone/siltstone is expected to occur from the highwall, forming an apron of debris at the base of the batter face.

The engineering geological model (Figure 3.11) identifies the key issues for footwall stability in respect to toe-buckle slab failures as follows:

- Batter slopes in the footwall are controlled by thin coal seams.
- Batter and bench configuration places batter drains in thin coal seams that recharge sheared coal “aquifers”.
- Buoyancy effects of pore water pressure in thin saturated coal seams influence stability of the (≤2m thick) slabs.
- There will be a critical relationship of slab length to slab thickness that defines the conditions for a toe-buckle slab failure.

3.6 Synthesis

Development of the Malvern Hills opencast coal mine required formation of footwall batters parallel to bedding at approximately 45°, with the highwall at about 4V:1H across the bedding surfaces. Nominal vertical separation of benches is 15m in the footwall, and the benches were constructed to a width of 5m to allow for future access and maintenance. The slab-buckle failure of 01 July 2004 resulted in the translation of a slab of mudstone/siltstone ~2m thick and 85m long (approximate volume 3,500 m³) down the recently completed batter face between the 370 RL and 355 RL benches in the footwall. Investigations of the failure described in this chapter have shown the following:

- The character of the cylindrical toe fold and slab movement changed slightly along strike as a consequence of minor changes in bedding attitude, and local pinning of the slab, but overall the failure mechanism was obvious.
• The slab was found to have translated ~6m down-dip on a clay-rich surface of low shear strength in weathered mudstone at the base of a thin (~0.1m thick) sheared coal seam which was saturated with water at the time of failure.

• Careful examination of the footwall batters has confirmed that they were formed by preferential excavation along similar thin coal seams, as the rock mass naturally failed along these weak flexural-slip bedding surfaces.

• The crushed coal within the thin footwall seams acted as aquifers that were recharged from the batter drains, and this resulted in elevated pore pressures acting on the base of the slab-buckle at the time of failure.

Detailed engineering geology logging and coring provided information on the nature of the buckled slab, with four distinct mudstone/siltstone units, and thin coal seam being identified beneath an upper surface developed in orange pug. This shows conclusively that the slab that failed was not in fact homogeneous, and suggests that its geometry (slab height to thickness ration) is a critical controlling factor.

The upper and lower surfaces of the failed slab continue below the 355 RL bench, and the potential clearly exists for further slab-buckle failures when the pit floor is lowered to 340 RL unless drainage measures are implemented. Testing of the materials involved in the 1 July 2004 failure are presented in Chapter 4 and analysis done in the remaining chapters to conform and mitigate the risk of future large toe-buckle failure with the potential to cause loss of life, damage to machinery, disruption of mining, and/or mine closure.
4 Material Characterisation and Geotechnical Testing

4.1 Introduction

This chapter presents the results of quantitative testing of the materials involved in the 1 July 2004 toe-buckle failure, including their physical properties and shear strength characteristics of the failure zone. This testing has been done with the intent of providing the parameters for the toe-buckle failure analysis presented in Chapter 5, where detailed discussion of the actual test results forms part of the sensitivity analysis.

According to Goodman (1980) toe-buckle failure analysis can be resolved using the Euler solution for column and beam bending. The formula provides an estimate of the critical slope length for failure to occur according to the following expression from (Goodman, 1980):

\[
 l_{\text{max}} = \frac{\pi^2 Et^2 \sin(90 + \phi)}{3L \gamma \sin(\delta - \phi)}
\]

Where:

- \( l_{\text{max}} \) = maximum length of slope above buckling section before failure occurs
- \( E \) = elastic modulus (MPa)
- \( t \) = thickness of slab (m)
- \( L \) = length of buckled section (m)
- \( \gamma \) = mass (MN/m\(^3\))
- \( \delta \) = angle of dip (°)
- \( \phi_j \) = interlayer friction angle (°)
Figure 4.1 Toe-buckled slope showing geometric parameters for the Euler solution after Goodman (1980).

The Euler solution is the classical method of toe-buckle analysis and uses the geometric parameters quantified in Chapter 3 and identified in Figure 4.1, as well as the geotechnical parameters elastic modulus (E), unit weight (γ), and interlayer friction angle (φj). This chapter outlines the sampling and testing program designed to provide the geotechnical input parameters for the Euler solution, with the following specific aims:

- To provide simple material characterisation of the materials involved in the toe-buckle failure.
- To quantify the geotechnical parameters for the Euler solution suggested by Goodman for analysing toe-buckle failures.

### 4.2 Sampling and Testing Program

#### 4.2.1 Required Geotechnical Parameters

A sampling and testing program to estimate the key geotechnical parameters must provide the following:

- **Elastic Modulus (E)**, which can be determined by stress-strain testing of intact material (direct testing $\rightarrow E_{stat}$) or by seismic velocity testing (indirect testing $\rightarrow E_{dyn}$).
Slab Natural Unit Weight ($\gamma$), or weight (solids plus water) per unit of total volume, irrespective of the degree of saturation (Brown, 1981), which requires measuring the dry density of disturbed laminated mudstone/siltstone core and back calculating the natural unit weight using the known (insitu) moisture content.

Interlayer Friction ($\phi_p$ and $\phi_r$); which requires an intact undisturbed sample of the failure surface and can be measured by direct shear (peak friction angle, $\phi_p$) and ring shear (residual friction angle, $\phi_r$) testing.

### 4.2.2 Field Sampling

Three campaigns of sample collection were carried out as follows:

- Two days of hand excavating holes at the base of the batter on the 355 RL bench to intersect the failure surface (18 and 21 October 2005) (Figure 4.2A).
- Hire of concrete corer and operator to get core samples by drilling through the insitu toe-buckle slab material on the 355 RL bench and intercept the 1 July 2004 toe-buckle failure surface (9 November 2005). See Figure 4.2A for location of drill holes.
- Trenching of the 355 RL bench perpendicular to the strike of the bedding to expose toe-buckle slab material, and cutting into the batter above the bench to intercept the failure surface and material stratigraphically below for sample collection and Schmidt hammer field testing (15 and 26 November 2005). See Figure 2.2 for the location of the trench.

The sampling was done more than one year after the large toe-buckle failure occurred, for reasons described in Section 1.6.1, and the following observations were made:

- An apron of carbonaceous debris is deposited at the base of the batter onto the 355 RL bench due to desiccation of the material exposed on the surface of the batter, and transport downslope by natural frittering and/or rainfall runoff (Figure 4.2A).
By comparison the batter above 355 RL bench is “smooth” and planar with no visible smaller failures, whereas the batter above the 370 RL bench has several small slab failures and is not as planar (Figure 4.2B).

![Figure 4.2](image)

**Figure 4.2** A) Photo showing the location of a hand dug hole and drill core sampling on the 355 RL bench in November 2005. Note the apron of debris at the base of the batter from desiccated frittering of the material on the batter above. B) Photo taken in November 2005 showing the “smooth” and planar batter of the old failure surface above the 355 RL bench in comparison to the batter above containing isolated failures. C) Hand dug hole exposing the three units of the failure surface. The reddish brown color of FSA and FSC can be easily seen on the lower and upper surfaces of the broken coal seam (FSB). D) Samples were cut into FSA for direct shear box testing. This photo shows where three of the samples were extracted and two more had been cut in preparation to being lifted out.

Care was taken during sampling to get in situ and undisturbed samples of the slab material, failure surface, and the material below the failure surface. The hand excavated holes to intercept the failure surface, and some of the toe-buckle slab material remains, were located at the base of the batter on the 355 RL bench.
underneath the apron debris material. Correlating the failure surface excavated in the hand dug hole to the batter surface above was done to confirm that the material in the hand excavated hole was the actual failure surface of the toe-buckle failure (Figure 4.2C). Samples for direct shear box testing were carefully cut in the material identified as the failure surface, and the samples kept as intact and as undisturbed as possible (Figure 4.2D).

Coring was unsatisfactory because the material is not volumetrically stable in water, and tends to swell when wetted. Core was drilled both vertically into the bench and perpendicular to the bedding in the base of the batter on the 355 RL bench, but oriented core for logging purposes was not possible as broken stumps of the core were left in the hole and/or stuck in the barrel. Core sample lengths were less than 0.3m in length, with most being ~50mm. The failure surface was not present in the retrieved core, even though that was what was attempted to sample (Figure 4.3), and the very limited budget available did not allow for additional drilling/techniques.

![Figure 4.3](image)

**Figure 4.3** Section of core showing the condition it came out of the barrel. Water had to be used to cool the barrel while drilling and the material would swell in the core and become lodged, break into pieces, or break off on a bedding plane and remain in the hole.

Trenching through the bench was the most successful method of slab sample collection, with easy correlation made between the failure surface found in the trench and the batter failure surface. Sampling from below the surface of the newly exposed
trench was easily achieved, as the materials could be loosened and extracted with hand tools. These samples were used for material characterisation, testing for water content, liquid limit, plastic limit, plasticity index, linear shrinkage, solid density, and particle size distribution.

The bench was logged and field testing of the relative strength of the materials forming the failed slab was done using the Schmidt hammer. The log and Schmidt hammer results are shown in graphically in Figure 3.10 and as a table in Appendix D.12.

Intact undisturbed sampling of the weak carbonaceous mudstones and sheared coal seam materials proved difficult for the following reasons:

- The moist laminated mudstone/siltstone was too compact and “tight” to cut with a blade or spade, yet when hit with a hammer the material “split” on a bedding/lamination surface.

- The drier laminated mudstone/siltstone flaked into >15cm plate shaped sheets along bedding planes, or was loose to compact and not able to hold its shape when attempts were made to prise and/or cut out a sample.

- Coring was unsatisfactory as cooling water altered the sample. The water content was increased, causing swelling of the cored sample, and the core bedding surfaces were disturbed when the sample became stuck and rotated in the core barrel. Only short core samples were retrieved and correlating them to where they fit into the sequence was not possible as part of the core was lost by water flushing it away, part from being stuck in the barrel, and part from breaking off in the hole.

4.2.3 Laboratory Testing

Samples collected in the field were wrapped in plastic wrap, bagged and labelled in an attempt to keep them at their natural (as of date of sampling) water content for later laboratory testing. The soft rock materials were described using engineering
geological soil descriptions, as this method was found to be more appropriate than the engineering rock descriptions. The soil description table used in Figure 3.10 is reproduced here for ease of reference (Table 4.1).

Table 4.1 Engineering geological soil description of the toe-buckle slab, failure surface, hanging wall, and footwall.

<table>
<thead>
<tr>
<th>Material</th>
<th>Engineering Geological Description for Soil Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hanging Wall</td>
<td></td>
</tr>
<tr>
<td>Stoney Coal Hwb</td>
<td>Slightly weathered to unweathered, dry, black COAL interbedded with “cemented” carbonaceous mudstone and orange staining on bedding surfaces.</td>
</tr>
<tr>
<td>Orange Pug Hwa</td>
<td>Moderately to highly weathered, moist, soft, orangish grey, finely laminated, MUDSTONE/SILTSTONE.</td>
</tr>
<tr>
<td>Hanging Wall Hwa</td>
<td></td>
</tr>
<tr>
<td>Orange Stained Mudstone/Siltstone 4</td>
<td>Moderately weathered, dry, compact to loose, white to light grey, finely laminated, MUDSTONE/SILTSTONE, easily parts on bedding planes.</td>
</tr>
<tr>
<td>Laminated Mudstone/Mudstone 3</td>
<td>Unweathered, dry, compact to loose, white to light grey, finely laminated, MUDSTONE/SILTSTONE.</td>
</tr>
<tr>
<td>Two-Buckle Slab</td>
<td></td>
</tr>
<tr>
<td>2C Orange Stained Laminated Mudstone/Siltstone 2C</td>
<td>Moderately weathered, dry, compact to loose, grey to dark grey, finely laminated, MUDSTONE/SILTSTONE with orange staining on bedding planes which easily part when disturbed.</td>
</tr>
<tr>
<td>2B Coal 2B</td>
<td>Unweathered, dry, black COAL interbedded with carbonaceous mudstone.</td>
</tr>
<tr>
<td>2A Carbonaceous Laminated Mudstone/Siltstone 2A</td>
<td>Unweathered, dry, compact, grey to dark grey grading to more carbonaceous, finely laminated MUDSTONE/SILTSTONE.</td>
</tr>
<tr>
<td>1C Laminated Mudstone/Siltstone 1C</td>
<td>Unweathered, moist to dry, compact, white to light grey, finely laminated, MUDSTONE/SILTSTONE with joints perpendicular to bedding (very low persistence &lt;300mm).</td>
</tr>
<tr>
<td>1B Laminated Mudstone/Siltstone 1B</td>
<td>Unweathered, moist to wet, compact to loose, white to light grey, finely laminated, MUDSTONE/SILTSTONE (no jointing).</td>
</tr>
<tr>
<td>1A Laminated Mudstone/Siltstone 1A</td>
<td>Slightly weathered to unweathered, moist to dry, compact, reddish white grading to white to light grey, finely laminated, MUDSTONE/SILTSTONE with joints perpendicular to bedding (very low persistence &lt;140mm).</td>
</tr>
<tr>
<td>Failure Surface</td>
<td></td>
</tr>
<tr>
<td>Red Brown Clay C</td>
<td>Moderately to highly weathered, moist, soft to firm, red to dark brown CLAY (sourced from laminated mudstone/siltstone).</td>
</tr>
<tr>
<td>Coal B</td>
<td>Unweathered, moist to wet, black, intact to broken (fine gravel to coarse sand sized), COAL.</td>
</tr>
<tr>
<td>Red Brown Clay A</td>
<td>Highly weathered, moist, soft to firm, red to dark brown CLAY interbedded with coal fines.</td>
</tr>
<tr>
<td>Footwall</td>
<td></td>
</tr>
<tr>
<td>Laminated Mudstone/Siltstone  Fwb</td>
<td>Slightly weathered, moist, soft to firm, reddish white to grey, finely laminated, MUDSTONE/SILTSTONE.</td>
</tr>
<tr>
<td>Laminated Mudstone/Siltstone Fwa</td>
<td>Unweathered, dry to moist, compact, white to grey, finely laminated, MUDSTONE/SILTSTONE.</td>
</tr>
</tbody>
</table>
Testing methods were limited as the slab materials resemble compacted soil more than hard rock. Most strength tests for rock require the material to be volumetrically stable in water and infer that strength is more than 10 MPa. Soil strength tests use remoulded homogeneous soils, which is not a possible or appropriate method for testing the parameters of the laminated mudstone/siltstone soft rock, and cutting an undisturbed sample of the material for soil strength testing is not possible as the material is too stiff to be cut and/or comes apart on the laminations.

The failure surface materials were tested using soil testing methods for peak and residual friction angle. Table 4.2 relates the material descriptions in Table 4.1 to the tests done for each unit, and the test methods and complete test results are detailed in the Appendices C, D, and E. The following sections use the unit labels (Hw, Hanging Wall; S, Toe-Buckle Slab; FS, Failure Surface; Fw, Footwall) given in Table 4.1, and the material characteristics are described from the footwall unit upwards to the hanging wall.

Table 4.2 Table showing the tests done for each unit of the hanging wall, toe-buckle slab, failure surface, and footwall. Refer to Table 4.1 for material descriptions of each unit.

<table>
<thead>
<tr>
<th>Testing Program of Material Sourced from 355 RL Bench</th>
<th>Schmidt Hammer</th>
<th>Water Content</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
<th>Linear Shrinkage</th>
<th>Solid Density</th>
<th>Particle Size Distribution</th>
<th>Direct Shear</th>
<th>Ring Shear</th>
<th>Triaxial Compression</th>
<th>Seismic Analyser</th>
<th>XRD</th>
<th>Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hanging Wall</td>
<td></td>
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<td></td>
<td></td>
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<tr>
<td>Hwb Stoney Coal</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Hwa Orange Pug</td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Toe-Buckle Slab</td>
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<td>2</td>
<td>Orange Stained Laminated Mudstone/Siltstone</td>
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<td>2</td>
<td>Laminated Siltstone/Mudstone</td>
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<td>3</td>
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<td>4</td>
<td>Orange Stained Laminated Mudstone/Siltstone</td>
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<td>1C</td>
<td>Coal</td>
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<td>Coal</td>
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<tr>
<td>1A</td>
<td>Carbonaceous Laminated Mudstone/Siltstone</td>
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<td>1D</td>
<td>Laminated Mudstone/Siltstone</td>
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<td>1A</td>
<td>Laminated Mudstone/Siltstone</td>
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<td>C</td>
<td>Red Bitten Clay</td>
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<tr>
<td>A</td>
<td>Red Bitten Clay</td>
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<td>Fwb Laminated Mudstone/Siltstone</td>
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<td>Fwa Laminated Mudstone/Siltstone</td>
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</tbody>
</table>

● Test done.
○ Test done with no conclusive results.

4.2.4 Geotechnical Data Limitations

As previously noted, there are three key geotechnical parameters (E, γ, and φ) required for Euler analysis of the toe-buckle failure. Assumptions of elasticity and homogeneity in the failed slab are inherent in this approach, and standard laboratory
tests are best applied to semi elastic hard rock. The present study has been complicated by the following:

- The slab materials are low strength soft rocks that swell and slake on exposure.

- There are eight sub units recognised within the slab (Table 4.1), and all except Slab 2B (a thin coal seam) are laminated mudstone/siltstone. Each of the seven mudstone/siltstone units have unique material characteristics and therefore assuming the slab to be homogeneous is not realistic.

- Sampling the weak materials of the slab is difficult to impossible due to repeated breakage on the laminations/bedding planes or disaggregation and swelling from water used while drilling the core.

- Small samples could be obtained to estimate the unit weight of the slab ($\gamma$), but the lack of satisfactory drill core meant that $E_{\text{stat}}$ and $E_{\text{dyn}}$ could not be determined.

- The ISRM standard method for uniaxial and triaxial testing to estimate $E_{\text{stat}}$ requires core length to diameter ratios of 2.5 to 3.0, and this was not possible (Brown, 1981).

- Another study on soft mudrocks by (Lea, 2006) taking place at the same time showed that there was no glue available in New Zealand to affix strain gauges to moist to wet core samples.

- When attempting to test for $E_{\text{dyn}}$ using the seismic analyser, platen contact with the sample could only be maintained by applying a small compressive load which caused the sample to fail.

In contrast, testing the failure surface materials was feasible because they behaved as soil materials and could be tested using NZ 4402:1986 standard methods.
4.3 Footwall Material Characterisation

Only limited material characterisation of the footwall material was considered necessary, as the units below the failure surface do not influence the toe-buckle failure. By definition from Table 4.3, taken from Figure 3.10, the footwall material represents the laminated mudstone/siltstone (Units Fwa & Fwb) stratigraphically below the failure surface (FS). Although both footwall units are identified as laminated mudstone/siltstone, the Fwb is altered and represents the gradational zone between the highly weathered FSA and unaltered laminated mudstone/siltstone footwall materials.

Table 4.3 Table showing the two Footwall units and their soil material descriptions. Note: The contact between these two units is gradational, rather than sharp.

<table>
<thead>
<tr>
<th>Footwall</th>
<th>Fwb</th>
<th>Laminated Mudstone/Siltstone</th>
<th>Slightly weathered, moist, soft to firm, reddish white to grey, finely laminated, MUDSTONE/SILTSTONE.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fwa</td>
<td>Laminated Mudstone/Siltstone</td>
<td>Unweathered, dry to moist, compact, white to grey, finely laminated, MUDSTONE/SILTSTONE.</td>
<td></td>
</tr>
</tbody>
</table>

In the field Schimdt hammer testing was done to determine the relative strength of the material in the trench. The more weathered/altered Fwb had a rebound number of 15 compared to the dryer and unaltered Fwa rebound number of 20, indicating lower strength and/or higher clay content suggested by the reddish colour of the Fwb unit in contact with the highly weathered FSA.

Samples of these two units were not taken for water content testing as it was not considered relevant at the time of trenching. In the trench, Fwa was compact, dry to moist, and very “tight” with no visible joints to assist in extracting a sample, yet Fwb was moist and soft to firm. The drill core containing Fwa was the most intact and least disturbed of all the samples collected, and was prepared for testing in the seismic analyser with the goal of computing the elastic modulus from the results, but testing was not conclusive as good contact with the platens could not be maintained and therefore no reproducible results were achieved. The formula (Farmer, 1983) and data for the one apparently satisfactory test follow.
\[ E_{\text{dyn}} = \rho V_s^2 \left( \frac{3V_p^2 - 4V_s^2}{V_p^2 - V_s^2} \right) \]

\[ V_p = (55-10.4) \mu s / 0.05377 m = 1206 \text{ m/s} \]

\[ V_s = (185-8) \mu s / 0.05377 m = 304 \text{ m/s} \]

\[ \rho = \text{bulk density} = \text{estimated 2250 kg/m}^3 \]

\[ E_{\text{dyn}} = 609 \text{ MPa} \]

\[ V_{\text{dyn}} = 1 - \frac{V_p^2}{2(V_p^2 - V_s^2)} \]

\[ V_{\text{dyn}} = 0.47 \]

The smaller pieces of intact core provided a known volume to calculate the dry bulk density (Appendix D.13), but were not valid for computing the natural moisture content or natural bulk density as water was used to cool the barrel whilst drilling.

In summary it is assumed that the difference in strength, characterised by the Schmidt hammer rebound number of these two units, is due to their different water and clay content. Fwa is the least altered and had the highest Schmidt hammer rebound number of all the units tested in the trench, and one piece of core from this unit was intact enough to use for testing \( E_{\text{dyn}} \) with the seismic analyser.
4.4 Failure Surface Material Characterisation

The physical and geometrical properties of the slab material and failure surface are crucial to the Euler solution, and emphasis for testing was to establish the quantitative input parameters needed. For the failure surface, the interlayer friction angle ($\phi_j$) was the sole variable needed for the Euler solution, and the material was tested using direct shear and ring shear tests to quantify both the peak friction angle ($\phi_p$) and the residual friction angle ($\phi_r$). An intensive characterisation of the failure surface material was also carried out including: water content, liquid limit, plastic limit, plasticity index, shrinkage, particle size distribution, and XRD analysis to identify the type of clay.

4.4.1 Failure Surface Sub-Units

The failure surface was logged from the two holes dug into the batter on the 355 RL bench, and from the logging of the excavated trench (Figure 3.10). Table 4.4 is an extract of the table in Figure 3.10 showing the stratigraphic relationships and material descriptions.

<table>
<thead>
<tr>
<th>Failure Surface</th>
<th>C</th>
<th>Red Brown Clay</th>
<th>Moderately to highly weathered, moist, soft to firm, red to dark brown CLAY (sourced from laminated mudstone/siltstone).</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Coal</td>
<td>Unweathered, moist to wet, black, intact to broken (fine gravel to coarse sand sized), COAL.</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Red Brown Clay</td>
<td>Highly weathered, moist, soft to firm, red to dark brown CLAY interbedded with coal fines.</td>
<td></td>
</tr>
</tbody>
</table>

Three units make up the failure surface. The middle unit (FSB) is a thin (~100mm) thick coal seam that contains broken coal (fine gravel to coarse sand in size) along with some larger areas of intact coal. In one of the hand dug holes there was free draining water visible in the coal seam, and it is reasonable to assume that this was also the case at the time of the toe-buckle failure. Water was stored in the cleat and voids of broken coal in the seam and is replenished often as the upper and lower laminated mudstone/siltstone units were weathered to a moist, soft to firm red to dark
brown clay. The lower unit (FSA) has visible interbedded coal fines, the result of interbed shearing during folding of the sequence as the sampled site was immediately below any toe-buckle influence. Figure 4.4 shows the three units of the failure surface exposed in the trench floor on a 250mm x 250 mm grid. The failure surface appears to be non planar in the photo but the scraping of the digger bucket pulled the “pliable” clay out of the trench floor and thus deformed the contacts between the coal, clay, and laminated mudstone/siltstones. In the field the Schmidt hammer did not rebound on the clay or coal units of the failure surface. It is interesting to note that rebound numbers registered and stayed fairly consistent when the laminated mudstone/siltstones transitioned to less weathered material in the Slab 1A unit.

Figure 4.4  Photo of the failure surface with a 250mm x 250mm grid for scale. Note FSA is pliable and has been pulled overtop a larger broken piece in the thin coal seam (FSB) when being excavated. The broken coal of the thin seam is visible along with the jointing in Slab 1A.
4.4.2 Failure Surface Soil Characterisation Tests

Unit FSA was tested extensively. Numerous water content tests (to establish natural water content) were conducted, as they were used to characterise the starting water content for soil direct shear box and ring shear testing. The average of 8 water contents for FSA was 31%, and the range 29 – 35% (Table 4.5).

Table 4.5 Table of water content testing for Failure Surface A.

<table>
<thead>
<tr>
<th>Strat Reference</th>
<th>Water Content (%)</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>FSA</td>
<td>31</td>
<td>Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>32</td>
<td>Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>31</td>
<td>Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>29</td>
<td>Before Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>28</td>
<td>After Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>29</td>
<td>Before Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>27</td>
<td>After Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>35</td>
<td>Before Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>38</td>
<td>After Soil Direct Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>29</td>
<td>Before Ring Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>40</td>
<td>After Soil Ring Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>29</td>
<td>Before Ring Shear</td>
</tr>
<tr>
<td>FSA</td>
<td>44</td>
<td>After Soil Ring Shear</td>
</tr>
</tbody>
</table>

Additional material characterisation testing carried out included liquid limit, plastic limit, plasticity index, activity, and linear shrinkage (Table 4.6).

Table 4.6 Summary table of material characterisation testing results for FSA.

<table>
<thead>
<tr>
<th>Natural Water Content (%)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
<th>Activity</th>
<th>Linear Shrinkage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>57</td>
<td>31</td>
<td>26</td>
<td>0.81</td>
<td>10</td>
</tr>
</tbody>
</table>

To establish the liquid limit (LL) of the red brown clay material the Casagrande cup was used. The material tested had 31% natural water content at the beginning of the
test, and the liquid limit was computed as 57 (See Appendices C.2 and D.2 for test methods and raw data).

The plastic limit (PL) results were 31, or exactly the same as the natural water content which suggests that the red brown clay of the failure surface is normally at the insitu plastic limit. The plasticity index (PI) is the LL-PL or 26. This type of material is classified as (CH) or a clay of high plasticity under the British Soil Classification System (Barnes, 2000).

The calculated activity of clay fraction (= PI/C where C is clay content), was 26/32 = 0.81, which is in the normal activity class (Barnes, 2000).

Linear shrinkage of 10% was measured for FSA and FSC (Appendices C.5 and D.5). This was achieved by preparing the sample to near the liquid limit, placing it in moulds, allowing the sample to air dry (slow drying to prevent cracking) and then completely drying the sample in the oven. The dried sample was measured and the percentage of shrinkage was calculated (Figure 4.5).

![Figure 4.5](image_url)  
*Figure 4.5 Photo of the linear shrinkage moulds showing dried, and 10% linear shrinkage of FSA. The sample was placed in the moulds at or near its liquid limit and allowed to air dry, then oven dry.*
4.4.3 Failure Surface Particle Size Distribution

Particle size distributions were achieved by wet sieving and pipette methods (See Appendices C.7, C.8 and D.7, D.8 for methods and raw data.) The semi log plot shown in Figure 4.6 shows two curves for FSA (red and pink), a blue curve for Slab 1A, and a green curve for Slab 1B results.

The distribution curves for FSA are very similar and an average of the two results yields 10% sand, 59% silt, and 31% clay (Table 4.7). X-ray diffraction of the +9Φ air dried fraction of both FSA samples of the pipette analysis showed the FSA sample to be 90% kaolinite and 10% illite, which is consistent with the normal activity reported in Section 4.4.2 (Appendix E).

<table>
<thead>
<tr>
<th>Sample</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FSA 7-1</td>
<td>9</td>
<td>62</td>
<td>29</td>
</tr>
<tr>
<td>FSA 7-2</td>
<td>11</td>
<td>56</td>
<td>33</td>
</tr>
<tr>
<td>Average</td>
<td>10</td>
<td>59</td>
<td>31</td>
</tr>
</tbody>
</table>
The FSA is comprised of laminated mudstone/siltstones that have weathered due to contact with the water stored in the coal seam to form a clayey silt with some fine to medium sand whose clay content is 31% compared to the slab material at 13% (refer Section 4.5.3).

### 4.4.4 Failure Surface Shear Strength

The interlayer friction angle ($\phi$) is the sole variable needed from the failure surface for the Euler solution, although this is not specified as being peak or residual. Both peak shear strength ($\tau_p$) and residual shear strength ($\tau_r$) of the material were therefore determined at different normal stresses in order to achieve the maximum and minimum friction angle values for the failure surface. Test methods and results are presented in Appendices C.9, C.10 and D.9, D.10.

![Figure 4.7](image)

Figure 4.7  A) An “undisturbed” cut sample of FSA was placed in the soil direct shear box.  B) Normal stress was applied and the pore pressures allowed to dissipate before the shear force was applied.  C) and D) Photos of a sample after shearing. Note the “smooth” distinct failure plane. The sample was at its plastic limit (31) and the water content of sample remained at 31% before and after the test.
The peak shear strength was tested using the soil direct shear box. Samples of FSA cut in the field (as undisturbed as possible and at natural water content of \( \sim 31\% / \text{plastic limit}; \) (see Figure 4.2D) were placed in the box (Figure 4.7A). Normal stress was applied and the sample was allowed to completely consolidate and pore water dissipate before shearing stress was applied (Figure 4.7B).

The calculated normal stress \( (\sigma_n) \) for \( \sim 2\text{m} \) of overburden (or failed slab material) is 50kPa if a density of 2500 kg/m\(^3\) is assumed. Since the beginning of mining the slab buckle surface has had a history of being unloaded during the November 2003 campaign, then reloaded when the 385 RL bench was narrowed and material pushed onto the batter in May 2004 to accommodate deepening of the pit, and then unloaded again when the new batter was formed above the 355 RL bench in June 2004 (see Figure 2.12). Direct shear testing was done on four samples in sets of three different normal stresses in order to simulate the range of normal stresses (1.5m to 8m of overburden) that the failure surface had experienced. Figure 4.7 C and D show the sample taken out of the box after shearing. The shear plane is easily seen in the sample, along with the slickensided failure surface (Figure 4.7D).

\[
\begin{align*}
y &= 0.8068x - 8.5184 \\
y &= 0.4184x + 2.6511 \\
y &= 0.5325x + 2.4118 \\
y &= 0.366x + 24.79 \\
y &= 0.478x + 5.2295 \\
R^2 &= 0.9138
\end{align*}
\]

Figure 4.8  Plot of three sets of direct shear testing data with trendline and formulae. The dashed line is the trendline for all the data giving a peak friction angle for FSA of 26\(^\circ\) and 5.2 kPa cohesion.
Each of the direct shear test results are plotted in Figure 4.8 with a trend line drawn for each set of tests and a combined trend line (dashed) showing both the friction angle ($\phi_p = 25.6^\circ$) and calculated cohesion ($c = 5.2$ kPa). The water content of the sample after testing was the same as before testing, at approximately 31% (Table 4.5).

The residual friction angle ($\phi_r$) was derived from two samples of FSA tested in the ring shear apparatus. Again different normal stresses, in sets of three were used to simulate the range of normal stresses (1.5m to 8m of overburden) that the failure surface had been subjected to, as for $\phi_p$. Sample preparation included submersing the apparatus with the sample in a water bath to prevent the sample drying out (Figure 4.9A). When normal stress was applied to the sample, instead of consolidating, it expanded and this was confirmed by water content testing, which showed an increase from 30% to 42%.

![Figure 4.9 A) Photo of sample placed in water bath of ring shear apparatus with normal force applied for consolidation. This sample swelled during consolidation, taking up water in the sample from the water bath. B) Photo of the bottom platen of the ring shear apparatus showing the shear plane within the sample.](image)

A shear plane was created in the material by rotating the bottom platen and then the shear stress applied and resistance plotted. After testing careful disassembly of the test cell was done to examine the sheared surface to confirm that shearing took place.
within the material, and not along the contact surface of the material to the platen (Figure 4.9B). The results are shown in Figure 4.10 for the two samples, and a combined trend line forced through zero (no cohesion for residual friction angle) shows a very low $\phi_r$ of $\sim 3^\circ$.

![Ring Shear Analysis Showing Residual Friction Angle of ~3° For Failure Surface A](image)

**Figure 4.10** Plot of results with trendline from two samples tested at three different normal stresses each. The dashed line is the trendline of all the data forced through zero (cohesion = 0) showing a residual friction angle of $\sim 3^\circ$.

### 4.4.5 Summary of Failure Surface Results

The sole variable needed from the failure surface for the Euler solution is the interlayer friction angle. The FSA, in which failure had been observed to occur, was tested using direct shear methods to give a peak friction angle ($\phi_p$) for the material of $25.56^\circ$ and cohesion ($c$) $5.2$ kPa. The ring shear method gave a residual friction angle ($\phi_r$) at $\sim 3^\circ$. It is important to note that the sample swelled, and that the water content during ring shear testing increased from 31% to 42%.

The FSA was found to be at a natural water content of $\sim 31\%$ at or near its plastic limit, with shrinkage from liquid limit ($57\%$) to dry of up to $10\%$. The particle size of
the FSA is 10% sand, 59% silt, 31% clay, and the clay fraction (+9 Φ or <2 μm) is 90% kaolinite with 10% illite. There is little doubt that the high clay content is due to shearing and subsequent alteration of the underlying mudstone/siltstone that is in contact with the thin coal seam “aquifer” (FSB), because FSA and FSC have the same clay fraction mineralogy as the Slab (Appendix E).

4.5 Slab Material Characterisation

The two parameters needed from the slab material for the Euler solution are the elastic modulus (E) and natural unit weight (γ). As mentioned the elastic modulus can be estimated by stress-strain testing of intact material by direct methods using strain gauges and uniaxial compression (E_{stat}), or by indirect methods using seismic velocity testing (E_{dyn}). The inherent problems of testing soft rocks using hard rock methods have been addressed in Section 4.2.4.

The slab natural unit weight (γ) or weight (solids plus water) per unit of total volume, irrespective of the degree of saturation, requires measuring the dry density of disturbed laminated mudstone/siltstone core and back calculating the natural unit weight using the natural moisture content. Characterisation of the material in the slab was done by determining the moisture content and particle size distribution. XRD analysis was used to identify the clay size fraction. The solid density of the soil particles was also quantified.

4.5.1 Failure Slab Sub-Units

The slab is divided into 4 main units in Figure 3.10. Table 4.8 is extracted from the table in Figure 3.10 showing the slab unit and sub-unit material descriptions.
Table 4.8 Table showing the four main Slab units and soil material descriptions for all the sub units.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>4L</td>
<td>Laminated Mudstone/Siltstone (light grey, finely laminated with orange staining on bedding planes and joints which easily part when disturbed.</td>
</tr>
<tr>
<td>3L</td>
<td>Laminated Siltstone/Mudstone (unweathered, dry, compact to loose, white to light grey, finely laminated, Mudstone/Siltstone, easily parts on bedding planes.</td>
</tr>
<tr>
<td>2C</td>
<td>Orange Stained Laminated Mudstone/Siltstone (moderately weathered, dry, compact to loose, grey to dark grey, finely laminated, Mudstone/Siltstone with orange staining on bedding planes which easily part when disturbed.</td>
</tr>
<tr>
<td>2B</td>
<td>Coal (unweathered, dry, black coal interbedded with carbonaceous mudstone.</td>
</tr>
<tr>
<td>2A</td>
<td>Carbonaceous Laminated Mudstone/Siltstone (unweathered, dry, compact, grey to dark grey grading to more carbonaceous, finely laminated Mudstone/Siltstone.</td>
</tr>
<tr>
<td>1C</td>
<td>Laminated Mudstone/Siltstone (unweathered, moist to dry, compact, white to light grey, finely laminated, Mudstone/Siltstone with joints perpendicular to bedding (very low persistence &lt;300mm).</td>
</tr>
<tr>
<td>1B</td>
<td>Laminated Mudstone/Siltstone (unweathered, moist to wet, compact to loose, white to light grey, finely laminated, Mudstone/Siltstone (no jointing).</td>
</tr>
<tr>
<td>1A</td>
<td>Laminated Mudstone/Siltstone (slightly weathered to unweathered, moist to dry, compact, reddish white grading to white to light grey, finely laminated, Mudstone/Siltstone with joints perpendicular to bedding (very low persistence &lt;140mm).</td>
</tr>
</tbody>
</table>

Unfortunately not all of the slab units were characterised. The units stratigraphically above the thin coal seam (Slab 2B) were dry and came apart on the laminations, and obtaining representative samples for material characterisation would have been difficult. There are also no Schmidt hammer readings for these units as the hammer would not rebound off the easily broken low strength laminations.

4.5.2 Slab Soil Characterisation Tests

As with the failure surface several water content tests were done on the slab. Unfortunately it was not possible to obtain a full suite of results, but the data are complete for the units closest to the failure surface. Slab 1A closest to the failure surface had at the time of sampling a natural water content of 17% (Table 4.9). This unit contained jointing perpendicular to bedding, as did Slab 1C (Figure 4.4) which has a similar water content. Slab 1B was distinctive for not only having a raised water content (24%), but also for its lack of jointing and cool moist feel when compared to the less compacted and lighter coloured laminated mudstone/siltstone. Slab 2A is
more carbonaceous yet similar in water content to the adjacent Slab 1C. Further up
the sequence the material is dryer, compact to loose, with orange staining on bedding
planes suggesting periodic saturation and drying out resulting in oxidation of iron
bearing minerals. Even the thin coal in Slab 2B is dry.

Table 4.9 Water content of slab units that were tested. The thin coal seam (Slab 2B) and the
units stratigraphically above were not tested.

<table>
<thead>
<tr>
<th>Strat Reference</th>
<th>w (%)</th>
<th>For Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>17</td>
<td>Before Rock Shear Box</td>
</tr>
<tr>
<td>1A</td>
<td>17</td>
<td>Schmidt Hammer</td>
</tr>
<tr>
<td>1B</td>
<td>24</td>
<td>Schmidt Hammer</td>
</tr>
<tr>
<td>1C</td>
<td>15</td>
<td>Schmidt Hammer</td>
</tr>
<tr>
<td>2A</td>
<td>16</td>
<td>Triaxial Testing</td>
</tr>
</tbody>
</table>

Soft rock mudstone/siltstones do disaggregate in water, but for liquid limit and plastic
limit testing the sample is only wetted and mixed, making it difficult to disaggregate
the particles completely and therefore liquid limit and plastic limit testing were not
carried out. The problem of incomplete particle disaggregation holds true for linear
shrinkage testing, yet in the field the laminated mudstone/siltstones were seen to dry
and shrink, exhibiting drying cracks perpendicular to bedding and allowing the
laminations to separate into playing card sized pieces (Figure 4.11). This
characteristic is seen in within Slab 4 and Slab 2C, and is also the type of
weathering/slaking that occurs on exposed batters resulting in debris depositing in an
apron at the batter base (Figure 4.2B).

Figure 4.11 Block of laminated mudstone/siltstone drying in the sun. Note drying cracks
perpendicular to bedding allowing the laminations to separate into playing card sized pieces.
4.5.3 Slab Particle Size Distribution

Particle size distributions were achieved by wet sieving and pipette methods, with test results and raw data given in Appendices C.7, C.8 and D.7, D.8. The semi log plot in Figure 4.6 shows a blue curve for Slab 1A, and a green curve for Slab 1B results. Slab 1A and 1B have similar distributions and are made up of ~27% sand, ~59% silt, and ~13% clay (Table 4.10).

Table 4.10  Percentage of sand, silt, and clay in Slab units 1A and 1B from wet sieve and pipette particle size analysis.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab 1A</td>
<td>27</td>
<td>59</td>
<td>14</td>
</tr>
<tr>
<td>Slab 1B</td>
<td>26</td>
<td>62</td>
<td>12</td>
</tr>
<tr>
<td>Average</td>
<td>26.5</td>
<td>60.5</td>
<td>13</td>
</tr>
</tbody>
</table>

X-ray diffraction of the +9Φ air dried fraction of the pipette analysis showed both the Slab 1A and 1B clay fraction sample to be 90% kaolinite and 10% illite (Appendix E). XRD was also done on two air dried samples taken from the bottom of a small piece of the slab that was still resting on the batter face on 6 July 2004. These samples were whole samples, not just one size fraction, and contain the full range of particle sizes. They are dominated by quartz (75-80%), with the remainder ~20% kaolinite and a trace of illite (Table 4.11).

Table 4.11  XRD analysis results for FSA, Slab 1A, and Slab 1B +9 Φ clay fraction. Also shown are samples taken from the bottom of a piece of the failed slab resting on the batter, which was a complete sample dominated by quartz yet still having a clay fraction composed of kaolinite and a trace of illite.

<table>
<thead>
<tr>
<th>Sample Reference</th>
<th>Sample Preparation</th>
<th>Percentage Quartz (%)</th>
<th>Percentage Kaolinite (%)</th>
<th>Percentage Illite (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FSA</td>
<td>Air dried +9 Φ from Pipette Analysis</td>
<td>90</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Slab 1A</td>
<td>Air dried +9 Φ from Pipette Analysis</td>
<td>90</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Slab 1B</td>
<td>Air dried +9 Φ from Pipette Analysis</td>
<td>90</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Slab 1A/FSC</td>
<td>Air dried sample of slab bottom collected from a small piece of the slab that was still resting on the batter face just below the 370 RL bench on 6 July 2004</td>
<td>80</td>
<td>15</td>
<td>5</td>
</tr>
<tr>
<td>Slab 1A/FSC</td>
<td>Air dried sample of slab bottom collected from a small piece of the slab that was still resting on the batter face just below the 370 RL bench on 6 July 2004</td>
<td>75</td>
<td>25</td>
<td>trace</td>
</tr>
</tbody>
</table>
4.5.4 Slab Unit Weight Determination

Slab natural unit weight ($\gamma$) or weight (solids plus water) per unit volume, irrespective of the degree of saturation, requires measuring the dry density of disturbed laminated mudstone/siltstone core and back calculating the natural unit weight using an assumed moisture content.

Before attempting the dry density measurements, the disaggregated samples prepared for particle size distribution were tested using the method described in Appendix C.6 for determining the solid density of soil particles ($Q_s$). Slab 1A is easily disaggregated and was used for this test. The results revealed a solid density of the particles in Slab 1A of 2.4 tonnes/m$^3$ or 2400 kg/m$^3$ (Appendix D.6), which reflects the presence of about 13% clay minerals (mostly kaolinite) in a quartz dominated material.

As a known volume is needed to calculate density and unit weight, the “best” or most intact/least disturbed core samples were used. Results from intact samples of core from Slab 1C, 2A and Fwa are shown in Table 4.12. The dry bulk density of the laminated mudstone/siltstone is estimated to be $\sim$1900 kg/m$^3$, and the dry unit weight $\sim$0.019 MN/m$^3$.

Table 4.12 Table showing raw data, dry bulk density, and unit weight for Slab 2A, Slab 1C and Fwb/Fwa.

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>Dried Volume (m$^3$)</th>
<th>Dried Mass (kg)</th>
<th>Dry Bulk Density (kg/m$^3$)</th>
<th>Dry Unit Weight (MN/m$^3$)</th>
<th>Estimated Water Content (%)</th>
<th>Bulk Density at est. w (kg/m$^3$)</th>
<th>Unit Weight at est. w (MN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab 2A</td>
<td>0.000355</td>
<td>0.6842</td>
<td>1926</td>
<td>0.019</td>
<td>16</td>
<td>2234</td>
<td>0.022</td>
</tr>
<tr>
<td>Slab 1C</td>
<td>0.000209</td>
<td>0.3841</td>
<td>1837</td>
<td>0.018</td>
<td>15</td>
<td>2113</td>
<td>0.021</td>
</tr>
<tr>
<td>Fwb/Fwa</td>
<td>0.000127</td>
<td>0.2486</td>
<td>1951</td>
<td>0.019</td>
<td>16</td>
<td>2264</td>
<td>0.022</td>
</tr>
</tbody>
</table>

The natural unit weight is needed for the Euler solution calculations and back calculated using the known natural water content and dry unit weight. The assumed natural unit weight of the slab is $\sim$0.022 MN/m$^3$, as shown in Table 4.12.
4.5.5 Schmidt Hammer Testing for Relative Strength of the Slab

Schmidt hammer rebound numbers were recorded along the length of the trench from the Fwa unit, through the failure surface, and into the slab until no hammer rebound occurred. Table 4.13 shows the raw data from the testing that was used to construct the bar graph in Figure 3.10.

Table 4.13  Schmidt hammer rebound raw data (using a L type Schmidt hammer), corrected data, and equivalent cube compressive strength (conversion sourced from the Schmidt hammer manual by Proceq) in 50 mm intervals from 250mm below the failure surface thin coal seam (FSB) to 1500mm above.

<table>
<thead>
<tr>
<th>mm from bottom of FSB coal seam</th>
<th>Lithology</th>
<th>Schmidt hammer rebound value (Rα)</th>
<th>Corrected (Rα) for Vertical Impact Direction</th>
<th>Equivalent cube compressive strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-250</td>
<td>Fwa</td>
<td>20</td>
<td>23.2</td>
<td>25</td>
</tr>
<tr>
<td>-200</td>
<td>Fwa</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>-150</td>
<td>Fwb</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>-100</td>
<td>FSA</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-50</td>
<td>FSA</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>coal/FSB</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>coal/FSB</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>FSC</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>Slab 1A</td>
<td>12</td>
<td>15.2</td>
<td>10</td>
</tr>
<tr>
<td>200</td>
<td>Slab 1A</td>
<td>13</td>
<td>16.2</td>
<td>12</td>
</tr>
<tr>
<td>250</td>
<td>Slab 1A</td>
<td>13</td>
<td>16.2</td>
<td>12</td>
</tr>
<tr>
<td>300</td>
<td>Slab 1A</td>
<td>10</td>
<td>13.2</td>
<td>not in plot area</td>
</tr>
<tr>
<td>350</td>
<td>Slab 1A</td>
<td>10</td>
<td>13.2</td>
<td>not in plot area</td>
</tr>
<tr>
<td>400</td>
<td>Slab 1B</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>450</td>
<td>Slab 1B</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>500</td>
<td>Slab 1B</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>550</td>
<td>Slab 1B</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>Slab 1B</td>
<td>9</td>
<td>12.2</td>
<td>not in plot area</td>
</tr>
<tr>
<td>650</td>
<td>Slab 1B</td>
<td>9</td>
<td>12.2</td>
<td>not in plot area</td>
</tr>
<tr>
<td>700</td>
<td>Slab 1B</td>
<td>11</td>
<td>14.2</td>
<td>not in plot area</td>
</tr>
<tr>
<td>750</td>
<td>Slab 1B</td>
<td>11</td>
<td>14.2</td>
<td>not in plot area</td>
</tr>
<tr>
<td>800</td>
<td>Slab 1C</td>
<td>14</td>
<td>17.2</td>
<td>14</td>
</tr>
<tr>
<td>850</td>
<td>Slab 1C</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>900</td>
<td>Slab 1C</td>
<td>14</td>
<td>17.2</td>
<td>14</td>
</tr>
<tr>
<td>950</td>
<td>Slab 1C</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>1000</td>
<td>Slab 1C</td>
<td>14</td>
<td>17.2</td>
<td>14</td>
</tr>
<tr>
<td>1050</td>
<td>Slab 1C</td>
<td>17</td>
<td>20.2</td>
<td>18</td>
</tr>
<tr>
<td>1100</td>
<td>Slab 1C</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>1150</td>
<td>Slab 2A</td>
<td>18</td>
<td>21.2</td>
<td>20</td>
</tr>
<tr>
<td>1200</td>
<td>Slab 2A</td>
<td>16</td>
<td>19.2</td>
<td>17</td>
</tr>
<tr>
<td>1250</td>
<td>Slab 2A</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>1300</td>
<td>Slab 2A</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>1350</td>
<td>Slab 2A</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>1400</td>
<td>Slab 2A</td>
<td>15</td>
<td>18.2</td>
<td>16</td>
</tr>
<tr>
<td>1500</td>
<td>Coal Slab 2B</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Data collected on 26 Nov 05 in Bench Trench
Table 4.13 shows that Fwa is the strongest unit (rebound value of 20), rebound numbers ranging from no rebound in the Slab 2B coal and Slab 1B loose laminated mudstone/siltstone to 18 at the less carbonaceous bottom of Slab 2A.

The rock shear box was used in an attempt to test the shear strength of the laminated mudstone/siltstone soft rock material. Appendix C.11 describes the method used and Appendix D.11 contains the raw data. Two tests were prepared and carried out. The sample was encased in plastic wrap to prevent added moisture from the casting compound entering the sample. One sample was tested with a normal force of 3kN the other with 5kN. It was found that the material was too “soft” for testing in the rock shear box. The gauges were not of a suitable range and the forces used were too low to be moderated by the rams. The material collapsed under the normal force and shearing appears to be more ductile than elastic upon inspection of the tested samples (Figure 4.12).

Figure 4.12  A) Samples cast and ready to be tested in the rock shear box. B) Samples after testing. Note the bottom of each sample is in the foreground and the top in the background. The samples appeared to have collapsed and dragged, showing ductile rather than elastic behaviour.

4.5.6 Summary

The Euler solution requires two slab geotechnical parameters; the natural unit weight ($\gamma$) and the elastic modulus (E). The unit weight of the slab is quantified to be 0.022 MN/m$^3$ based on an assumed 16% water content and the calculated dry unit weight. Unfortunately the elastic modulus will need to be estimated using standard published
information found for similar material, due to the lack of usable core or samples for testing and the problems encountered using hard rock testing methods for soft rock. However, one apparently valid seismic velocity test did give an $E_{\text{dyn}}$ value of 609 MPa, which has been used in the Euler analysis in Chapter 5.

Material characterisation shows that the slab is not homogeneous but exhibits variability in lithology, water content, strength, and the amount of weathering due to wetting and drying processes. This observation is directly relevant to the simplifying assumptions used in the classical Euler analysis.

Table 4.14 Summary table of slab and failure surface data including Schmidt hammer rebound value, water content, and clay content. This illustrates the correlation of low rebound number to higher water content and clay content and also the non homogenous nature of the slab.

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Schmidt hammer rebound value ($R_\alpha$)</th>
<th>Water Content (%)</th>
<th>Clay Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fwa</td>
<td>15-20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fwb</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FSA</td>
<td>no rebound</td>
<td>31</td>
<td>31</td>
</tr>
<tr>
<td>coal/FSB</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FSC</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slab 1A</td>
<td>10-13</td>
<td>17</td>
<td>14</td>
</tr>
<tr>
<td>Slab 1B</td>
<td>no rebound - 11</td>
<td>24</td>
<td>12</td>
</tr>
<tr>
<td>Slab 1C</td>
<td>14-17</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Slab 2A</td>
<td>15-18</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>coal Slab 2B</td>
<td>no rebound</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.14 shows the relationships of rebound number, water content and clay content in Slab 1A – Slab 2B. Upwards from Slab 2B there was no rebound in the loose to compact units. The table illustrates a general correlation of low to no rebound of the Schmidt hammer to higher water content and clay content. For example, the jointed Slab 1A and Slab 1C units are dryer than the loose, moister middle material found in Slab 1B. Slab 1B is moister, less compact, and has a lower clay size fraction, and is just one example of the non homogenous and variable nature of the toe-buckle slab.
4.6 Synthesis

The geotechnical parameters, $\phi_j$, $E$, and $\gamma$ are required for the Euler analysis of toe-buckling, as described by Goodman (1980) and further discussed in Chapter 5. The present chapter has provided all the geotechnical data generated for this project, including:

- Friction results for the toe-buckle slab failure surface, showing a peak value of $\phi_p = 25.6^\circ$ and $c = 5.2$ kPa, and a residual value of $\phi_r = 3^\circ$ ($c = 0$).

- Particle size distribution of the toe-buckle slab failure surface indicated a 31% clay size fraction, a PI value of 26, normal activity (0.81), and a clay fraction dominated by kaolinite (90%) and illite (10%).

- A back calculated unit weight for the failed slab of 0.022 MN/m3, based on averaged results of 3 units, Fwa, Slab 1C, and Slab 2A.

- Schmidt hammer and other test data showing a highly variable and non-homogeneous slab with rebound values ranging from zero to 20 and indicating equivalent cube compressive strength range from $<10$ to $\sim20$ MPa.

- A single apparently valid $E_{dyn}$ value of 609 MPa for footwall mudstone/siltstone that is similar in properties to the failed slab material, yet tested as the highest Schmidt rebound value.

- An inability to determine $E_{stat}$ for the failed slab by conventional strength testing methods because of difficulties experienced in coring and field sampling of the closely laminated and weak mudstone/siltstones.

The principal conclusions from the field and laboratory testing program have been:

1. The failed slab is not homogeneous, with variable lithology, water content and strength parameters that were determined by indirect methods (Schmidt hammer).
2. The weak mudstone/siltstones are closely laminated and difficult to sample. They do not behave elastically as evident from several tests.

3. The determination of the elastic modulus value has proven difficult due to the nature of the materials and this provides limitations to the Euler analysis in Chapter 5.

4. In contrast, the failure surface testing provided satisfactory results. The ring shear testing confirmed that the laterally extensive failure surface material developed a very low friction when saturated and sheared.
5 Failure Analysis using the Euler Solution

5.1 Introduction

Chapter 5 provides Euler analysis of toe-buckling using the methods presented by Goodman (1980), Cavers (1981), and Hu and Cruden (1993). The geometric parameters quantified in Chapter 3 and the geotechnical parameters determined in Chapter 4 are applied to a back analysis of the large toe-buckle failure at the Malvern Hills Opencast Mine using the variety of approaches presented to determine the sensitivity of the Euler model.

Goodman (1980) states that;

“Natural failures represent giant “test specimens”. Due to the unknown importance of scale effects, it is far more suitable to rework field data using an appropriate model than to attempt a program of field tests, although the latter will also be useful to check assumptions and specific geological structure not represented in case histories.”

This thesis is an engineering geological investigation of the toe-buckle failure at the Malvern Hills Opencast Mine. The investigation includes testing for the parameters needed to model the problem using the Euler solution. This chapter applies those parameters to the formula, reports the outcome, and discusses the sensitivity of the parameters. It is important to note that this is not meant to be a structural engineering analysis of beam and column buckling. Williams and Todd (2000) provide in Chapter 11 “Buckling and Instability” the following description of buckling instability in engineering structures:

“....If the compressive load on a relatively slender member is increased to some critical value then the member remains in equilibrium but ceases to be stable – a very small lateral load or imperfection will cause it to bow out sideways, or buckle.”

The authors go on to comment that buckling is likely to occur at loads substantially lower than the yield strength of the material.
5.2 Euler Solution for Toe-Buckling

5.2.1 The Euler Solution

The Euler solution as used by Goodman (1980) was introduced in Chapter 4 in the following form, and is reproduced here with the geometrical parameters shown in Figure 5.1. It is important to note that Figure 5.1 is slightly misleading. L is the length of the buckled portion of the slope before the buckle occurs, and \( l \) is the length of the slope above the buckled section, therefore the overall length of slope pre-failure is \( L + l \).

\[
    l_{\text{max}} = \frac{\pi^2 Et^2 \sin(90 + \phi)}{3L \gamma \sin(\delta - \phi)}
\]

Where:

\( l_{\text{max}} \) = maximum length of slope above buckling section before failure occurs (m)

\( E \) = elastic modulus (MPa)

\( t \) = thickness of slab (m)

\( L \) = length of buckled section/span over which buckling occurs (m)

\( \gamma \) = unit weight (MN/m\(^3\))

\( \delta \) = angle of dip (°)

\( \phi_j \) = interlayer friction angle (°)
Figure 5.1 A toe-buckled slope showing the geometric parameters for the Euler solution from Goodman (1980). The L measurement is misleading in this diagram, as L equals the total length of the buckled section prior to buckling. The displaced portion at the top therefore has to be added to the “L” value as shown.

The solution uses Euler’s critical stress for buckling ($\sigma_E$), where L equals the length of the span that has buckled, and has the following expression using the symbols given above.

$$\sigma_E = \frac{\pi^2 Et^2}{3L^2}$$

A sliding slab can only move if the slide/failure surface daylights into free space and the inclination of the slip plane is greater than the friction angle of the failure surface. The 85m x 22.5m x 2m slab at the Malvern Hills Opencast Mine did not have a daylighting failure surface, but buckled at the toe. Therefore the stress parallel to the axis of the column or slab ($\sigma_t$) caused failure when it reached Euler’s critical stress for buckling ($\sigma_E$).

Buckling failure occurs when $\sigma_E = \sigma_t$, and again using the symbols above, the stress parallel to the axis is:
Figure 5.2 shows the free body diagram for stress parallel to the axis of the column or slab ($\sigma_l$), and the force polygon where $W$ equals the slab weight and $R$ is the resultant force.

![Figure 5.2](image.png)

Failure occurs when $\sigma_E = \sigma_l$, as follows:

$$\sigma_E = \frac{\pi^2 Et^2}{3L^2} = \frac{\gamma l}{\sin(\delta - \phi)}$$

Rearranging to solve for $l_{\text{max}}$, the critical failure length, gives:

$$l_{\text{max}} = \frac{\pi^2 Et^2 \sin(90 + \phi)}{3L\gamma \sin(\delta - \phi)}$$
This formula assumes that the toe of the slope is fully drained and neglects the weight of the buckled column, and is the form of the Euler equation presented in Goodman (1980), outlined at the start of Chapter 4.

The following sections present a worked example from Goodman (1980), the calculated value for $l_{\text{max}}$ using the geometrical and geotechnical parameters presented in this thesis for the Malvern Hills toe-buckle failure, and the sensitivity to various input parameters in an attempt to explain the failure mathematically. It is important to note that Goodman (1980) assumes that $L = 40\text{m}$. It is apparent that this example is a back calculation, as one would not normally have this parameter available prior to a buckle failure occurring.

### 5.2.2 Goodman’s Worked Example

Goodman (1980) provides a worked example of the calculation procedure to estimate the critical length ($l_{\text{max}}$) for buckling to occur in the rock slope shown in Figure 5.1. His input parameters for the Euler solution are shown in Table 5.1, with the Malvern Hills data found from field and laboratory investigations for comparison. Using his data Goodman calculates a value of 59.9m for $l_{\text{max}}$, which was checked and found to be correct for the input parameters in Table 5.1. The calculation shows that for the conditions modelled by the Euler solution, the slope will stand at $80^\circ$ to an overall length of 100m ($L + l_{\text{max}} = 40 + 59.9\text{m}$) before buckling occurs. This corresponds to a vertical height of 98.4m before buckling because of the steepness of the slope.

<table>
<thead>
<tr>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>E (MPa)</td>
<td>$\phi$ (°)</td>
</tr>
<tr>
<td>$\gamma$ (MN/m$^3$)</td>
<td>$t$ (m)</td>
</tr>
<tr>
<td>$L$ (m)</td>
<td>$\delta$ (°)</td>
</tr>
<tr>
<td>$l_{\text{max}}$ (m)</td>
<td></td>
</tr>
<tr>
<td>Goodman Example</td>
<td>3000</td>
</tr>
<tr>
<td>Actual Malvern Hills Data</td>
<td>609</td>
</tr>
</tbody>
</table>

Some interesting comparisons can be made between the data used by Goodman (1980) and those obtained for the Malvern Hills toe-buckle failure:
• The buckled length (L) is 40m in Goodman’s example, whereas the actual value at Malvern Hills (MH) is only 12.5m.

• The length of slope subjected to buckling is 100m (Goodman’s value), whereas the MH batter face failed when its dip length reached 22.5m (or 15m vertical height at an average dip angle of 42°).

• The ratio of L to the total length of the slope (L+l) appears to be similar for the two. Goodman’s ratio is 0.4 (40m/100m), whereas the ratio at MH is 0.56 (12.5m/22.5m).

• The thickness of the failed slab is thinner in the Goodman example (0.5m), whereas the MH slab was ~2m thick.

• The slope angle at MH for buckling to occur was much flatter (42°) instead of Goodman’s 80°, which means a significantly greater component of the slab weight was being transferred to the footwall materials at MH.

• The basal friction angles are low for both cases, 3° for the MH case and 10° for Goodman, and this means that $\phi_j$ has negligible effect on the calculation.

• The unit weights are comparable (0.022MN/m$^3$ at MH; 0.027MN/m$^3$ in Goodman), so the differences have little effect.

• The elastic modulus (E) values differ significantly, with Goodman using a figure of 3000MPa (3GPa), which still indicates a relatively low strength rock material. The indirect ($E_{dy}$) value for MH was ~600MPa and appears realistic by comparison, but is unsatisfactory in so far as the result only represents one test on the Fwa, the “strongest” unit from Schmidt hammer rebound number, and not an actual slab material.

The following section uses the Malvern Hills data to calculate the maximum length of slope.
5.3 Goodman’s Version of the Euler Analysis Using Malvern Hills Data

5.3.1 Forward Calculate $l_{\text{max}}$ from Malvern Hills Data

Using the Goodman approach and the parameters listed in Table 5.1 the computed length of slope above the buckled section is 3700 m, which is clearly incorrect (Table 5.2). The reason for this obvious discrepancy is unclear, and therefore a sensitivity analysis was carried out.

The following section takes the Malvern Hills data and attempts to cause failure ($l_{\text{max}} = 10$ m) by testing the sensitivity of the geotechnical parameters and adjusting the values to achieve failure. The geometric parameters will not be changed, as they reflect actual details of the failed toe-buckle slab.

5.3.2 How can we get it to fail?

Table 5.2 shows various attempts to test the sensitivity of the parameters for the Goodman approach. Note that the yellow cell shows the parameter that was changed and the pink cell is the result of that change.

Table 5.2 Table showing the results for the Malvern Hills data using the Euler solution per Goodman (1980). The sensitivity of the parameters $\gamma$, $\phi_j$, and $E$ are tested also. A yellow cell denotes the parameter changed, and pink cell the result of that change.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual Malvern Hills Data</td>
<td>$E$ (MPa)</td>
<td>$\phi_j$ (°)</td>
</tr>
<tr>
<td>Goodman Example</td>
<td>3000</td>
<td>10</td>
</tr>
<tr>
<td>Malvern Hills Data using $\phi_j$</td>
<td>609</td>
<td>3</td>
</tr>
<tr>
<td>Malvern Hills Data using dry $\gamma$</td>
<td>609</td>
<td>3</td>
</tr>
<tr>
<td>Malvern Hills Data using Goodman’s $\gamma$</td>
<td>609</td>
<td>3</td>
</tr>
<tr>
<td>Back Calculate $\gamma$ from Malvern Hills Data</td>
<td>609</td>
<td>3</td>
</tr>
<tr>
<td>Back Calculate $E$ from Malvern Hills Data</td>
<td>1000</td>
<td>3</td>
</tr>
</tbody>
</table>

Unit Weight (MN/m$^3$)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Malvern Hills Data using dry $\gamma$</td>
<td>609</td>
<td>3</td>
</tr>
<tr>
<td>Malvern Hills Data using $\phi_j$</td>
<td>609</td>
<td>3</td>
</tr>
</tbody>
</table>

Friction Angle

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Malvern Hills Data using $\phi_j = 10$</td>
<td>609</td>
<td>10</td>
</tr>
</tbody>
</table>

Elastic Modulus (MPa)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Malvern Hills Data using $E = 2$ MPa</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Malvern Hills Data using $E = 20$ MPa</td>
<td>20</td>
<td>3</td>
</tr>
<tr>
<td>Malvern Hills Data using $E = 1000$ MPa</td>
<td>1000</td>
<td>3</td>
</tr>
</tbody>
</table>
The results of the sensitivity analysis tabulated in Table 5.2 are broken down into the effects of changing the unit weight, friction angle, and the elastic modulus in the following sections.

5.3.3 Unit Weight Sensitivity

The unit weight ($\gamma$) has only a minor effect, and changes of the order of 20% make no significant differences. The MH data of 0.022 MN/m$^3$ is assumed to be valid. It is interesting to note that when $\gamma$ is back calculated a value of 8 MN/m$^3$ is needed to achieve the 10m maximum length of slope above the toe-buckled section, and this is totally an unreasonable unit weight for such a material. A value of 0.022 MN/m$^3$ is considered to be entirely realistic for the material observed in the field and tested in the laboratory.

5.3.4 Friction Angle Sensitivity

The friction angle ($\phi$) for Goodman’s approach was assumed to be the residual friction angle ($\phi_r$), and the measured $\phi_r = 3^\circ$ was used in Section 5.3.2 when back calculating the MH data to find $l_{\text{max}}$. Table 5.2 shows the results of using the MH peak friction angle ($\phi_p = 26^\circ$) giving a $l_{\text{max}}$ of 7602m, which is clearly not correct. Goodman’s data for the friction angle (10°) was also used yielding a result of 4333m. The MH essentially frictionless surface of 3° is therefore assumed to be valid, even though intuitively 3° seems low, especially as this is dominantly a kaolinite and not a smectite clay material.

5.3.5 Elastic Modulus Sensitivity

The elastic modulus (E) is the most questionable of all the geotechnical parameters quantified in Chapter 4. The value was determined dynamically using the seismic analyser on only one sample. The sample was part of Fwa that was cored, and also had the highest rebound number (25) with the Schmidt hammer. Most of the slab values were between zero and 20. This sample was used because it was intact and of sufficient length for the test, which is not necessarily a good reason. The platen contact was questionable and the sample was placed under compression, and some
deformation took place in the sample while timing the P and S velocities. Therefore it is assumed that this parameter is the weak link in the analysis, as no samples of actual slab materials could be tested and assumptions of isotropy, elasticity, and homogeneity are not necessarily valid.

Back calculation of E to fit the MH data yields a value of 1.6MPa. This value is very low, and as seen in Table 5.3, with values for E extracted from Lee et al. (1983), it approaches the lowest value for silts and is within the range of soft clay. The soft rock slab material at Malvern Hills was described using engineering geological description methods for soils, rather than rock, and perhaps E for the slab is represented more appropriately as a soil value.

Table 5.3  Table of values for E (elastic modulus) after Lee et al. (1983).

<table>
<thead>
<tr>
<th>Soil and Rock Type</th>
<th>E (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>silt</td>
<td>2 - 20</td>
</tr>
<tr>
<td>soft clay</td>
<td>0.5 - 3</td>
</tr>
<tr>
<td>stiff clay</td>
<td>5 - 20</td>
</tr>
<tr>
<td>mudstone</td>
<td>20,000 - 50,000</td>
</tr>
<tr>
<td>limestone</td>
<td>10,000 - 80,000</td>
</tr>
</tbody>
</table>

The value for E back calculated for the Malvern Hills data remains questionable, and its sensitivity was further tested using the approaches of both Cavers (1981) and Hu and Cruden (1993).

5.4  Cavers Version of Buckling Analysis

5.4.1 Cavers Definition of a Buckled Slope

Cavers (1981) states that buckling typically occurs in sedimentary rocks where slabs are separated by bedding planes. The failure involves buckling at the toe followed by a translational slide. Figure 5.3 shows four types of buckling, flexural buckling of a plane slope, three hinge buckling, flexural buckling of a curved slope and three hinge buckling of a curved slope.
Cavers explains that flexural buckling, as seen at the Malvern Hills site, occurs when the through going discontinuity parallel to the slope separates a thin slab of rock which is dipping more steeply than the inter layer friction angle ($\phi_j$). Failure occurs by external forces being applied to the slab such as the weight of the slab, from groundwater pore pressures, and if the slab plane is curved convexly upward as in example C of Figure 5.3. He also states that the buckle may be initiated by pore pressures along the boundary of the discontinuity, the slope geometry (upwards convex curve of slope), forces external to the slope (e.g. an earthquake), and high stresses in the slab that occur on very high and continuous slopes.

### 5.4.2 Base Friction Modelling for the Length of the Buckle

Cavers’ approach to flexural buckling applies the Euler solution for elastic buckling after making simplifying assumptions for the geometry of the slab. To identify the ratio of the span of the buckled portion of the slab to the overall length of slab he used the base friction table. The base friction table (Goodman, 1976) is a simple two-dimensional physical model using base friction to simulate the gravity loading of a
rock mass structure. This simple machine can be stopped to observe failure modes of modelled slopes, jointed rocks and underground excavations.

The kinematic model was constructed using sandpaper mounted on a board with a three sided frame to hold the modelling compound formulated to most closely represent the intact properties of the rock. The modelling compound was troweled and smoothed onto the board within the frame, and the rock defects and excavations were “cut” into the modelling compound. The modelling compound was a mixture of oil, sand and flour, all cheap and easily obtainable facilitating quick changes in mix properties. The frame was then pushed against the direction of gravity along the sandpaper board. The model is operated on a horizontal surface, but simulates the vertical. This two dimensional model mathematically simulates gravity by the transfer of force through shear on the base of the model (Figure 5.4). Another version of the base friction model uses a motor driven belt, with the model restrained from moving by a barrier (Figure 5.5).

![Figure 5.4](image)

Figure 5.4 Procedure for conducting a base shear kinematic model study of a gravity loaded rock structure. a) Using a trowel, smooth out a sheet of model material. b) Give the sheet a push to break the bond along its base. c) Cut the outline of the excavation (in this example, a rock cut with benches) and the system of discontinuities. d) Push the model against the direction of gravity. The slope is failing by toppling, with the lower limit of toppling defined by the discontinuity inclined towards the free surface (Goodman, 1976).
A study was done by Pratt (1976) to determine the thickness of the modelling compound in the frame. It was assumed that a column of constant thickness (t) would fail at some height, $H_{\text{max}}$ and a column of 2t would fail at $H_{\text{max}}$ independent of thickness. By modelling a column of variable thickness (from 2t decreasing constantly to as small as possible) the height of the column was increased to twice $H_{\text{max}}$ (Pratt, 1976).

**Figure 5.5** a) Kinematic model machine used by Golder-Brawner Associates to study strata movement in underground mining ventures. b) Kinematic modelling machine in the rock mechanics laboratory at the University of California (Goodman, 1976).

From the base friction analysis Cavers (1981) assumed that:

- The column is elastic and obeys Hookes Law.
- The slope of the deflection curve is approximated by a linear function.
- The column is weightless.
- The column is perfectly straight.
- The ratio of the overall length of slope, to the buckled length of slope must be assumed, but can be modelled using the base friction table.

The material used in the base friction table modelling had elastic modulus to strength ratios lower than rock. The results showed that a ratio of the length of buckle (L) to
the total overall slope length of 0.46 gave a slab that was too thick to buckle, and that 0.36 defines a slab thinner than that required for buckling. These were assumed to be the upper and lower bounds, and Cavers chose a slightly conservative value for rock of 0.50 (L/total overall slope length). It is worth noting here that the Malvern Hills data yields a ratio of 0.56 (12.5/22.5), only slightly greater than Cavers’ assumed value, and that Goodman (1980) did not address this aspect in his worked example. The L = 40m value was assumed, and the formula derived to solve for $l_{\text{max}}$ where the overall length of slope is understood to equal $L + l_{\text{max}}$ at failure.

5.4.3 Cavers Approach using Flexural Buckling and the Euler Solution

Cavers (1981) states that the Euler theory assumes the slab is weightless. Since the footwall slab has a finite weight, the portion of the slab above the hinge point of the buckle is assumed to be at the mid-point of the curved (buckled) length, and will add to the driving forces acting on the buckle. Figure 5.6 shows Cavers simplified model of flexural buckling on a plane slope where $\ell$ is the overall slope, $\ell_D$ is the driving segment, and $\ell_B$ is the buckled segment. There is also a component of the weight of the slab resisting movement outward from the slope during buckling. He assumes end conditions to be $K = 1$, as most rock has cross jointing perpendicular to bedding (as does the MH buckle, refer Figure 3.6).

![Simplified model of flexural buckling on a plane slope](image)

Figure 5.6 Simplified model of flexural buckling on a plane slope where $\ell$ is the overall slope, $\ell_D$ is the driving segment, and $\ell_B$ is the buckled segment (Cavers, 1980).
Using the Goodman terminology for clarity, Cavers formula for solving the overall length of slope is:

\[
(L + \ell)^{2} = \frac{\pi^{2}Et^{2}}{2.25\left(\gamma \sin \delta - \gamma \cos \delta \tan \phi_{j} - \frac{c}{t}\right)}
\]

Where:

- \(L + \ell\) = overall length of slope (m) [Cavers “\(\ell\)’’]
- \(E\) = elastic modulus (MPa)
- \(t\) = thickness (m) [Cavers uses “d”]
- \(\delta\) = slope angle [Cavers uses “\(\alpha\)”]
- \(\phi_{j}\) = friction angle (°)
- \(\gamma\) = unit weight (MN/m\(^3\))
- \(c\) = cohesion = 0 (MPa)

The variables for Cavers approach have been changed to those used by Goodman to avoid confusion. Also the units have been assumed to be the same as Goodman’s, as unfortunately Cavers (1981) does not clearly indicate the units that he used.

### 5.4.4 Applying the Malvern Hills Data to the Cavers Approach

Table 5.4 gives the geotechnical and geometrical parameters for the Malvern Hills toe-buckle and applies these values to solve for the overall slope using the Cavers approach. The result is an overall slope of 91.7 m which is the length needed to cause the slab to buckle at the toe. This is clearly not what occurred since the slope that did fail has an overall length of 22.5 m, and it therefore can be concluded that Cavers approach is not valid for determining the length of slope that would form a toe-buckle would occur in the footwall materials at the Malvern Hills Opencast Mine.
Table 5.4 Table showing the results of using the Cavers approach to solve for the overall slope length \((L + l)\).

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
<th>(L + l_{\text{max}}) (m)</th>
<th>Cavers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual Malvern Hills Data</td>
<td>609</td>
<td>3</td>
<td>0.022</td>
<td>2.0</td>
</tr>
</tbody>
</table>

5.5 Hu and Cruden Version of Buckling Analysis

5.5.1 Hu and Cruden Definition of a Buckled Slope

Hu and Cruden (1993) describe the buckling of sedimentary rocks in the Highwood Pass, Alberta, Canada where buckling occurs on steep dip slopes and the modes of buckling are determined by the bedding thickness. They use the term buckle for folds that occur close to the ground surface, and which are controlled by discontinuity spacings, stating that Euler buckling occurs where joints perpendicular to bedding are not mechanically significant and buckled beds behave like continuous beams. They assume the boundary conditions at the top of the column to be free, and at the bottom fixed.

5.5.2 Hu and Cruden’s Approach for Euler and Driving Forces to Induce Buckling

The critical buckling load \(P_{cr}\) is shown in the following formula where \(I\) is the moment of inertia and is modelled after the Euler solution:

\[
P_{cr} = \frac{\pi^2 EI}{4L^2}
\]

Hu and Cruden describe the driving load \(P_d\) as the upper portion of the slab over the lower portion of the slab \((L)\) that is buckled although what this means is unclear from the paper. \(P_d\) can be calculated from the weight of the driving portion of the bed, the angle of dip of the bed, the friction angle of the failure surface, and cohesion.

Hu and Cruden use a ratio of \(L\) to overall length of slope where:
or 0.67.

Therefore using the terminology above Hu and Cruden’s approach to solving for the overall length of slope is:

\[
(L + \ell)^3 = \frac{9\pi^2 E t^2}{64 \left( \gamma \sin \delta - \gamma \cos \delta \tan \phi - \frac{C}{t} \right)}
\]

As with Cavers (1980), Hu and Cruden do not specify the units to be used for the various parameters in this formula.

5.5.3 Applying the Malvern Hills Data to the Hu and Cruden Approach

Table 5.5 uses the values for the geotechnical and geometrical parameters from Malvern Hills and solves for the overall slope length using the Hu and Cruden approach. An overall slope of 62.5m is calculated where the actual slope is 22.5m. The Hu and Cruden approach would not be an effective method of forward calculating the batter stability on this basis, although it is closer than the estimate by the Cavers approach.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
<th>$L + \ell_{\max}$ (m) using Hu &amp; Cruden</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual Malvern Hills Data</td>
<td>609</td>
<td>3</td>
<td>0.022</td>
</tr>
</tbody>
</table>

5.6 Review of Buckle Analysis Results

5.6.1 Comparison of Goodman, Cavers, and Hu and Cruden Methods

The analysis of the Malvern Hills data is unsatisfactory as the Goodman approach estimates the maximum length of slope above the buckle before buckling to be 3.7km!
The Cavers and Hu and Cruden methods state that they solve for the overall length of slope. The results using the Malvern Hills data show a maximum overall slope of 91.7m for the Cavers method, and 62.5m for the Hu and Cruden method results that are on a significantly different scale from the Goodman method.

To reconcile this difference, and to check the reliability of the three methods, Goodman’s data was used as the values for the parameters in the three methods. Table 5.6 shows the results and also the L/(L+l) ratios used for each method keeping the Goodman terminology of \( L = \) buckled slab length before failure and \( l = \) the slab above the unbuckled slab.

Table 5.6  Goodman data is used in each of the three approaches to the Euler solution. The results for the overall length of slope (\( L + l \)), and the ratio stated by each method for the buckled length (\( L \)) to overall length, are shown.

<table>
<thead>
<tr>
<th></th>
<th>( L ) (m)</th>
<th>( l ) (m)</th>
<th>( L + l ) (m)</th>
<th>( L/(L+l) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goodman Approach</td>
<td>40.00</td>
<td>59.90</td>
<td>99.90</td>
<td>0.40</td>
</tr>
<tr>
<td>Cavers Approach</td>
<td>unknown</td>
<td>unknown</td>
<td>50.35</td>
<td>0.50</td>
</tr>
<tr>
<td>Hu and Cruden Approach</td>
<td>unknown</td>
<td>unknown</td>
<td>34.40</td>
<td>0.67</td>
</tr>
</tbody>
</table>

If one were to assume that these formulae were sound, one could also assume that the Cavers and Hu and Cruden formulae were actually solving for \( l_{\text{max}} \), or the length of slope needed above the buckled section for buckling to occur. Table 5.7 does just this by changing the results in the Cavers and Hu and Cruden method to reflect \( l_{\text{max}} \) instead of the overall slope (\( L+l \)). Using the ratio of \( L \) to (\( L+l \)) the results make more sense and the overall slope is \( \sim100\text{m} \) for each of the approaches with the slight differences due to the ratios of \( L/(L+l) \) assumed by Cavers and Hu and Cruden.

Table 5.7  The Goodman data used to check the validity of Cavers and Hu and Cruden approaches to solve for \( l_{\text{max}} \) not the overall length of slope (\( L+l \)).

<table>
<thead>
<tr>
<th></th>
<th>( L ) (m)</th>
<th>( l ) (m)</th>
<th>( L + l ) (m)</th>
<th>( L/(L+l) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goodman Approach</td>
<td>40.00</td>
<td>59.90</td>
<td>99.90</td>
<td>0.40</td>
</tr>
<tr>
<td>Cavers Approach</td>
<td>50.35</td>
<td>50.35</td>
<td>100.70</td>
<td>0.50</td>
</tr>
<tr>
<td>Hu and Cruden Approach</td>
<td>68.80</td>
<td>34.40</td>
<td>103.20</td>
<td>0.67</td>
</tr>
</tbody>
</table>
For the Malvern Hills data, the computation were carried out following the logic in the worked Goodman example (Table 5.7) and it is assumed that Cavers and Hu and Cruden are computing $l_{\text{max}}$ and not the overall length of slope. The results are shown in Table 5.8.

Table 5.8 Results for the Malvern Hills data using three different approaches, and assuming that the parameter solved in the Cavers and Hu and Cruden formulae was for $l_{\text{max}}$. Note that the Goodman approach only computes $l$ (m); 12.5m for L is the observed buckle dimension.

<table>
<thead>
<tr>
<th>Results Using Malvern Hills Data</th>
<th>L (m)</th>
<th>l (m)</th>
<th>L + l (m)</th>
<th>L/(L+l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goodman Approach</td>
<td>12.50</td>
<td>3699.50</td>
<td>3712.00</td>
<td>0.003</td>
</tr>
<tr>
<td>Cavers Approach</td>
<td>91.70</td>
<td>91.70</td>
<td>183.40</td>
<td>0.500</td>
</tr>
<tr>
<td>Hu and Cruden Approach</td>
<td>125.00</td>
<td>62.50</td>
<td>187.50</td>
<td>0.667</td>
</tr>
</tbody>
</table>

If the Cavers and Hu and Cruden (L+l) values are divided by 10, values of 18.34m and 18.75m are obtained for the overall length of slope. These are plausible values since they are only a few metres less than the actual slope that failed. Similarly if the computed “l” value in the Goodman analysis is divided by 1000 a value of 3.70m is obtained and this would give an (L+l) value of 16.2m which again is plausible for the critical length of slope at failure.

To get these changes, either $\gamma$ would need to be in kN/m$^3$ or E would have to be in GPa, but not both. Since the cube root of 1000 equals 10, and both Cavers and Hu and Cruden compute $l^3$, it is tempting to infer an error of this magnitude in the Malvern Hills data. However the Goodman data provides sensible results as long as the parameter of (L+l) is changed to l for both Cavers and Hu and Cruden. A dimensional analysis was therefore carried out (Section 5.6.2) to check the units.

### 5.6.2 Dimensional Check and Comments

The assumption that Goodman, Cavers, and Hu and Cruden were all working with the same units is questioned in this section. Goodman was the only author to state the units used, which is surprising for internationally refereed papers. A review of the units and formula provided by Goodman follows.
Chapter 5  
Failure Analysis using the Euler Solution

\[ l_{\text{max}} = \frac{\pi^2 Et^2 \sin(90 + \phi)}{3L\gamma \sin(\delta - \phi)} \]

Where:

- \( l_{\text{max}} \) = maximum length of slope above buckling section before failure occurs (m)
- \( E \) = elastic modulus (MPa or MN/m\(^2\))
- \( t \) = thickness of slab (m)
- \( L \) = length of buckled section/span over which buckling occurs (m)
- \( \gamma \) = unit weight (MN/m\(^3\))
- \( \delta \) = angle of dip (°)
- \( \phi \) = interlayer friction angle (°)

Substituting the units in the equation and solving for \( l_{\text{max}} \) in meters.

\[ l_{\text{max}} = \frac{D^2 \frac{MN}{m^2} \times \frac{D}{MN \times m^2}}{3m^2} = \frac{m^3}{m} = m \]

Where \( D \) is used for a dimensionless term.

The formula for the Cavers method was similarly analysed as follows:

\[ (L + \ell)^3 = \frac{\pi^2 Et^2}{2.25 \left( \gamma \sin \delta - \gamma \cos \delta \tan \phi_j - \frac{c}{t} \right)} \]

Substituting the units:
Cavers (1981) formula is therefore dimensionally correct, provided $\gamma$ is in MN/m$^3$ and $E$ is in MPa (or equivalent). Hu and Cruden use essentially the same formula, and can be considered to be dimensionally correct also, with “l” in units of metres.

The conclusions from this dimensional analysis are:

- Either MPa for the elastic modulus ($E$), and MN/m$^3$ for unit weight ($\gamma$), or kPa and kN/m$^3$ respectively, must be used.
- It is not possible to use other combinations of units (e.g. kN/m$^3$ and GPa) without introducing errors.
- Neither Cavers nor Hu and Cruden indicate the actual units to be used, a most surprising situation.
- The units for the Goodman data are valid for all the methods, and therefore the formula must be consistently used for the Malvern Hills data.

5.6.3 Conclusions Regarding Euler Analysis

After careful review of the Euler method for column and beam buckling, it has to be concluded that it is not applicable to the flexural slip toe-buckle failure that occurred at the Malvern Hills Opencast Mine. It has been shown that none of the methods used (Goodman, 1980; Cavers, 1981; Hu and Cruden, 1993) provided realistic $l_{\text{max}}$ values using the Malvern Hills data. The units were checked by dimensional analysis to confirm the units used for the Cavers and Hu and Cruden formula.
The alternative conclusion, that the $E$ value obtained from the Malvern Hills testing was wrong, must also be rejected. The value of 600MPa is reasonable for the compacted mudstone/siltstone, and the 1.6MPa value for the elastic modulus to obtain slab failure cannot be correct as this is less than published values for loose silt.

Certainly the lack of a robust set of $E$ data is a weakness but the conclusion seems inescapable that the Euler method does not work for the Malvern Hills soft rock materials.

### 5.6.4 Other Analysis Methods

Traditionally two dimensional limit equilibrium techniques are used to model footwall failures incorporating the Euler elastic column analysis. Stead and Eberhardt (1997) used numerical modelling techniques, specifically the distinct element method, to investigate the failure mechanisms in footwalls with some success. They suggest that failure by elastic buckling of beds according to the Euler theory is rare unless very thin beds exist, and that pure flexural Euler buckling does not appear to be a realistic failure mechanism. Therefore according to Stead and Eberhardt (1992) these methods should not be used to predict the maximum length of a stable slope, and this has certainly been confirmed in the detailed analysis carried out using the Malvern Hills data.

Distinct element modelling offers a means to solve more complex problems in mining geotechnics. Stead and Eberhardt (1997) state that the simple geometry of a coal mine footwall makes constructing the model fairly straightforward. The intact deformation of the slab uses assumed representative values for the elastic constants, friction angle, cohesion and tensile strength. Slab geometry of varying thickness and lengths can be tested easily, and the failure models can be repeated with slight changes to test for sensitivity.

After several inquiries about the distinct element method, specifically UDEC, I was fortunate to take part in the UDEC training course run Michael Coulthard of M.A. Coulthard and Associates from Malvern, Victoria. It has proven impossible to carry out the necessary analyses within the time frame of this thesis. Email discussion with
Doug Stead at Simon Fraser University, Canada, elicited interest but no practical assistance in solving this problem. Regrettably a full UDEC analysis cannot be carried out at the present time in New Zealand, and this has been identified as the main recommendation for further work arising from this thesis. However, the UDEC analysis will require a more robust set of elastic modulus values for the Malvern Hills slab failure if an elastic type analysis is realistic.

5.7 Synthesis

Chapter 5 is an analysis of the Malvern Hills geotechnical and geometric data using the Euler method of flexural slip buckling prediction. A worked example from Goodman (1980) was the basis of the analysis, and similar methods in Cavers (1981) and Hu and Cruden (1993) were applied to the data also.

The Euler method of analysis of toe-buckle failures assumes simple flexural slip and toe-buckling of the slab on a low friction surface and no pore pressures. The main geotechnical controls are the friction angle of the failure surface, the unit weight of the slab, and the elastic modulus of the slab materials. There are assumptions of homogeneity and the elastic modulus of the slab for using the Euler method, yet Hu and Cruden (1993) claim success using the method analysing observed buckling failures. Buckling failures in footwall slopes are also well known (Goodman, 1980).

From the start it became apparent that there were problems in the analysis methods used by Cavers and Hu and Cruden, as neither formula dealt satisfactorily with the Goodman worked example. After dimensional checks the parameter “l” used by both Cavers and Hu and Cruden was in fact the length of the unbuckled slope segment, and not the total length of the slope as claimed.

When applied to the Malvern Hills data where the buckled length was 12.5m (L) and the unbuckled length was 10.0m (l) the Goodman approach gave an “l” value of 3.7km. The Cavers and Hu and Cruden method, adjusted to solve for “l” gave values of 91.7m and 62.5m respectively and a total overall length of the slope using the ratio assumed in their formula of 183m and 187m. None of these values are realistic for the toe-buckle slab failure at Malvern Hills.
Stead and Eberhardt (1997) suggest that the UDEC method of numerical modelling is an appropriate method for analysing buckling in footwall slopes, but unfortunately this type of analysis could not be done. The elastic modulus is an important parameter in both Euler and UDEC analyses and would have to be more robust than just one value from one sample in order to result in sound UDEC modelling. This would be difficult to achieve in the variable, non-homogeneous, soft rock material of the failed slab.

This part of the project has shown just how difficult it is to carry out a realistic quantitative analysis of toe-buckling in soft rocks that behave in many ways as stiff engineering soils. Therefore it is important to go back to first principles and look at the engineering geology of the failure to understand what triggered it and how it may be prevented in the future. The original intention of this thesis was to provide quantitative analysis evaluating the bench and batter parameters in order to modify the pit design to prevent buckling failures, but this outcome has not been possible.
6 Engineering Geology Reassessment of the Failure

6.1 Introduction

The Euler method of toe-buckling analysis has been shown to be unsatisfactory at the Malvern Hills Opencast Mine site for a number of reasons (Chapter 5):

- Modelling assumes that the slab is a homogeneous, isotropic and elastic material. This is not the case for the laminated mudstone/siltstone material in the footwall.

- Of the three geotechnical parameters required (E, \(\gamma\), \(\phi\)) only the unit weight (\(\gamma = 0.022\text{MN/m}^3\)) and the residual friction angle (\(\phi_r = 3^\circ\)) have been satisfactorily determined by laboratory testing.

- The determination of the elastic modulus (E) has been poorly constrained by this study because of the weak nature of the soft rock material making it impossible to core satisfactorily as well as the difficulties of testing using \(E_{\text{dyn}}\).

- Accepting that the value for \(E = 600\text{ MPa}\) is an upper bound value, the back calculated value required is \(\approx 2\text{MPa}\) and this is also unrealistic.

- Euler method derived from structural engineering methods for column and beam bending is thus not a satisfactory method for analysing the toe-buckle failure at Malvern Hills. So therefore in the absence of access to UDEC software for further analyses, as recommended by Stead and Eberhart (1997), it is necessary to go back to first principles and consider the site engineering geology and the reconstructed failure history.

6.2 Overview of Buckled Slab

The toe-buckle failure in the footwall of the Malvern Hills Opencast Mine that occurred immediately after formation of the batter below the 370 RL bench extended almost the entire length of the pit (85m), with side release joints allowing a 2m thick
slab to buckle and translate ~6m down dip as one complete intact unit. The failed slab had an estimated volume of 3,700m$^3$ and weighed over 8,000 tonnes.

The Malvern Hills toe-buckle failure is composed of a non homogenous slab containing laminated mudstone/siltstones that have contrasting moisture content, strength, and oxidation/weathering. Two thin coal seams are present, one within the slab material and the other within the failure surface, and the latter is clearly sheared with clay rich margins.

The slab itself was pre sheared by flexural slip folding of the units resulting in a syncline axis in the pit floor and an anticline axis in the highwall separated by a thrust fault that displaced the entire pit sequence and replicated it in the highwall. This shearing involved movement in the thin coal seam, and broke some of the coal into small granule sized pieces. The basal thin coal seam when excavated at the heel of the buckled slab had free draining water, and thus it is assumed that water had been stored in the cleat and voids of the fractured coal.

Fluctuating water levels in the thin coal seam “aquifer” caused the laminated mudstones above and below the saturated coal to weather over time, and become more clay rich (31% <2μm compared to 14% tested in the other slab materials). The water content for the weathered/clayey laminated mudstone/siltstone next to the failure surface is calculated to be 31%, which is at the plastic limit for the material. In contrast the water content of the slab further away from the thin basal coal seam is 17% to 24%, and then back to 16% in the units above, showing that the slab is not homogenous in water content, clay content, or strength as determined by the Schmidt hammer.

It may be assumed that the clayey laminated mudstone/siltstones of the failure surface (FSA and FSC) had taken on some of the free water from the thin coal seam “aquifer”. This was seen in the laboratory during ring shear testing of FSA. The material was placed in the ring shear apparatus, a normal pressure was applied and water added to a water bath to prevent drying of the sample. Instead of further consolidation taking place the material began to swell. At the end of testing for the
residual friction of the material, which resulted in a very low $\phi_r$ value of $3^\circ$, the water content was tested and shown to have risen from 31% to 42%. A similar unloading and loading history occurred on the failed batter.

### 6.3 Loading History of the Slab Pre-Failure

The loading and unloading history of the batter material began with the original excavation of the pit. Two batters were originally formed, one at 385 RL and one at 370 RL. The batter below 370 RL had some thin slab delamination failures along the bedding, and the batter below the 385 RL bench had a thick (1.5m) slab failure on a side release joint. The pit was deepened and the 385 RL bench narrowed, with the material pushed over the batter loading the 370 RL bench and batter below. Then the new 370 RL bench was formed and more material pushed over onto the batter below. Finally the batter below the 370 RL bench was formed in a matter of days as the material easily parted on a thin coal seam to form the batter. When the last of the material was removed from the batter to form the 355 RL bench, the Main Seam coal was exposed ready for extraction. The next morning the failure, a 2m thick slab over 85m of strike, translated down dip along the discontinuity formed by a thin wet coal seam with clayey very wet laminated mudstone/siltstones immediately above and below.

It is reasonable to infer that the unloading and then loading of the batter above the 355 RL bench caused the free draining water in the thin sheared and broken coal seam to wet up the adjacent clay rich materials, and as in the ring shear test result in some swelling of the clayey laminated mudstones/siltstones in contact with the coal. Even though the clay mineral is mostly kaolinite and not a high swelling clay, the material is assumed to be at the plastic limit before loading, and during compression the water was taken up in the clay just as it did during the ring shear test.

While the batter below the 370 RL bench was being formed a photo was taken along the 370 RL bench (Figure 3.6 F). In this photo it is easy to see the expression of the failure surface on the bench floor. This is apparent because of the thin coal seam, clay
rich and reddish change in colour of the laminated mudstones/siltstones, and perhaps because there is movement on the surface, appearing not as a tension crack but creep.

The implication from field observations is that the thin clay rich sheared coal seam, which has been previously formed the batter above the 370 RL bench (Figure 2.12), was in fact storing significant water. The relatively rapid loading and then unloading of the slope during batter formation caused both pore pressure effects and strength lowering to residual friction angles by wetting up. They together these created a situation where the rapidly unloaded face was “jacked” outwards at the base. Also of note is the occurrence of a small magnitude earthquake that morning which may have provided sufficient external force to assist in exceeding the critical stress for buckling of the slab at its base, pushing out the toe a small amount.

6.4 Comment on the Euler Analysis

Using the Euler solution, the structural engineering formula for column and beam bending, and three different approaches (Goodman, Cavers, Hu and Cruden), it was found that the toe-buckle slab at Malvern Hills could not have been predicted mathematically. All three methods gave unrealistically long overall slopes (180m to 3.7km), and there have to be explanations for this other than variations on the “E” value used in calculation.

Goodman, Cavers, and Hu and Cruden agree that flexural buckling of a slab occurs mostly in sedimentary rocks along a boundary of discontinuity where the critical force for buckling of the slab material has been exceeded by an external force, e.g. the slab weight, groundwater pressures, and/or other external forces. The formulae for all the approaches do not factor in pore water pressure. Cavers (1981) and Hu and Cruden (1993) use an assumed ratio of the length of the buckled section of the slope to the overall slope in their equations to define the driving slab force. The Malvern Hills geometric parameters show a $L/(L+l)$ ratio of 0.56 within the values of Cavers and Hu and Cruden (0.50 and 0.67 respectively).

With the benefit of hindsight it may have been unrealistic to expect the Euler method to provide a means of assessing the critical slope height before buckling occurred. An
early aim of this project was to establish a value for $l_{\text{max}}$ that would have allowed reduction in vertical bench separation to a value (say $\sim 12$ m) that would prevent future buckling failures that presented safety risks to staff and the economics of the project. However, the assumptions for the Euler method were clearly not met (for example, the assumption of a dry slope), and in the circumstances the nature of the soft rock materials and the experience of this major toe-buckle failure provide a pragmatic basis for mine planning without the benefit of quantitative forward analysis.

### 6.5 Pragmatic Engineering Geological Approach to Toe-Buckle Failures

The Malvern Hills toe-buckle slab failure occurred rapidly upon final excavation of the toe of the batter. The 85 m long and 2 m thick slab buckled at the toe and translated down dip $\sim 6$ m on a failure surface composed of a previously sheared thin fractured coal seam. This contained free draining water, weathering of the laminated mudstones/siltstones above and below to a clay rich material at its plastic limit had occurred, and this slope was then loaded and the clay materials swelled. As a result, the clay materials dropped to a low residual friction angle, the critical buckling stress was reached and the driving slab initiated the buckling failure following slight flexing of the lower slope segment.

The non homogenous slab was thicker and its buckled length to overall length were greater than the Euler solution using three approaches (Goodman, Cavers, Hu and Cruden) could reconcile mathematically for a dry slope. It is therefore assumed that the external forces to exceed the Euler critical stress for buckling were of great importance in this failure. The pore water pressures from the coal “aquifer” acted on the slab base, which had a very low residual friction angle. Ground acceleration from an earthquake of magnitude 3.1 on the Richter scale at a depth of 9.1 km in the immediate vicinity of the mine also contributed to the external forces acting on the slab, resulting in the critical Euler stress being exceeded. Once the Euler critical stress for buckling was exceeded the toe of the slab began to buckle, and allowed the slab to translate down dip rapidly as there was not sufficient basal restraint.
During the mine design stages it was assumed that the moderately dipping beds of the coal bearing units could be used as the batter surface with vertical separation between benches of 15m in the footwall. During excavation and forming of the batters it was found that the batters formed easily, and failed back to thin coal seams. Fortunately for the mine design configuration, the thin coal seams were strategically placed and the mine design did not need to be altered greatly, simply accommodating the thin coal seam surfaces for the batters. There was no reason to think that the slope lengths and the thin coal seam separations would allow the toe-buckle failure of a slab of the sort of magnitude of the 1 July 2004 failure, which could have injured mining staff or damaged equipment had they been working below the face at that time. The inconvenience in this case was the time needed and cost to remove the failed material.

The failure model now proposed, based on pragmatic engineering geology reasoning rather than quantitative geotechnical analysis, is shown in Figure 6.1. This is based on the model presented in Figure 3.11, and the focus is on the toe-buckle failure mechanism.

### 6.6 Implications for Future Mining

As shown in Figure 6.1, lowering the pit floor to 340 RL from 355 RL will create a similar footwall failure situation on the down dip toe-buckle failure surface. Just as before, the batter drain on the 355 RL bench is located in the thin coal seam of the toe-buckle failure surface, and is continuing to recharge with ground water, so the clay rich materials will be wet. (Note that the drains pipes drilled into the batter below the 355 RL bench only drain the water in the footwall above that level.) The potential scenario for future failures would be as follows:

- Lowering the pit floor to 340 RL along most of the strike length to expose coal against the footwall.

- Rapid coal extraction to one to two metres below the 340 RL floor level (it is common to extract below the planned floor level).
Further unloading of the footwall between 355 and 340 RL to excavate back to material that does not contribute to the acid mine drainage problem at the mine.

Figure 6.1  Figure taken from Chapter 3 and reproduced here to show the Engineering Geological model of the pit and the potential failure onto the 340 RL bench shown in red.

Since the Euler method has not provided satisfactory results for the Malvern Hills case and UDEC is not available, it is realistic to use the precedent approach for future planning of the pit. Future footwall stability can be achieved by:

- Either not removing material below the base of the Main Seam and leaving footwall material in place to support the base of the batter.
- Or draining the recognised failure surface at least mid way between the 355 RL and 340 RL floor in an attempt to keep the surface dry and increase the friction on the driving portion of the slab ($I_{max}$).

Other options such as extracting in sections and backfilling the pit to prevent a long exposure of the slope along strike are available but not practical. Considering the
options drainage of the recognised failure surface is the best. Significant pressures would not be allowed to build up, and therefore result in a decreased friction angle on the dryer clay rich materials. The placement of several sub horizontal drains at 20m spacing along the footwall by drilling into the midpoint of the batter (say 348 RL) to the appropriate depth to intersect the perceived failure surface with slotted PVC casing is suggested. Regular maintenance would be necessary to keep them clear and free flowing. This pragmatic approach to future mining is strongly recommended, along with daily visual observation during extraction, and is preferred to attempting an analysis using UDEC or other methods with potentially uncertain results.
7 Summary and Conclusions

7.1 Project Aims and Methods

This thesis describes and analyses a 3700m$^3$ toe-buckle slab failure that occurred in the footwall of the Malvern Hills Opencast Mine on 1 July 2004. The project had the following specific aims:

- To describe the site engineering geology and develop a suitably detailed model of the 1 July 2004 failure.
- To characterise both the physical and mechanical properties of the materials involved using direct and indirect test methods.
- To apply the Euler solution for column and beam buckling to assess the validity and sensitivity of this analytical method to account for the observed buckle failure geometry and dimensions.
- To use the above techniques to develop a geotechnical model for the toe-buckle failure, and to recommend forward mine design and practice.

It is important to record that this thesis is based on pit excavation to 350 RL, and does not refer (except indirectly) to subsequent pit development and coal extraction.

7.2 Engineering Geology of Malvern Hills Opencast Mine

The Malvern Hills Opencast Mine is located in inland Canterbury and is part of the Malvern Hills Coalfield. The coal horizons are in the Broken River Formation of Late Cretaceous age, with lensoidal seams of ~1-3m in thickness, and dips to the southeast of ~45°. The Malvern Hills Opencast Mine is the first large open pit developed in the coalfield, the previous workings being underground.

The engineering geology of the pit is as follows:
Footwall – Laminated mudstones/siltstones with associated thin seams of coal. The footwall batters are cut back to parallel the \( \sim 45^\circ \) bedding surface, and the benches are 5m wide with 15m vertical separation.

Pit Floor/Coal Horizon – Two economic seams, the 1.5 m Engine Seam and 3m Main Seam separated by \( \sim 5 \)m of mudstone/siltstone interburden.

Highwall – Thin coal seams, laminated mudstones/siltstones, sands and loess cover. The highwall batters are formed at 76\(^\circ\) (4V:1H) and the bedding dips into the face.

The footwall sequence examined in this study is subject to:

- Delamination and sliding failure on the scale of 1 - 10m\(^3\).
- Slaking and frittering to form aprons of fine debris.
- Large scale toe-buckle slab failure on a 21m dip slope over the entire length of the pit.

This study has focused only on the toe-buckle failure that occurred on 1 July 2004, which developed between the 370 RL and 355 RL benches immediately after the batter formation.

### 7.3 Geotechnical Characterisation of Materials

The engineering geological model for the footwall toe-buckle failure comprises the following four units:

- The footwall soft rock mudstones/siltstones immediately below the failed slab are relatively unweathered, and based on the Schmidt hammer data have the highest strength of the units tested.

- The failure surface is divided into three sub units, with a thin, wet sheared coal seam between clay rich laminated mudstones above and below. The toe-buckle failure took place on the lower clay rich unit.
• The failed slab is 2m true thickness, and consists of eight logged sub units one of which is a thin coal seam and the remainder mudstones/siltstones.

• The hanging wall soft rocks stratigraphically above the slab comprise laminated mudstones/siltstones and stoney coal.

Three geotechnical parameters were needed for the Euler analysis, along with the geometry of the slab. Table 7.1 shows the seven parameters used in the Euler solution for toe-buckle analysis.

**Table 7.1 Parameters used in the Euler solution analysis of the Malvern Hills toe-buckle failure of 1 July 2004.**

<table>
<thead>
<tr>
<th>Geotechnical Parameters</th>
<th>Geometric Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus (E) (MPa)</td>
<td>Interlayer Friction Angle (φ) (°)</td>
</tr>
<tr>
<td>Unit Weight of Slab (γ) (MN/m³)</td>
<td>True Thickness of Slab (t) (m)</td>
</tr>
<tr>
<td>Length of Buckled Section (L) (m)</td>
<td>Angle of Dip (δ) (°)</td>
</tr>
<tr>
<td>Length of Slab Above the Buckled Section (lmax) (m)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>E (MPa)</th>
<th>φ (°)</th>
<th>γ (MN/m³)</th>
<th>t (m)</th>
<th>L (m)</th>
<th>δ (°)</th>
<th>lmax (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>609</td>
<td>3</td>
<td>0.022</td>
<td>2.0</td>
<td>12.5</td>
<td>42</td>
<td>10</td>
</tr>
</tbody>
</table>

Of the geotechnical parameters the unit weight (γ) was obtained from the average of several slab samples (0.022 MN/m³), and it was shown during testing that the water content of the slab varied between 16% and 24%. Particle size analysis was done on several of the slab units, and showed that the laminated mudstone/siltstone contained 13% clay (mostly kaolinite), 60.5% silt, and 26.5% sand. The solid density of the “soil” particles in the slab was quantified at 2.40 tonnes/m³.

The value for the interlayer friction angle (φi) in the Euler solution was assumed to be the residual friction angle of the failure surface, although both the peak (φp) and residual (φr) friction angles were quantified by direct shear and ring shear testing respectively. The residual friction angle results were a very low 3° showing a continuous smooth, wet and polished shear surface, with the material increasing in moisture content from 31% to 42% and some swelling occurring during consolidation.
The direct shear testing gave $c = 5\text{kPa}$ and $\phi_p = 25.6^\circ$, with a water content of 31% before and after testing which was also its plastic limit.

Testing for the elastic modulus (E) was not possible using compressive strength analysis methods for $E_{\text{stat}}$ due to the lack of suitable core. A single $E_{\text{dyn}}$ value of 609MPa was determined for a piece of core from the footwall, as no core could be recovered from the slab. It is acknowledged that the E value obtained is probably high.

### 7.4 Euler Back Analysis of Buckling Failure

The structural engineering Euler formula for column and beam buckling has been used by several authors to analyse toe-buckle footwall failures in coal mines. Three different approaches were used to analyse the Malvern Hills toe-buckle failure (or flexural buckle). A worked example provided by Goodman (1980) was used as the basis of the analysis, and two further methods (Cavers, 1981; Hu and Cruden, 1993) were used as cross checks and to calibrate the Euler model. Essentially the Euler solution calculates the critical stress for buckling, and when this stress is exceeded by external forces (such as the weight of slab) buckling of the toe occurs. The formula can be used to forward calculate an overall stable length for the slope, or may be used to back analyse a failure. The results obtained are given in Table 7.2.

<table>
<thead>
<tr>
<th>Results Using Malvern Hills Data</th>
<th>L (m)</th>
<th>l (m)</th>
<th>L + l (m)</th>
<th>L/(L+l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goodman Approach</td>
<td>12.50</td>
<td>3699.50</td>
<td>3712.00</td>
<td>0.003</td>
</tr>
<tr>
<td>Cavers Approach</td>
<td>91.70</td>
<td>91.70</td>
<td>183.40</td>
<td>0.500</td>
</tr>
<tr>
<td>Hu and Cruden Approach</td>
<td>125.00</td>
<td>62.50</td>
<td>187.50</td>
<td>0.667</td>
</tr>
</tbody>
</table>

The results clearly do not match the actual Malvern Hills toe-buckle failure dimensions, which have an L value of 12.5m, an l value of 10m, overall slope length of 22.5m, and a ratio of 0.56. The Euler solution is not an appropriate analytical method for the Malvern Hills toe-buckle failure. In fact the results would give a false
sense of safety if one were to use this solution to forward calculate the safe overall length of a footwall slope.

7.5 **Engineering Geology Reassessment**

There are a number of possible reasons for the inability of the Euler solution to provide meaningful data on the Malvern Hills toe-buckle failure. These include:

- The Euler analysis used by Goodman assumes the slope is dry, which is not correct as water was observed draining from the crushed coal during field investigations.

- The Euler solution in its various forms is not applicable to the soft rocks present at the Malvern Hills Opencast Mine, in part because of the problem in determining a robust value for the elastic modulus of the slab material.

- The limited core testing confirmed that the mudstone/siltstone materials were not fully elastic, with plastic deformation being observed, and yet the Euler solution assumes that the slab is elastic and obeys Hooke’s Law.

- External factors such as a small earthquake in the vicinity on the morning of the failure may have been enough to cause dilation of the slope toe that initiated buckling on an essentially frictionless shear surface.

For these reasons the Euler solution has been rejected as a method of analysing footwall buckling. Instead it is suggested that a pragmatic engineering geology approach using slope drainage and regular slope monitoring and inspection be implemented at the mine site. Horizontal drains (or similar) can easily be constructed to drain at least the top half of each benched interval, thereby effectively stabilising the driving slab. Engineering geology precedent is an entirely reasonable method of solving practical problems in mining and civil engineering.
7.6 Further Work

In terms of modelling the 1 July 2006 toe-buckle slab at Malvern Hills OpenCast Mine specific aspects warranting further research include:

- The determination of a robust set of values for the slab elastic modulus (E) would permit better estimates of $l_{\text{max}}$ using forward analysis methods.

- Distinct element numerical modelling (UDEC) has been successful in analysing similar toe-buckle failures, and would be a valuable method for refining possible failure geometries and parameter sensitivity.

- Development of a Euler analysis method that would incorporate the stress produced by pore water pressure on the slab may provide more realistic $l_{\text{max}}$ values.

- Careful field observations should be undertaken throughout future mining campaigns to provide a robust engineering geology database.

All of the above require appropriate budget allocations and/or more detailed sampling and testing that were possible in this study.
References


References


Resource Consent Applications to Canterbury Regional Council, Central Plains Water Trust, Christchurch.


Appendix A

A.1 Copy of David Bell’s Field Notebook with comments on toe-buckle failure of 1 July 2004.

Figure A.1 Copy of David Bell’s field notebook showing his description of the toe-buckle failure on 1 July 2004.
Figure A. 2 Copy of David Bell’s field notebook showing further description of the toe-buckle failure (from Figure A.1) and description of toe-buckle failure material removal on 6 July 04.
Appendix B

B.1 Engineering Geological Description for Rock Material

Figure B.1 Engineering Geological Field Description for Rock Material, the standard used in text (Bell and Pettinga, 1983).
B.2 Engineering Geological Description for Soil Material

Figure B.2 Engineering Geological Field Description for Soil Material, the standard used in text (Bell and Pettinga, 1983).
Appendix C - Testing Methods

C.1 Water Content


For fine soils, clean, dry, and weigh container with lid to at least 0.01 g and record. Place at least 30 g of the soil in the container and replace lid. Weigh to at least 0.01 g and record. Remove lid and place container, contents, and lid in drying oven (105°C). The sample shall be deemed to be dry when the change in mass of the cooled sample after successive drying periods of 4 h at 105°C does not exceed 0.1% of the dry mass of the sample. After drying, replace lid and allow to cool in a desiccator. Weigh the container, contents, and lid to 0.01g and record.

Calculate the water content of the soil (w) as a percentage of the dry soil mass from the formula:

\[ w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \]

Where:

\( M_1 \) = mass of the container in (g)

\( M_2 \) = mass of container and wet soil (g)

\( M_3 \) = mass of container and dry soil (g)
C.2 Liquid Limit

NZ 4402:1986 Standards, Part 2 Soil classification tests, 2.2 Determination of the liquid limit.

Sample material should be well mixed, but not mixed with layers of soil which may have different plasticity. Use whole soils at natural content when all material would pass a 425\(\mu\)m test sieve or fraction of whole soil passing a 425\(\mu\)m test sieve. Place approximately 250g of specimen on mixing plate and mix thoroughly at either natural water content or with addition of distilled water in small increments to make the material workable. Add distilled water until water content is slightly higher than the liquid limit. Collect into a compact mass and cover and leave overnight. Remix on mixing plate for 5 to 10 minutes. Place sufficient material in the cup of the liquid limit apparatus and level off with palette knife to a horizontal surface. Form a full depth groove in the material with a single stroke using the grooving tool. Keep removed material. Rotate the handle of the apparatus and count the number of blows till the groove comes in contact over a length of 13mm, which must be between 15 and 25 blows. Repeat until the difference between the number of blows for closure in two consecutive determinations is not greater than one. Record the number of blows for closure of the last determination. Take small portion on either side of the groove and the material removed in the grooving tool and place in closed container for weighing. Mix remaining specimen on mixing plate allowing water to be lost by evaporation. Test the drier sample in the apparatus as before but do not exceed 35 blows. Repeat steps until determinations of the number of blows have been made at four separate water contents. Ideally two will be with the number of blows lying between 15 and 25 and two between 25 and 35. Determine water content of specimens and plot on graph. X axis as number of blows (log scale) and Y axis as water content (%). Draw the best fit line. The liquid limit (LL) is the water content (but not reported as a percent when used as the liquid limit) where the line crosses 25 blows.
C.3 Plastic Limit

NZ 4402:1986 Standards, Part 2 Soil classification tests, Test 2.3 Determination of the plastic limit.

Take a specimen of at least 30g and place on mixing plate and mix thoroughly at natural water content or with the addition of sufficient distilled water in small increments to make the material workable. Allow drying to take place until the material becomes sufficiently plastic to be shaped into a ball without sticking to the fingers. Mould the soil between the fingers and roll until the heat of the hands has dried the soil sufficiently for slight cracks to appear on the surface. From this specimen take two sub samples of about 10g each and carry out separate determination for each. Divide each sub sample into 4 equal parts and form a thread about 6mm in diameter by rolling between the first finger and thumb of each hand, then roll the thread between the palm of the hand and the glass plate. Exert pressure to reduce the diameter to 3 to 5mm in 10 complete forward and back movements. Gather up and repeat until the thread shears longitudinally and transversely. Gather the thread portions and immediately transfer to a closed weighing canister. Treat the other three sub samples the same and place all four in the weighing canister. Then repeat with the other half of the sample. Determine water content of the samples and if exceeds 1.05 times the lower water content repeat the test with fresh sub samples. Report the plastic limit (PL) to the nearest whole number written without a percentage symbol.

C.4 Plasticity Index

NZ4402:1986 Standards, Part 2 Soil classification tests, Test 2.4 Determination of the plasticity index.

Report the plasticity index (PI) as the numerical difference between the liquid limit (LL) and the plastic limit (PL).
C.5 Linear Shrinkage

NZGS 4402:1986 Standards, Part 2 Soil classification tests, Test 2.6 Determination of the linear shrinkage.

Clean mould thoroughly and apply a thin film of silicone grease to the inner walls. Take a specimen of at least 150g from whole fine soil passing 425μm test sieve and mix thoroughly with distilled water until it becomes a smooth homogeneous paste with water content equal to or slightly greater than the liquid limit of the soil. Place soil and water mixture in mould so that it stands slightly proud of the side of the mould. Jar mould to remove air pockets and level surface off along the top of the mould. Place mould so that the mixture can air dry slowly away from draughts until shrinkage has largely ceased, then complete drying in a 105°C oven. Cool mould and soil and determine the mean length of the oven dried specimen and record.

Calculate the linear shrinkage of the soil as a percentage of the original length of the specimen from the formula:

\[ \text{Linear shrinkage} = 1 - \frac{\text{length of oven dry specimen}}{\text{internal length of mould}} \times 100 \]

Report linear shrinkage (LS) to the nearest whole number.

C.6 Solid Density of Soil Particles

NZ 4402:1986 Standards, Part 2 Soil classification tests, Test 2.7 Determination of the solid density of soil particles.

Take and dry 500g of fine soil and split into two equal portions taking care to obtain truly representative samples. Oven dry each portion and store in airtight container to cool. Dry and weigh a gas jar and ground glass plate to 0.2g and record \((M_1)\). Place sample into gas jar and weigh contents of gas jar, soil, and glass plate to 0.2g and record \((M_2)\). Add 500ml of room temperature water and insert rubber stopper. Mix end over end for 4h. Remove rubber stopper and add water to brim and place glass
plate on top. Carefully add water till filled, with no air bubbles. Dry outside of gas jar and glass plate. Weigh and record ($M_3$). Empty gas jar and refill with water only to the brim and place glass plate on top. Carefully add water till filled, with no air bubbles. Dry outside of gas jar and glass plate. Weigh and record ($M_4$).

Calculate the solid density of the soil particles from the formula:

$$Q_s = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \times Q_w$$

Where:

$M_1 =$ mass of gas jar, and ground glass plate (g)

$M_2 =$ mass of gas jar, plate and soil (g)

$M_3 =$ mass of gas jar, plate, soil, and water (g)

$M_4 =$ mass of gas jar, plate, and water (g)

$Q_w =$ density of the water used at the temperature of the test (t/m$^3$)

If the two results differ by more than 0.03 t/m$^3$, repeat the test.

**C.7 Particle Size Distribution – Wet Sieving**


Determination of the particle size distribution of the fractions greater than and equal to the sand fraction.

Wet sieving uses distilled water to disaggregate the representative sample. Appropriate sieve sizes are used and the material retained on each sieve is dried and weighed. Cumulative percentages are calculated and plotted on a graph.
C.8 Particle Size Distribution – Pipette Method


Determination of the silt and clay fraction of a soil sample.

Pipette analysis is based on calculations of the settling velocities for particles of different size (Stokes’ Law) and is used to analyse the size of silt and clay fractions of a soil. Sub samples of a specific volume are extracted from a suspension of mud at specified times and depths. The weight of each dried sub sample is representative of the proportion of the total mud fraction remaining in suspension above that specified depth at that specified time.

Obtain a sub sample that will yield 15-20g of mud. Fully disaggregate the sample and wet sieve with a 63μm (4Φ) sieve. Use as little distilled water as possible (less than 900ml). Dry the sand fraction retained on the sieve and weigh. Transfer the mud fraction and water to a measuring cylinder and add a dispersant. Top off water to 1000ml. Check for flocculation. As per schedule in (Lewis and McConchie, 1994) agitate and use pipette to collect 20ml samples representing the sizes +4 Φ to +10 Φ. Dry each pipetted sub sample and weigh. Cumulative percentages are calculated and plotted on a graph.

C.9 Direct Shear Test

British Standards BS 1377: Part 7: (1990)

Determination of peak friction and cohesion using the direct shear test.

To determine the peak friction or maximum shear strength of a soil under shearing stresses undisturbed samples must be used. The test involves creating a shear surface within the intact sample while applying a constant strain. During the test the sample is subjected to a load applied normal to the plane of shearing. Displacement readings with lapsed time and load cell readings are recorded and shear force vs displacement are plotted during the test, resulting in a curve from which the peak
strength and residual strength are measured. By carrying out three or more tests at
different normal pressures a shear strength envelope can be derived and the cohesion
of the material computed.

The normal pressure ($\sigma_n$) should represent the overburden in the sample. Subsequent
tests should use a $\sigma_n$ value that replicates any change in overburden.

The normal stress is calculated as:

$$\text{normal stress} (\sigma_n) = \frac{\text{mass of top plates + mass on hanger} \times 9.806}{\text{sample area} \times 100} \text{kPa}$$

Where:

- Mass of top plates = 1.287 kg
- Sample area is measured in $m^2$

The normal stress is applied to the sample and the sample is allowed to fully
consolidate. The rate of shear is calculated to allow full draining during shearing. For
this case the rate of shear was 0.024 mm/min. The computer records the lapsed
minutes (from which the amount of shearing displacement can be calculated), the
force in N, and plots the shear force vs displacement. The test continues until the
obvious peak shear force has passed. The peak shear force is then calculated:

$$\text{peak shear stress} (\tau) = \frac{P}{A} \text{kPa}$$

Where:

- $P$ = peak force (N) measured by the computer
- $A$ = surface area of sample in $m^2$

At least two more tests are done on fresh samples at different normal stresses. The
peak shear stress vs normal stress are plotted and a best fit line drawn as in:
\[ \tau = c + \sigma_n \tan \phi \]

Moisture content before and after test are calculated.

This test can also produce a residual shear stress, but the ring shear test is more appropriate.

**C.10 Ring Shear Test**

British Standards BS 1377: Part 7: (1990)

To determine the residual shear strength of a soil a remoulded sample at or lower than the plastic limit is loaded and levelled between platens of a ring. The Bromhead Ring Shear Apparatus measures the residual effective shear stress (\( \tau' \)) by creating a horizontal shear plane in an annular sample, rotating the bottom half of the sample in relation to the top, recording the resisting force, while applying a force normal to the plane of shearing. Displacement readings with lapsed time and load cell readings are recorded and shear force vs displacement are plotted during the test, resulting in a curve from which the residual strength is measured. By carrying out three or more tests at different normal pressures a shear strength envelope can be derived and the cohesion of the material resolved.

The normal pressure (\( \sigma_n \)) should represent the overburden in the sample. Subsequent tests should use a \( \sigma_n \) that replicates any change in overburden.

\[
\text{normal stress, } \sigma_n = \frac{\text{torque arm mass} + (\text{mass on hanger} \times 10) \times 9.806}{\text{sample area} \times 100} \text{ kPa}
\]

Where:

Torque arm mass = 1.155 kg

Sample area = 4.006 \times 10^{-3} \text{ m}^2
The normal stress is applied to the sample and the sample is allowed to fully consolidate. A shear surface is created in the sample by manually rotating the lower half 5 times over two minutes. The rate of shear for the test is set to 0.035616mm/min. The average residual force created by reaction on two load cells transferred from the sample to the torque arm on the top/stationary half of the shear plane is recorded along with the lapsed minutes (from which the amount of shearing displacement can be calculated), and plots the force vs displacement. The test continues until the plotted line has levelled out. The residual shear force is then calculated:

\[
\tau = \frac{3 \times P_r \times L}{2\pi (R_2' - R_1')}
\]

Where:

\( P_r \) = average residual force in N

\( L \) = distance between the load cell points (0.15 for outer position)

\( R_2 \) = outer radius of sample = 0.050m

\( R_3 \) = inner radius of sample = 0.035m

At least two more tests are done on fresh remoulded samples at different normal stresses. The residual shear stress vs normal stress are plotted and a best fit line drawn as in:

\[
\tau = c' + \sigma'_n \tan \phi'
\]

It is assumed that the cohesion is generally at or close to zero.

The effective residual friction angle (\( \phi'_r \)) is the slope of the graph.

Moisture content before and after test are calculated.
**C.11 Rock Direct Shear Test**


The portable direct shear box apparatus measures peak and residual direct shear strength of a rock sample at natural water content. Usually the tested shear plane aligns with a plane of weakness (bedding plane or joint).

The sample test horizon is selected and the specimen is clamped into the form so that the plane of weakness will be centred between the two halves of the form and the shear force direction is correctly aligned. The area over which the shearing will take place is measured and recorded. Casting material is placed around the specimen and allowed to harden. The form and sample are inverted over the opposite half of the form and this side is also cast and allowed to harden. The form and casting with aligned sample is placed in the shear box apparatus. A normal load is applied and consolidation allowed to take place. The shear force is applied at a displacement rate of 0.1mm/min. Shearing and recording of applied loads and displacement continue until the peak load is achieved. Shear force and displacement can be reversed to achieve the residual load.

Repeat the test ten times on fresh samples and plot normal stress ($\sigma_n$) vs shear stress ($\tau$).

$$\sigma_n = \frac{P_n}{A}$$

Where $A$ is the area and $P_n$ is the total normal force.

$$\tau = \frac{P_s}{A}$$

Where $A$ is the area and $P_s$ is the total shear force.

A best fit line drawn as in: $\tau = c + \sigma_n \tan \phi$
C.12 Schmidt Hammer

The Schmidt hammer is an instrument used for testing concrete and rock using impact energy which can be correlated to compressive strength and/or Schmidt hardness.

The test hammer is used by lightly pressing on the head of the impact plunger (1) until the plunger is released and slides out of the housing (3) by itself. The plunger is pressed against the concrete or rock material (2) which is to be tested. Just before it disappears completely into the housing the hammer is released. At the moment of impact the hammer must be at right angle to the surface. After impact the hammer mass (14) rebounds and recorded on the scale (19). The reading gives the rebound value (R_{\alpha}) in percent of the forward movement of the hammer mass. The hammer is calibrated for horizontal impact direction (vertical surfaces) and needs to corrected for vertical impact. (Reading of 10 R_{\alpha} add 3.2, 20 add 3.4, 30 add 3.1.)

The rebound number (R_{\alpha}) can be converted to a cube compressive strength (N/mm\(^2\)) using the graph in. Note: When the Schmidt hammer is used in the vertical downward position on a horizontal surface the line used on the graph is \(\alpha = -90^\circ\).
Appendix D – Test Results

Refer to Appendix C for Test Methods.

D.1 Water Content

<table>
<thead>
<tr>
<th>Date Collected</th>
<th>Sample ID</th>
<th>Sample Location</th>
<th>Sample Lithology</th>
<th>M1 (g)</th>
<th>M2 (g)</th>
<th>M3 (g)</th>
<th>Strat Reference</th>
<th>For Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>18-Oct-05</td>
<td>#6</td>
<td>David's Hole</td>
<td>clay</td>
<td>53.9</td>
<td>99.0</td>
<td>88.2</td>
<td>FSA 31</td>
<td>Soil Direct Shear</td>
</tr>
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<td>David's Hole</td>
<td>clay</td>
<td>55.3</td>
<td>181.1</td>
<td>150.7</td>
<td>FSA 32</td>
<td>Soil Direct Shear</td>
</tr>
<tr>
<td>21-Oct-05</td>
<td>#6</td>
<td>David's Hole</td>
<td>clay</td>
<td>53.9</td>
<td>129.9</td>
<td>112.9</td>
<td>FSA 29</td>
<td>Before Soil Direct Shear</td>
</tr>
<tr>
<td>21-Oct-05</td>
<td>#6</td>
<td>David's Hole</td>
<td>clay</td>
<td>53.9</td>
<td>185.1</td>
<td>154.4</td>
<td>FSA 28</td>
<td>After Soil Direct Shear</td>
</tr>
<tr>
<td>21-Oct-05</td>
<td>#7</td>
<td>David's Hole</td>
<td>clay</td>
<td>55.6</td>
<td>153.4</td>
<td>131.7</td>
<td>FSA 29</td>
<td>Before Soil Direct Shear</td>
</tr>
<tr>
<td>21-Oct-05</td>
<td>#7</td>
<td>David's Hole</td>
<td>clay</td>
<td>55.6</td>
<td>149.0</td>
<td>128.9</td>
<td>FSA 27</td>
<td>After Soil Direct Shear</td>
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<tr>
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<td>David's Hole</td>
<td>clay</td>
<td>54.4</td>
<td>135.0</td>
<td>114.3</td>
<td>FSA 35</td>
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<td>21-Oct-05</td>
<td>#8</td>
<td>David's Hole</td>
<td>clay</td>
<td>20.3</td>
<td>50.3</td>
<td>44.1</td>
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</tr>
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<td>David's Hole</td>
<td>clay</td>
<td>20.4</td>
<td>30.2</td>
<td>27.4</td>
<td>FSA 40</td>
<td>After Soil Ring Shear</td>
</tr>
<tr>
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<td>David's Hole</td>
<td>clay</td>
<td>20.3</td>
<td>35.1</td>
<td>31.8</td>
<td>FSA 29</td>
<td>After Soil Direct Shear</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20.3</td>
<td>36.9</td>
<td>31.8</td>
<td>FSA 44</td>
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<tr>
<td>15-Nov-05</td>
<td>Slab bottom</td>
<td>Bench trench</td>
<td>slab 200-300mm</td>
<td>54.1</td>
<td>225.9</td>
<td>201.1</td>
<td>1A 17</td>
<td>Before Rock Shear Box</td>
</tr>
<tr>
<td>15-Nov-05</td>
<td>Slab middle</td>
<td>Bench trench</td>
<td>slab 500-600mm</td>
<td>53.8</td>
<td>192.9</td>
<td>169.9</td>
<td>1B 24</td>
<td>Schmidt Hammer</td>
</tr>
<tr>
<td>15-Nov-05</td>
<td>Slab upper</td>
<td>Bench trench</td>
<td>slab 900-1100mm</td>
<td>27.9</td>
<td>80.1</td>
<td>73.4</td>
<td>1C 15</td>
<td>Schmidt Hammer</td>
</tr>
<tr>
<td>9-Nov-05</td>
<td>Drill Hole #11</td>
<td>David's Hole</td>
<td>slab</td>
<td>27.9</td>
<td>150.9</td>
<td>134.2</td>
<td>2A 16</td>
<td>Triaxial Testing</td>
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</table>

Water Content (w)
D.2 Liquid Limit

Form 2.2, 2.3, 2.4
DETERMINATION OF THE LIQUID AND PLASTIC LIMITS, PLASTICITY INDEX AND WATER CONTENT
(Tests 2.2, 2.3 and 2.4)

Job: Seale Thesis
Location: David’s Hole - Clay
Depth(s): Sample no.: 18 Oct 05 #6
Test details: Tested by: J. Seale

Test performed on fraction passing
Date: 18 Nov 05
425 μm sieve/whole soil
History: Natural/air-dried/oven-dried/
unknown Natural
Soil equilibrated with water for 24 h
Liquid limit machine no.

<table>
<thead>
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<th>Test no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
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<tr>
<td>Type of test t</td>
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<td>LL</td>
<td>LL</td>
<td>LL</td>
<td>PL</td>
<td>PL</td>
</tr>
<tr>
<td>No. of blows (liquid limit test)</td>
<td>18</td>
<td>16</td>
<td>18</td>
<td>18</td>
<td>20</td>
<td>18</td>
</tr>
<tr>
<td>Container no.</td>
<td>16</td>
<td>19</td>
<td>27</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass of container and wet soil $M_2 \text{ g}$</td>
<td>51</td>
<td>52</td>
<td>64</td>
<td>88</td>
<td>70</td>
<td>68</td>
</tr>
<tr>
<td>Mass of container and dried soil $M_1 \text{ g}$</td>
<td>37.1</td>
<td>40.8</td>
<td>40.7</td>
<td>37.5</td>
<td>30.4</td>
<td>28.6</td>
</tr>
<tr>
<td>Mass of container $M_1 \text{ g}$</td>
<td>30.9</td>
<td>33.3</td>
<td>33.5</td>
<td>31.4</td>
<td>28.1</td>
<td>28.8</td>
</tr>
<tr>
<td>Mass of water $M_2 - M_1 \text{ g}$</td>
<td>20.2</td>
<td>20.1</td>
<td>20.9</td>
<td>20.3</td>
<td>20.4</td>
<td>21.1</td>
</tr>
<tr>
<td>Mass of dried soil $M_3 - M_1 \text{ g}$</td>
<td>6.2</td>
<td>7.6</td>
<td>7.2</td>
<td>6.1</td>
<td>2.3</td>
<td>1.0</td>
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<tr>
<td>Water content $w = \frac{M_2 - M_1}{M_3 - M_1} \times 100$ %</td>
<td>10.7</td>
<td>13.2</td>
<td>12.6</td>
<td>11.1</td>
<td>7.7</td>
<td>5.7</td>
</tr>
</tbody>
</table>

| Water content | 57.9 | 56.8 | 57.1 | 54.9 | 29.9 | 31.5 |

Water content 31 %
Liquid limit 57%
Plastic limit 26 %
Plasticity Index 26 %

* Delete inappropriate words.
* Water content test to be marked w:
Liquid Limit, LL; and Plastic Limit, PL.
D.3 Plastic Limit

See D.2 Liquid Limit.

D.4 Plasticity Index

See D.2 Liquid Limit.

D.5 Linear Shrinkage

<table>
<thead>
<tr>
<th>Date Collected</th>
<th>Sample ID</th>
<th>Sample Location</th>
<th>Sample Lithology</th>
<th>length of oven dry sample</th>
<th>internal length of mould</th>
<th>LS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18-Oct-05</td>
<td>#6-1</td>
<td>David’s Hole</td>
<td>clay/FSA</td>
<td>134.61</td>
<td>150.48</td>
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<td>18-Oct-05</td>
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<td>David’s Hole</td>
<td>clay/FSA</td>
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<td>#6-3</td>
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<td>clay/FSA</td>
<td>135.47</td>
<td>150.56</td>
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</table>

D.6 Solid Density of Soil Particles

<table>
<thead>
<tr>
<th>Date Collected</th>
<th>Sample ID</th>
<th>Sample Location</th>
<th>Sample Lithology</th>
<th>M1 (g)</th>
<th>M2 (g)</th>
<th>M3 (g)</th>
<th>M4 (g)</th>
<th>Qw (t/m³)</th>
<th>Qs (t/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>21-Oct-05</td>
<td>#3-1</td>
<td>David’s Hole</td>
<td>Slab 1A</td>
<td>1358.9</td>
<td>1594.4</td>
<td>2767.8</td>
<td>2630.0</td>
<td>0.9982</td>
<td>2.4061</td>
</tr>
<tr>
<td>21-Oct-05</td>
<td>#3-2</td>
<td>David’s Hole</td>
<td>Slab 1A</td>
<td>1423.1</td>
<td>1654.7</td>
<td>2792.6</td>
<td>2657.5</td>
<td>0.9982</td>
<td>2.3957</td>
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</table>

D.7 Particle Size Distribution – Wet Sieving

<table>
<thead>
<tr>
<th>Sieve Size (μm)</th>
<th>Weight of Retained Sample (g)</th>
<th>Weight of Fraction (g)</th>
<th>Cumulative % Passing</th>
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</thead>
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<td>600</td>
<td>0.486</td>
<td>18.630</td>
<td>97</td>
</tr>
<tr>
<td>475</td>
<td>0.093</td>
<td>18.537</td>
<td>97</td>
</tr>
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<td>0.141</td>
<td>18.396</td>
<td>96</td>
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<td>212</td>
<td>0.154</td>
<td>18.242</td>
<td>95</td>
</tr>
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<td>90</td>
<td>0.647</td>
<td>17.595</td>
<td>92</td>
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<tr>
<td>63</td>
<td>0.195</td>
<td>17.400</td>
<td>91</td>
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</table>

Total Sand (g) 1.716
### Wet Sieve Analysis Clay Sample #7/2 FSA

<table>
<thead>
<tr>
<th>Sieve Size (μm)</th>
<th>Weight of Retained Sample (g)</th>
<th>Weight of Fraction (g)</th>
<th>Cumulative % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>0.24</td>
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<td>475</td>
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<td>98</td>
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<td>300</td>
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<td>18.166</td>
<td>97</td>
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<td>212</td>
<td>0.173</td>
<td>17.993</td>
<td>96</td>
</tr>
<tr>
<td>90</td>
<td>0.956</td>
<td>17.037</td>
<td>91</td>
</tr>
<tr>
<td>63</td>
<td>0.337</td>
<td>16.700</td>
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</table>

Total Sand (g) 2.011

### Wet Sieve Analysis Slab 1A

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<th>Sieve Size (μm)</th>
<th>Weight of Retained Sample (g)</th>
<th>Weight of Fraction (g)</th>
<th>Cumulative % Passing</th>
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</thead>
<tbody>
<tr>
<td>600</td>
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<td>20.777</td>
<td>92</td>
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<tr>
<td>475</td>
<td>0.367</td>
<td>20.410</td>
<td>91</td>
</tr>
<tr>
<td>300</td>
<td>0.617</td>
<td>19.793</td>
<td>88</td>
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<tr>
<td>212</td>
<td>0.658</td>
<td>19.135</td>
<td>85</td>
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<tr>
<td>90</td>
<td>2.092</td>
<td>17.043</td>
<td>76</td>
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<tr>
<td>63</td>
<td>0.592</td>
<td>16.451</td>
<td>73</td>
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</tbody>
</table>

Total Sand (g) 6.056

### Wet Sieve Analysis Slab 1B

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<th>Weight of Retained Sample (g)</th>
<th>Weight of Fraction (g)</th>
<th>Cumulative % Passing</th>
</tr>
</thead>
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<tr>
<td>475</td>
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<td>93</td>
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<td>0.706</td>
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<td>90</td>
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<td>63</td>
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<td>16.200</td>
<td>74</td>
</tr>
</tbody>
</table>

Total Sand (g) 5.631
## D.8 Particle Size Distribution – Pipette Method

### Data Sheet for Pipette Analysis Clay Sample #7/1 FSA

<table>
<thead>
<tr>
<th>Diameter Φ</th>
<th>Temperature</th>
<th>Withdrawal Depth (cm)</th>
<th>Time</th>
<th>Beaker #</th>
<th>WL Sample + Beaker (g)</th>
<th>WL Beaker (g)</th>
<th>Weight Sample (g) x 50</th>
<th>Cumulative % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>+4.0</td>
<td>19</td>
<td>20.0</td>
<td>20 sec</td>
<td>A</td>
<td>28.021</td>
<td>27.653</td>
<td>0.368</td>
<td>17.400</td>
</tr>
<tr>
<td>+4.5</td>
<td>19</td>
<td>20.2</td>
<td>2 min</td>
<td>B</td>
<td>29.638</td>
<td>29.268</td>
<td>0.368</td>
<td>17.400</td>
</tr>
<tr>
<td>+5.0</td>
<td>19</td>
<td>20.1</td>
<td>4 min</td>
<td>C</td>
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<td>27.479</td>
<td>0.359</td>
<td>16.950</td>
</tr>
<tr>
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<td>19</td>
<td>20.2</td>
<td>8 min</td>
<td>D</td>
<td>28.813</td>
<td>28.472</td>
<td>0.341</td>
<td>16.000</td>
</tr>
<tr>
<td>+6.0</td>
<td>19</td>
<td>20.2</td>
<td>15 min</td>
<td>E</td>
<td>27.844</td>
<td>27.528</td>
<td>0.316</td>
<td>14.800</td>
</tr>
<tr>
<td>+7.0</td>
<td>19</td>
<td>10.0</td>
<td>30 min</td>
<td>F</td>
<td>28.881</td>
<td>28.646</td>
<td>0.235</td>
<td>10.750</td>
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<td>10.0</td>
<td>2 hr</td>
<td>G</td>
<td>27.753</td>
<td>27.565</td>
<td>0.168</td>
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</tr>
<tr>
<td>+9.0</td>
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<td>10.0</td>
<td>8 hr</td>
<td>H</td>
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<td>27.165</td>
<td>0.129</td>
<td>5.400</td>
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<tr>
<td>+10.0</td>
<td>19</td>
<td>10.0</td>
<td>32 hr</td>
<td>I</td>
<td>no data</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Peptizer - Calgon 1g/1000ml

### Data Sheet for Pipette Analysis Clay Sample #7/2 FSA

<table>
<thead>
<tr>
<th>Diameter Φ</th>
<th>Temperature</th>
<th>Withdrawal Depth (cm)</th>
<th>Time</th>
<th>Beaker #</th>
<th>WL Sample + Beaker (g)</th>
<th>WL Beaker (g)</th>
<th>Weight Sample (g) x 50</th>
<th>Cumulative % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>+4.0</td>
<td>20</td>
<td>20.0</td>
<td>20 sec</td>
<td>A</td>
<td>28.941</td>
<td>28.587</td>
<td>0.354</td>
<td>16.700</td>
</tr>
<tr>
<td>+4.5</td>
<td>20</td>
<td>20.7</td>
<td>2 min</td>
<td>B</td>
<td>29.747</td>
<td>29.394</td>
<td>0.353</td>
<td>16.650</td>
</tr>
<tr>
<td>+5.0</td>
<td>20</td>
<td>20.6</td>
<td>4 min</td>
<td>C</td>
<td>27.653</td>
<td>27.310</td>
<td>0.343</td>
<td>16.150</td>
</tr>
<tr>
<td>+5.5</td>
<td>20</td>
<td>20.7</td>
<td>8 min</td>
<td>D</td>
<td>27.903</td>
<td>27.572</td>
<td>0.331</td>
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<tr>
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<td>20.6</td>
<td>15 min</td>
<td>E</td>
<td>27.602</td>
<td>27.291</td>
<td>0.311</td>
<td>14.550</td>
</tr>
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<td>10.3</td>
<td>30 min</td>
<td>F</td>
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<td>27.403</td>
<td>0.251</td>
<td>11.550</td>
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<td>2 hr</td>
<td>G</td>
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<td>28.112</td>
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</table>

Peptizer - Calgon 1g/1000ml

### Data Sheet for Pipette Analysis Bottom of Slab 200-300mm Slab 1A

<table>
<thead>
<tr>
<th>Diameter Φ</th>
<th>Temperature</th>
<th>Withdrawal Depth (cm)</th>
<th>Time</th>
<th>Beaker #</th>
<th>WL Sample + Beaker (g)</th>
<th>WL Beaker (g)</th>
<th>Weight Sample (g) x 50</th>
<th>Cumulative % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>+4.0</td>
<td>20</td>
<td>20.0</td>
<td>20 sec</td>
<td>A</td>
<td>28.002</td>
<td>27.653</td>
<td>0.349</td>
<td>16.450</td>
</tr>
<tr>
<td>+4.5</td>
<td>20</td>
<td>20.7</td>
<td>2 min</td>
<td>B</td>
<td>29.610</td>
<td>29.268</td>
<td>0.342</td>
<td>16.100</td>
</tr>
<tr>
<td>+5.0</td>
<td>20</td>
<td>20.6</td>
<td>4 min</td>
<td>C</td>
<td>27.789</td>
<td>27.479</td>
<td>0.309</td>
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<td>20.7</td>
<td>8 min</td>
<td>D</td>
<td>28.181</td>
<td>28.472</td>
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<td>20.6</td>
<td>15 min</td>
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<td>27.294</td>
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<td>10.3</td>
<td>30 min</td>
<td>F</td>
<td>28.795</td>
<td>28.646</td>
<td>0.149</td>
<td>6.400</td>
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<td>2 hr</td>
<td>G</td>
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<td>0.107</td>
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<td>H</td>
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<td>27.165</td>
<td>0.082</td>
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Peptizer - Calgon 1g/1000ml
### Data Sheet for Pipette Analysis Middle of Slab 500-600 mm Slab 1B

<table>
<thead>
<tr>
<th>Diameter $\phi$</th>
<th>Temperature</th>
<th>Withdrawal Depth (cm)</th>
<th>Time</th>
<th>Beaker #</th>
<th>Wt. Sample + Beaker (g)</th>
<th>Wt. Beaker (g)</th>
<th>Weight Sample (g) x 50</th>
<th>Cumulative % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>+4.0</td>
<td></td>
<td></td>
<td>20 sec</td>
<td>A</td>
<td>28.931</td>
<td>28.587</td>
<td>0.344</td>
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<td>+4.5</td>
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<td>2 min</td>
<td>B</td>
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<td>29.394</td>
<td>0.322</td>
<td>15.100</td>
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<td>20</td>
<td>20.6</td>
<td>4 min</td>
<td>C</td>
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<td>27.310</td>
<td>0.261</td>
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<td>15 min</td>
<td>E</td>
<td>27.684</td>
<td>27.528</td>
<td>0.156</td>
<td>6.800</td>
</tr>
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<td>20</td>
<td>10.3</td>
<td>30 min</td>
<td>F</td>
<td>27.517</td>
<td>27.403</td>
<td>0.114</td>
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<td>10.3</td>
<td>2 hr</td>
<td>G</td>
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<td>28.112</td>
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<td></td>
<td>32 hr</td>
<td>I</td>
<td></td>
<td></td>
<td></td>
<td>no data</td>
</tr>
</tbody>
</table>

Plotted results: Red, #7/1 FSA; Pink, #7/2 FSA; Blue, Bottom of Slab/Slab 1A; Green, Middle of Slab/Slab 1B.
### D.9 Direct Shear Test

**Data Sheet for Direct Shear Results for FSA**

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Water Content before Test (%)</th>
<th>Water Content after Test (%)</th>
<th>Peak Friction Angle $\phi$</th>
<th>Mass on Hanger</th>
<th>$\sigma_n$ (kPa)</th>
<th>$\tau$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#9</td>
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<td>38.9</td>
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<td>31.80</td>
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<td></td>
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<td>38</td>
<td>37.26</td>
<td>21.08</td>
</tr>
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<td></td>
<td></td>
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<td></td>
<td>44</td>
<td>43.15</td>
<td>26.52</td>
</tr>
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<td>22.62</td>
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<td>72.66</td>
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<td></td>
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</tr>
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<td></td>
<td></td>
<td></td>
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<td>99.32</td>
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<tr>
<td></td>
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<td></td>
<td></td>
<td>*20</td>
<td>197.38</td>
<td>95.10</td>
</tr>
</tbody>
</table>

*On different machine with lever arm.

**Direct Shear Analysis Showing Friction Angle of 26 degrees for Failure Surface A**

- Sample #9
- Sample #7
- Sample #8
- Sample #6
- Linear (Combined)

\[
y = 0.366x + 24.79
\]

\[
y = 0.418x + 2.6511
\]

\[
y = 0.8068x - 8.5184
\]

\[
y = 0.478x + 5.2295
\]

\[
y = 0.5325x + 2.4118
\]

\[
y = 0.478x + 5.2295
\]

\[
R^2 = 0.9138
\]

\[
\phi = 25.56^\circ
\]

\[
c = 5.2 \text{ kPa}
\]
### D.10 Ring Shear Test

**Data Sheet for Ring Shear Results for FSA**

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Water Content before Test (%)</th>
<th>Water Content after Test (%)</th>
<th>Residual Friction Angle φ</th>
<th>Mass on Hanger</th>
<th>$\sigma_{n}$ (kPa)</th>
<th>$P_r$ (N)</th>
<th>$\tau_r$ (Pa)</th>
<th>$\tau_r$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#7</td>
<td>29</td>
<td>40</td>
<td>3.09</td>
<td>1</td>
<td>27.305</td>
<td>2.564296</td>
<td>2236.274</td>
<td>2.236274</td>
</tr>
<tr>
<td>#8</td>
<td>29</td>
<td>44</td>
<td>2.64</td>
<td>3</td>
<td>76.262</td>
<td>5.203061</td>
<td>4537.492</td>
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<tr>
<td></td>
<td></td>
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<td></td>
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<td>247.610</td>
<td>12.712240</td>
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<td>11.086105</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td>*3</td>
<td>76.262</td>
<td>4.652041</td>
<td>4056.957</td>
<td>4.056957</td>
</tr>
</tbody>
</table>

* Sample is unloaded to first stage
Appendix D

Ring Shear Analysis Showing Residual Friction Angle of ~3° For Failure Surface A

- Sample #7
- Sample #8
- Linear (Combined)

\[ y = 0.0473x \]
\[ R^2 = 0.9476 \]
\[ \phi_r = 2.71° \]
\[ c = 0 \text{kPa} \]

Mohr-Coulomb Failure:
- \( c = 0 \text{kPa} \)
- \( \phi = 2.71° \)
- Tensile strength = 0 kPa
- Uniaxial compressive strength = 0 kPa
- \( \alpha = 37.25° \)

Analysis of DIRECT SHEAR Lab Data:
- No. of lab data points = 6
- Sum square of errors (Root Mean Square) = 3.208
- Current strength model is NOT a best fit.
### D.11 Rock Shear Test

#### Data Sheet for Rock Shear Box Results for Slab 1A

<table>
<thead>
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<th>Time (min)</th>
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Sample Area (m²) 0.00549

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Sample Area (m²) 0.005882

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#### Rock Shear Box Shear Stress vs Shear Displacement for Slab 1A

![Graph showing shear stress vs shear displacement for Slab 1A](image-url)
Appendix D

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Rock Shear Box Analysis of Slab 1A Showing Friction Angle of 1.3 degrees

\[ y = 0.0242x + 123.67 \]

Mohr-Coulomb Criteria:
- cohesion = 123.67 kPa
- friction angle = 1.39 degrees
- uniaxial compressive strength = 262.8 kPa
- \( \alpha = 36.1 \) degrees

Analysis of DIRECT SHEAR Lab Data
- Nl of tab data points = 2
- Sum squares of errors (RMSLUL) = 3
- Current strength model is LEVENEED

Rock Shear Box Analysis of Footwall
## D.12 Schmidt Hammer

Data collected on 26 Nov 05 in Bench Trench

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### D.13 Bulk Density and Unit Weight Data

**Data for Unit Weight and Bulk Density of Laminated Mudstones/Siltstones**

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# Appendix E – XRD Analysis Results

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Data Sheet for XRD Results