ENGINEERING GEOLOGY ASSESSMENT OF THE SLOVENS-AVOCA RAIL CORRIDOR AND SLOVENS CREEK VIADUCT, MIDLAND LINE, CANTERBURY

A thesis submitted in partial fulfilment of the requirements for the Degree of Master of Science in Engineering Geology at the University of Canterbury by Julia M. Watson

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Abstract

This project was initiated by ENGEIO Limited and KiwiRail Holdings Limited to assess the stability of Slovens Creek Viaduct (specifically its western abutment) and a 3km section of rail corridor between Slovens Creek Viaduct and Avoca on the Midland Line (MDL). Commonly known as the scenic TranzAlpine rail journey (through Arthurs Pass National Park) the MDL connects Greymouth to Christchurch via Rolleston, where the MDL meets the Main South Line into Christchurch. The project area is approximately 40km southeast of Arthurs Pass Township, in the eastern extension of the Castle Hill Basin which is part of the Waimakariri Catchment and Canterbury Foothills.

The field area is underlain by Rakaia Terrane, which is part of the Torlesse Composite Terrane forming the basement rock unit for the field area. Cretaceous-Tertiary rocks of the Castle Hill Basin overlie the basement strata and record a transgression-regression sequence, as well as mid-Oligocene submarine volcanism. The stratigraphic sequence in the Castle Hill Basin, and its eastern extension to Avoca, comprises two formations of the Eyre group, the older Broken River Formation and the younger Iron Creek Formation. Deep marine Porter Group limestones, marls, and tuffs of Oligocene age succeed the Iron Creek Formation of the Eyre Group, and probably records the maximum of the transgression. The Enys Formation lies disconformably on the Porter Group and is overlain unconformably by Late Pleistocene glacifluvial and glacial deposits. The Tertiary strata in the Slovens-Avoca rail corridor are weak, and the clay-rich tuff derived from mid-Oligocene volcanism is particularly prone to slaking.

Extensive mapping carried out for this project has identified that some 90 percent of the surface along the length of the Slovens-Avoca corridor has been subject to mass movement. The landslides of the Slovens-Avoca rail corridor are clearly younger than the Last Glaciation, and Slovens Creek has been downcutting, with associated faulting and uplift, to form the present day geomorphology of the rail corridor. Deep-seated landslides in the rail corridor extend to Slovens Creek, locally deflecting the stream course, and a generic ground failure model for the rail corridor has been developed.

Exploratory geotechnical investigations, including core drilling, installation of an inclinometer and a piezometer, enabled the construction of a simple ground model and cross section for the Slovens Creek Viaduct western abutment. Limit-equilibrium and pseudo-static slope stability analyses using both circular and block critical slip surface search methods were applied to the ground model for the western abutment of Slovens Creek Viaduct. Piezometric and strength data obtained during laboratory testing of core material have been used to constrain the western abutment stability assessment for one representative section line (C-C’). Prior to pseudo-static sensitivity analyses peak ground acceleration (PGA) for various Ultimate Limit State (ULS) design return periods, defined by an equation given in NZS1170.5:2004, were calculated and have been used as a calibration technique to find and compare specific PGA values for pseudo-static analyses in the Slovens
Creek Viaduct area. The main purpose has been to provide an indication of how railway infrastructure could be affected by seismic events of various return periods defined by ULS design standards for the area.

Limit equilibrium circular slip surface search methods, both grid search and auto refine search, indicated the slope is stable with a FoS greater than 1.0 returned from each, although one particular surface returned the lowest FoS in each. This surface is in the lower portion of the slope, adjacent to Slovens Stream and northeast of the MDL. As expected, pseudo-static analyses returned a lower FoS overall when compared to limit equilibrium analyses. The PGA analyses suggest that partial ground failure at the Slovens Creek Viaduct western abutment could occur in a 1 in 25-year return period event within materials on the slower slope beyond the immediate rail corridor. A ULS (1 in 500-year) event in the Slovens Creek Viaduct area would likely produce a PGA of ~0.9g, and the effects on the western abutment and rail infrastructure would most likely be catastrophic.

Observed ground conditions for the western abutment of the Slovens Creek Viaduct suggest there is no movement within the landslide at depth within the monitoring timeframe of this project (22 May 2015 – 4 August 2015). Slope stability monitoring is recommended to be continued in two parts: (1) the inclinometer in BH1 is to be monitored on a six monthly basis for one year following completion of this thesis, and then annually unless ground movements become evident; and (2) surface movement monitoring should be installed using a fixed datum on the stable eastern abutment. Long-term stability management strategies for the Slovens Creek Viaduct western abutment are dependent upon future observed changes and ongoing monitoring. Hazard and risk assessment using the KiwiRail Qualitative Risk Assessment Framework (QRA) is recommended, and if slope stability becomes problematic for operation of the Midland Line consideration should be given to deep slope drainage. In the event of a large magnitude or high PGA earthquake all monitoring should be reviewed.
# Table of Contents

Acknowledgements ........................................................................................................ ii

Abstract ......................................................................................................................... iii

List of Figures ................................................................................................................ ix

List of Tables .................................................................................................................. xii

1 Introduction .................................................................................................................. 1

1.1 Project Background ............................................................................................... 1

1.2 The Midland Line ................................................................................................. 1

1.2.1 Midland Line Construction History ................................................................. 1

1.2.2 Avoca ............................................................................................................... 4

1.2.3 Slovens Creek Viaduct ....................................................................................... 4

1.3 Thesis Aims and Objectives .................................................................................. 5

1.4 Geological and Geomorphological Setting ......................................................... 6

1.5 Landslide Terminology ......................................................................................... 9

1.6 Track Terminology ............................................................................................... 12

1.6.1 Metrage ............................................................................................................. 12

1.6.2 Track Logs and Track Faults ............................................................................. 12

1.7 Thesis Format ........................................................................................................ 12

2 Thesis Methodology ................................................................................................. 14

2.1 Introduction ........................................................................................................... 14

2.2 Aerial Survey ........................................................................................................ 14

2.3 Fieldwork and Mapping ....................................................................................... 15

2.4 Boreholes, Inclinometer, and Piezometer ............................................................ 18

2.5 Laboratory Analysis ............................................................................................. 20

2.5.1 Uniaxial Compressive Strength ....................................................................... 20

2.5.2 Point Load Test .................................................................................................. 20

2.5.3 Direct Shear (Rock) .......................................................................................... 20

2.5.4 Direct Shear (Soil) ............................................................................................ 20
2.6 Landslide modelling ........................................................................................................................ 23

3 Physical Setting ................................................................................................................................... 24

3.1 Introduction ..................................................................................................................................... 24

3.2 Climate ............................................................................................................................................ 24

3.3 Cretaceous-Tertiary Rocks of the Castle Hill Basin ........................................................................ 25

3.3.1 Rakaia Terrane ............................................................................................................................ 26

3.3.2 Eyre Group ................................................................................................................................ 26

3.3.3 Porter Group ............................................................................................................................... 26

3.3.4 Motunau Group .......................................................................................................................... 29

3.3.5 Previous Formation Names ..................................................................................................... 29

3.4 Geological Structure of the Castle Hill Basin .................................................................................. 31

3.5 Quaternary Geology and Geomorphology of the Castle Hill Basin ................................................ 33

3.5.1 Gage (1958) ............................................................................................................................. 33

3.5.2 Rother et al. (2015) .................................................................................................................. 35

3.6 Geology of the Rail Corridor ........................................................................................................... 37

3.6.1 Structure in the rail corridor .................................................................................................... 37

3.6.2 Lithology of the Rail Corridor ................................................................................................. 38

3.7 Quaternary Geology and Geomorphology of the Rail Corridor ...................................................... 45

3.7.1 Geomorphology ....................................................................................................................... 45

3.7.2 Landslides ................................................................................................................................ 47

3.7.3 Engineered and Non-Engineered Fill of the Rail Corridor ...................................................... 47

3.8 Summary .......................................................................................................................................... 47

4 Ground Model of Slovens-Avoca Rail Corridor ..................................................................................... 49

4.1 Introduction ..................................................................................................................................... 49

4.2 Engineering Geology Descriptions ................................................................................................. 49

4.2.1 Rakaia Terrane Rocks – Torlesse Supergroup ........................................................................ 50

4.2.2 Puffer Formation ...................................................................................................................... 50

4.2.3 Thomas Formation ................................................................................................................... 50

4.2.4 Enys Formation ....................................................................................................................... 51

4.2.5 Quaternary Glacifluvial Deposits ............................................................................................ 51
4.3 Map 1 ............................................................................................................................................... 52
  4.3.1 Map 1A - Geology ................................................................................................................... 52
  4.3.2 Map 1B – Engineering Geomorphology .................................................................................. 52
4.4 Map 2 ............................................................................................................................................... 54
  4.4.1 Map 2A – Geology .................................................................................................................. 54
  4.4.2 Engineering Geomorphology - Map 2B .................................................................................. 54
4.5 Map 3 ............................................................................................................................................... 61
  4.5.1 Map 3A - Geology ................................................................................................................... 61
  4.5.2 Map 3B - Engineering Geomorphology .................................................................................. 61
4.6 Map 4 ............................................................................................................................................... 67
  4.6.1 Map 4A – Geology .................................................................................................................. 67
  4.6.2 Map 4B – Engineering Geomorphology .................................................................................. 67
4.7 Map 5 ............................................................................................................................................... 70
  4.7.1 Map 5A – Geology .................................................................................................................. 70
  4.7.2 Map 5B – Engineering Geomorphology .................................................................................. 72
4.8 Ground Failure Model ..................................................................................................................... 75
4.9 Summary .......................................................................................................................................... 77
5 Ground Model for Slovens Creek Viaduct Western Abutment .............................................................. 79
  5.1 Introduction ..................................................................................................................................... 79
  5.2 Map 6 – Slovens Creek Viaduct Area ............................................................................................. 80
    5.2.1 Map 6A - Geology ................................................................................................................... 80
    5.2.2 Map 6B - Engineering geomorphology ................................................................................... 82
  5.3 Subsurface Geology ......................................................................................................................... 87
  5.4 Engineering Geology Model ........................................................................................................... 91
  5.5 Limit Equilibrium Analysis ............................................................................................................. 93
    5.5.1 Background .............................................................................................................................. 93
    5.5.2 Input parameters ...................................................................................................................... 93
    5.5.3 Assumptions and limitations ................................................................................................... 96
    5.5.4 Output/Results ....................................................................................................................... 97
  5.6 Pseudo-static Analysis .................................................................................................................... 100
List of Figures

Figure 1-1 The Midland Line (MDL) ................................................................................................................ 2
Figure 1-2 Historic photographs of the Midland Line ...................................................................................... 3
Figure 1-3 Historic photographs of Avoca ........................................................................................................ 3
Figure 1-4 Historic photograph of Slovens Creek Viaduct nearing completion in 1909.. .............................. 5
Figure 1-5 Figure to illustrate the Terrane accreted across New Zealand ......................................................... 7
Figure 1-6 Map adopted from Gage (1970) which details the Castle Hill Basin triangle. .............................. 8
Figure 1-7 Illustration of landslide terms described in the text ....................................................................... 11
Figure 2-1. 3D rail corridor ground model created in Surfer 11 ................................................................. 16
Figure 2-2 Orthophotography overlain the 3D ground model of the rail corridor. ........................................ 16
Figure 2-3 A reference map for Maps 1 to 6 created for the rail corridor.. .................................................. 17
Figure 2-4 Photographs of the drilling rig set up on the western abument of Slovens Creek Viaduct........... 19
Figure 2-5 Photographs of uniaxial compressive strength testing ................................................................... 21
Figure 2-6 Photographs of point load testing ................................................................................................. 22
Figure 2-7 Photographs of shear strength testing .......................................................................................... 22
Figure 3-1 Illustration of annual precipitation in the central South Island.. ................................................... 25
Figure 3-2 Site location map .......................................................................................................................... 27
Figure 3-3 Stratigraphic column taken and adjusted from Bradshaw (1975). .................................................... 28
Figure 3-4 Major faults and ranges of the Castle Hill Basin ........................................................................... 32
Figure 3-5 Representation of the angular unconformity in the Castle Hill Basin. ......................................... 32
Figure 3-6 Glaciations of the Waimakariri Catchment as mapped by Gage (1958). ....................................... 34
Figure 3-7 Extent of the LGM, as modelled by Rother et al., 2015.. .............................................................. 36
Figure 3-8 Faults within, and surrounding, the rail corridor.. ........................................................................ 37
Figure 3-9 Photographs of Rakaia Terrane ..................................................................................................... 39
Figure 3-10 Photographs of Thomas Formation Tuff, and Puffer Formation at the western abutment of Slovens Creek Viaduct ........................................................................................................... 40
Figure 3-11 Thin section images of the Puffer Formation .................................................................................. 42
Figure 3-12 A, Thomas Formation Tuff photographs ....................................................................................... 43
Figure 3-13 Thin section images of Thomas Formation Tuff ........................................................................ 44
Figure 3-14 Photographs of the Enys Formation in Bryce Gully ......................................................... 45
Figure 3-15 Photograph to show the growth of vegetation below the postulated contact of the glacifluvial outwash terrace ........................................................................................................... 46
Figure 4-1 Three-dimensional photography of Map 1 .................................................................... 53
Figure 4-2 Shaded relief model of Map 1 ........................................................................................ 53
Figure 4-3 Aerial photograph of the engineered drainage structure referred to in the text .............. 55
Figure 4-4 Aerial orthophotography of the area mapped for Map 2 .................................................. 55
Figure 4-5 Hill shade model to illustrate landslide Area 2 and landslide Area 3, as referred to in text ... 56
Figure 4-6 Cross section A-A’ of the Slovens-Avoca corridor showing an interpreted failure surface for the multiple rotational landslide of Area 2 .............................................................................. 57
Figure 4-7 Aerial orthophotography of the Map 3 area .................................................................. 60
Figure 4-8 Outline of landslide areas on Map 3 referred to in text .................................................. 60
Figure 4-9 Landslide Area 4 extracted from Map 3B ................................................................. 62
Figure 4-10 Landslide Area 5 extracted from Map 3B ................................................................. 63
Figure 4-11 Orthophotography showing the area of Map 4 and part of Map 3 .................................. 65
Figure 4-12 Illustration of numbered landslide areas referred to in text ....................................... 65
Figure 4-13 Cross section B-B’ ..................................................................................................... 66
Figure 4-14 Landslide Area 6 extracted from Map 4B ................................................................. 68
Figure 4-15 Photographs of the timber and steel retaining structure in Landslide Area 6 .......... 69
Figure 4-16 Landslide Area 7 extracted from Map 4B ................................................................. 70
Figure 4-17 Orthophotography for the areas of Map 5 and Map 6 ................................................. 71
Figure 4-18 Outline of numbered landslide areas for Map 5 and Map 6, as referred to in text ........ 71
Figure 4-19 Photographs of the landslides of Landslide Area 8 ..................................................... 73
Figure 4-20 Landslide Area 9 extracted from Map 5B ................................................................. 74
Figure 4-21 Schematic diagram to illustrate slope failure development in the Slovens-Avoca rail corridor ................................................................. 76
Figure 5-1 Image from Map 6A to illustrate the location of BH1 (inclinometer), BH2 (piezometer), culverts, Slovens Creek Viaduct, Craigieburn Road, and the unnamed road beneath Slovens Creek Viaduct ................................................................. 80
Figure 5-2 Photographs of the main large landslide and subsequent outcrop of Puffer formation P4 at the western abutment of Slovens Creek Viaduct .................................................................................. 81
Figure 5-3 Aerial photo to indicate the spatial distribution of drilling fluid seepage ................. 83
Figure 5-4 Photographs of drilling fluid seepage ........................................................................... 84
Figure 5-5 Piezometer recovery rate following ‘blowout’ experiment as described in the text................. 84
Figure 5-6 Landslide Area 10 extracted from Map 6. ................................................................. 85
Figure 5-7 Summary log of BH1................................................................................................ 88
Figure 5-8 Photographs of core from BH1.................................................................................. 89
Figure 5-9 Detailed log of core box 12 at depth 25.8m to 27.9m, and including the clay failure surface. .... 90
Figure 5-10 Cross section C-C’ of the Slovens Creek Viaduct western abutment showing an interpreted failure surface for the large landslide (Ls6) of Area 10. ................................................................. 92
Figure 5-11 Illustration of the method of calculation for the Hu coefficient for inclined groundwater surfaces as performed automatically by SLIDE. .................................................................................. 96
Figure 5-12 Limit-equilibrium models. ......................................................................................... 98
Figure 5-13 Hazard Factor Map for the South Island. Figure taken from NZS 1170.5:2004. ............... 102
Figure 5-14 Pseudo-static models.................................................................................................. 104
Figure 5-15 Example of inclinometer data from Stark and Choi (2008)............................................. 108
Figure 5-16 Data from Slovens Creek Viaduct inclinometer............................................................ 109
Figure 5-17 Qualitative Risk Assessment Framework for rail corridor management as provided by KiwiRail. ................................................................................................................................. 112
List of Tables

Table 1-1 Terms for type of movement when applied to the type of material.............................................................................................................. 10
Table 1-2 Summary of terms for forming complete names of landslides.................................................................................................................... 10
Table 1-3 Summary of thesis chapters and contents ........................................................................................................................................... 13
Table 3-1 Summary of formation name changes and the terms used in this thesis............................................................................................ 30
Table 3-2 Summary of similarities between the Brechin Formation and Enys Formation, at Brechin Burn and Castle Hill Basin, respectively................................................................................................................................. 30
Table 3-3 Summary of similarities between the Esk Formation and Puffer Formation, at Brechin Burn and Castle Hill Basin, respectively................................................................................................................................. 30
Table 3-4 Summary of glacial advances as proposed by Gage (left) compared to cosmogenic dates determined by Rother et al. (2015)................................................................................................................................................. 35
Table 5-1 Input parameters for limit equilibrium model as determined by laboratory and field work. .............................................................. 94
Table 5-2 Summary of results for FoS and lowest FoS from sensitivity analysis................................................................................................. 99
Table 5-3 Values of C(T) calculated with the above equation from NZS 1170.5:2004.................................................................................. 102
Table 5-4 Summary of limit equilibrium and pseudo static analysis results................................................................................................. 105
1 Introduction

1.1 Project Background

This project was initiated by ENGEIO Limited and KiwiRail Holdings Limited to assess the stability of Slovens Creek Viaduct (specifically the western abutment) and a 3km section of rail corridor between Slovens Creek Viaduct and Avoca on the Midland Line (MDL). Commonly known as the scenic TranzAlpine rail journey (through Arthurs Pass National Park) the MDL connects Greymouth to Christchurch via Rolleston, where the MDL meets the Main South Line into Christchurch (Figure 1-1). The project area lies in the Castle Hill Basin of the Waimakariri Catchment and Canterbury Foothills, which is approximately 40km southeast of Arthurs Pass Township (Figure 1-1).

Rail infrastructure has been adversely affected by the instability of soft Tertiary strata in the Slovens – Avoca corridor since construction, particularly at Slovens Creek Viaduct. Therefore, a geotechnical approach is employed in this project to highlight particular areas of instability.

Detailed engineering geomorphology and geological mapping was undertaken during primary stages of the project, and followed by deep geotechnical investigations in the western abutment of Slovens Creek Viaduct within a mapped landslide. Ground models of both the viaduct and rail corridor have been created using cross sections and three dimensional imagery, in addition to the engineering geomorphology and geological maps. Slope stability analyses and computer-generated models are limited to the western abutment of Slovens Creek Viaduct, where deep geotechnical investigation indicated a failure surface and provided core for use in laboratory testing.

The history of the Midland Line, Avoca and Slovens Creek Viaduct are covered in the following sections of this chapter, as are the thesis aims and objectives.

1.2 The Midland Line

1.2.1 Midland Line Construction History

Construction of the Midland Line commenced in 1890, and in the same year a small portion opened for traffic near Stillwater on the West Coast. The MDL was complete by 1900 west of the Main Divide (between Greymouth and Otira), however railway construction between Springfield and Arthurs Pass (east of the Main Divide) was hindered by difficult terrain and subsequent requirements of extensive infrastructure, such as four viaducts and 16 tunnels. The length of rail from Springfield was complete to the third viaduct at Broken River in early November 1906; 64km between Broken River to Otira remained to be constructed at that time (Figure 1-1; Figure 1-2 A; Wright & Wright 2009).
In 1908, drill and blast excavation of the 8.5 km Otira tunnel commenced from both the Arthurs Pass portal (east) and the Otira portal (west) (Figure 1-1; Wright & Wright, 2009). Unexpected difficulties such as extremely weathered rock and loose ‘shale’ slowed down the tunnelling project, which was initially signed and contracted for completion in five years. Whilst the Otira tunnel was in progress, the Slovens Creek Viaduct reached completion in 1909, and the railway line east of the Main Divide was completed to Cass in 1910 (Figure 1-1). The railway to Arthurs Pass was fully completed prior to the winter of 1914 (Wright & Wright, 2009).
Figure 1-2 A: Part of the journey to the West Coast made by coach between Broken River and Otira in lieu of the MDL, 1906. B: A photograph of the Otira Tunnel breakthrough on 20 July 1918. C: Workmen of the Otira Tunnel prior to the tunnel opening of 04 August 1923. Photographs from Wright & Wright, 2009.

Figure 1-3 A: Self-acting jig descending Broken River coal to coal bins at Avoca, locomotive moved the coal along the top of the hill. B: Avoca residents participating in the local jazz band C: Homes of the mining families during the 1920’s, although these houses are no longer present at Avoca. Photographs from Wright & Wright, 2009.
In May 1918 tunnel workmen could hear sounds of drilling and blasting from the other tunnelling team, indicating that the project was nearing completion. On 20 July 1918 the breakthrough was made (Figure 1-2; B, C). The official opening of the tunnel took place on 04 August 1923; total time from construction to opening had been 15 years for the Otira tunnel, three times longer than the initially agreed five years (Wright & Wright, 2009).

1.2.2 Avoca

Avoca became populated as a mining town when the Mount Torlesse Coal Company pursued mining of the Broken River Coal Measures in 1918. Amenities such as a store, post office, and school were built to service approximately 55 employees of the mine (Figure 1-3 B, C; Wright & Wright, 2009). A total of 72,501 tonnes of coal were exported from the Broken River coal mine during almost a decade of production. The complex operations at Avoca comprised four traction components - horse, steam, locomotive, and a self-acting jig - to transport coal from the Broken River Mine to the MDL (Figure 1-3A; Wright & Wright, 2009).

A number of fires occurred at the Avoca mine, the first on 23 May 1924, and part of the railway was washed out during a flood later in the 1920’s. Disasters combined with competition of cheap coal from the West Coast resulted in the Avoca coal operations losing economic viability, and the mine was forced to close in 1927 (Wright & Wright, 2009).

1.2.3 Slovens Creek Viaduct

Slovens Creek Viaduct was designed and built by G M Fraser, a Dunedin based contractor who won the contract in 1908. The viaduct comprises twelve spans over nine piers, with a total length of 166m and maximum height of 39.9m above ground. Nine spans are steel plate girders, with the remaining three 24.8m long steel deck trusses. Piers at each end of the viaduct are concrete abutments with integral ballast guards, three further piers are concrete pillars, and the remainder are steel towers that aid in span support (Loader, 2009; Figure 1-4).

The beginning of a tunnel was bored in the western abutment of Slovens Creek Viaduct, however the tunnel was aborted due to the soft and unstable nature of the Tertiary strata. The hill side was excavated as an alternative (Wright & Wright, 2009). Despite excavation efforts, the soft rock of the Avoca Bank proved troublesome for a number of years, with slips continually triggered by rainfall. Twenty-four hour surveillance was required during and after periods of intense rainfall (White 1998).
Landslide management and mitigation on the MDL was proposed in 1951, and commenced at Joyces Creek near Otarama Station in early 1952 (Figure 1-1; White, 1998). Following a period of prolonged rainfall in 1952 tertiary rocks of the Avoca Bank “slipped” and covered the railway in debris up to 3m thick, initiating intensive excavation and stabilisation works in the Slovens-Avoca corridor (White, 1998; NIWA, 2014). The costly maintenance and disruptions caused by this event (and many others) led to major excavation works, and excavation in the Slovens-Avoca corridor was complete by 1955 (White, 1998).

1.3 Thesis Aims and Objectives

The principal geotechnical matter to be addressed in this thesis is the relationship between lithologies and their control on slope stability. The thesis aims to:

- Assess evidence for an apparent translational landslide and the long term stability of Tertiary strata on the western abutment of the Slovens Creek Viaduct.
- Assess further evidence for slope instability, and long term stability of the rail corridor, in the Tertiary sequence between Slovens Creek Viaduct and Avoca.
- Provide a suitable basis for future geotechnical hazard and risk assessment of the 3km long rail corridor between Slovens Creek Viaduct and Avoca.

To determine the thesis aims, the project objectives are to carry out:
• Local and contextual mapping, for example engineering geomorphology, of the 1) Slovens Creek Viaduct area, and 2) the Slovens – Avoca rail corridor.
• Detailed geological and engineering geomorphology mapping, including geotechnical investigation of Slovens Creek Viaduct.
• Laboratory work and geotechnical classification of subsurface material for use in landslide modelling of the Slovens Creek Viaduct western abutment.
• Identification of potential hazardous landslides along the rail corridor between the viaduct and Avoca, which require hazard and risk assessment in future work.

1.4 Geological and Geomorphological Setting

Strictly speaking in a geographical sense, and in accordance with the work of Gage (1970), the Avoca area is additional to the Castle Hill Basin. Although this is the case, for the purposes of simplicity terminology in this thesis may refer to Avoca as the eastern extension of the Castle Hill Basin.

The Rakaia Terrane, accreted on the Gondwana margin approximately 145Ma, denotes the basement rock for a large portion of the eastern province in New Zealand, and particularly of the eastern South Island as illustrated in Figure 1-5, Area 2 (King 2000). The Torlesse Fault Zone, Cheeseman Fault, and Flock Hill Fault are found at the base of the Torlesse Range, Craigieburn Range, and Broken Hill of the Canterbury Foothills respectively (Figure 1-6). Each range consists of Rakaia Terrane geology, and the Castle Hill Basin lies in a triangular tectonic depression with Rakaia Terrane as the basement rock (Figure 1-1; Figure 1-6). Periodic movement on these faults, and on the Esk Fault (east of the Castle Hill Basin; Figure 1-6), has resulted in Rakaia Terrane lying in faulted contact with calcareous Mid-Oligocene strata throughout the Castle Hill Basin and specifically along the Esk Fault adjacent to Slovens Stream and the rail corridor (Figure 1-6; Gage 1970; McLennan and Bradshaw, 1984).

Late Cretaceous and Tertiary strata of the Castle Hill Basin rest unconformably on Rakaia Terrane, and are separated by an angular discordance (McLennan and Bradshaw, 1984). The stratigraphic succession of the Late Cretaceous and Tertiary strata is depicted in the top left of Figure 1-6. Stratigraphy in the east of the Castle Hill Basin excludes the Late Cretaceous Broken River Coal Measures and Late Cretaceous – Eocene Iron Creek Formation. McLennan and Bradshaw (1984) suggest that the Broken River Coal Measures and Iron Creek Formation were stripped from the eastern part of the Castle Hill Basin as a result of (pre-Oligocene) erosion, which was governed by normal faulting and local uplift nearby the current location of the Puketeraki Ranges (Figure 1-1). Pre-Oligocene uplift in close proximity to the east of the Castle Hill Basin is also considered the reason for the (early Oligocene) angular discordance across the Castle Hill Basin, which gently dips towards the southwest at approximately 6° (McLennan and Bradshaw, 1984).

The Tertiary rail corridor geology comprises the Puffer and Thomas Formations of the Porter Group, and the Enys Formation of the Motunau Group. The Porter Group was established by Gage (1970) and includes the
Figure 1-5 Figure to illustrate the Terrane accreted across New Zealand, with metamorphic overprints highlighted. The eastern province is dominated by Rakaia Terrane, particularly in the South Island. Rakaia Terrane is the basement rock of the field area and rail corridor, and is part of the Torlesse Composite Terrane that was accreted at approximately 145 Ma. Source: King (2000).
Figure 1-6 Map adopted from Gage (1970) which details the Castle Hill Basin triangle outlined in Figure 1-1. The name of the Iron Creek Greensand has since been changed to Iron Creek Formation by McLennan (1981). Note the major faults associated with each the Craigieburn Range (west), the Broken Hills (north), and the Torlesse Range to the south of the Castle Hill Basin, as well as the Avoca Fault obscured by Quaternary deposits southwest of Avoca.
Coleridge Formation, which is found in the west of the Castle Hill Basin. Collectively, the Porter Group consists of calcareous sandstone, limestone, and argillaceous (calcareous) tuff. The Puffer Formation and Coleridge Formation were deposited at approximately the same time, and therefore sit in the same stratigraphic horizon, however they can be distinguished by slight compositional differences and are found in the east and west of the Castle Hill Basin respectively (Figure 16; Gage 1970). The succeeding Enys Formation is non-marine and consists of siltstone, mudstone, lignite, and some coal.

Unconformably resting on Tertiary strata are Quaternary glacial and glacifluvial deposits, the extent of which are depicted in yellow in Figure 16. Quaternary deposits up to 1km wide remain east and west of the rail corridor between Slovens Creek Viaduct and Avoca, and obscure both the Avoca fault west of the rail corridor and the Esk Fault to the east (Figure 16).

The rail corridor, adjacent to Slovens Stream, is bounded by several landslides, which are the focus of this study and are explored throughout chapters 4 and 5. Although there are mapped landslides on the eastern side of Slovens Stream, they pose negligible risk to the rail corridor they are not considered in depth in this study.

1.5 Landslide Terminology

The term landslide is defined by Cruden and Varnes (1996) as “the movement of a mass of rock, debris or earth downslope”. With respect to failure mechanisms, the landslide type should be stated by using one or a combination of the following terms:

- **Fall** displays little or no shear displacement along the failure plane, and occurs when soil or rock detaches from a steep face. Typically, the material will descend rapidly to extremely rapidly by rolling, bouncing, or falling through the air.

- **Topple** refers to a mass of soil or rock moving outward from a slope in a forward rotation with the displaced mass’ centre of gravity above the point (or axis) of rotation.

- **Slide** is typically a translational or rotational failure and describes shear displacement across a narrow zone, or along one to numerous discrete surfaces.

- **Lateral spread** implies lateral extension of a softer underlying soil or rock, exacerbated by subsidence of overlying fractured material. It can be caused by liquefaction and is without a basal shear surface.

- **Flow** refers to slow to rapid viscous deformation in soil, and to slow steady deformation in rock.

Table 1-1 summarises how the above terms can be used in conjunction with the type of material in which the mass movement occurs. Each of the mass movements referred to in Table 1-1 are also illustrated in Figure 1-7. According to Cruden and Varnes (1996), ‘earth’ can be defined as a material
Table 1-1 Terms for type of movement when applied to the type of material. From Cruden and Varnes (1996).

<table>
<thead>
<tr>
<th>Type of Movement</th>
<th>Type of Material</th>
<th>Engineering Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fall</td>
<td>Rock Fall</td>
<td>Debris Fall</td>
</tr>
<tr>
<td>Topple</td>
<td>Rock Topple</td>
<td>Debris Topple</td>
</tr>
<tr>
<td>Slide</td>
<td>Rock Slide</td>
<td>Debris Slide</td>
</tr>
<tr>
<td>Spread</td>
<td>Rock Spread</td>
<td>Debris Spread</td>
</tr>
<tr>
<td>Flow</td>
<td>Rock Flow</td>
<td>Debris Flow</td>
</tr>
</tbody>
</table>

Table 1-2 Summary of terms for forming complete names of landslides. From Cruden and Varnes (1996).

<table>
<thead>
<tr>
<th>Activity</th>
<th>State</th>
<th>Distribution</th>
<th>Style</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Active</td>
<td>Advancing</td>
<td>Complex</td>
<td></td>
</tr>
<tr>
<td>Reactivated</td>
<td>Retrogressive</td>
<td>Composite</td>
<td></td>
</tr>
<tr>
<td>Suspended</td>
<td>Widening</td>
<td>Multiple</td>
<td></td>
</tr>
<tr>
<td>Inactive</td>
<td>Enlarging</td>
<td>Successive</td>
<td></td>
</tr>
<tr>
<td>Dormant</td>
<td>Confined</td>
<td>Single</td>
<td></td>
</tr>
<tr>
<td>Abandoned</td>
<td>Diminishing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stabilised</td>
<td>Moving</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relict</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Description of First Movement

<table>
<thead>
<tr>
<th>Rate</th>
<th>Water Content</th>
<th>Material</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely Rapid</td>
<td>Dry</td>
<td>Rock</td>
<td>Fall</td>
</tr>
<tr>
<td>Very Rapid</td>
<td>Moist</td>
<td>Soil</td>
<td>Topple</td>
</tr>
<tr>
<td>Rapid</td>
<td>Wet</td>
<td>Earth</td>
<td>Slide</td>
</tr>
<tr>
<td>Moderate</td>
<td>Very Wet</td>
<td>Debris</td>
<td>Spread</td>
</tr>
<tr>
<td>Slow</td>
<td></td>
<td>Flow</td>
<td></td>
</tr>
<tr>
<td>Very Slow</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extremely Slow</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Description of Second Movement

<table>
<thead>
<tr>
<th>Rate</th>
<th>Water Content</th>
<th>Material</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely Rapid</td>
<td>Dry</td>
<td>Rock</td>
<td>Fall</td>
</tr>
<tr>
<td>Very Rapid</td>
<td>Moist</td>
<td>Soil</td>
<td>Topple</td>
</tr>
<tr>
<td>Rapid</td>
<td>Wet</td>
<td>Earth</td>
<td>Slide</td>
</tr>
<tr>
<td>Moderate</td>
<td>Very Wet</td>
<td>Debris</td>
<td>Spread</td>
</tr>
<tr>
<td>Slow</td>
<td></td>
<td>Flow</td>
<td></td>
</tr>
<tr>
<td>Very Slow</td>
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<td></td>
</tr>
<tr>
<td>Extremely Slow</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extremely Rapid</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 1-7 Illustration of the terms described in text and by Varnes with examples of use of a combination of terms also discussed in text. The above figure was sourced from the British Geological Survey on 28 October 2015 at: http://www.bgs.ac.uk/landslides/how_does_BGS_classify_landslides.html
containing more than 80 percent particles less than 2mm in size, and ‘debris’ is defined as a soil containing 20-80 percent of particles greater than 2mm.

Table 1-2 provides further detail by summarising rates of movement, in addition to the type of activity (including style, distribution, and state). For reference, an extremely rapid landslide would typically be at a velocity greater than 3m/s, and an extremely slow landslide would move less than 60/mm/yr. Part of the information provided in Table 1-2 is also illustrated in Figure 1-7.

1.6 Track Terminology

1.6.1 Metrage

Metrage is marked out by trackside pegs every kilometre and every half kilometre, and is used as a way of communicating the track location of features such as culverts, train signals, or tack faults. Metrage is measured and read in kilometres, however is not always analogous to a metric kilometre. For example, if there have been changes in the track, such shortening or extending a corner, the metrage will remain the same for nearby infrastructure because the feature at the location has not changed, but the adjusted section of track will not remain the same length. Where the full kilometre metrage pegs on the trackside read a whole number, for example 70, the metrage of infrastructure is expressed in the format of 70.452. The whole number (70) is the metrage of the full metrage peg, and the following three digits are the distance from the full kilometre peg in meters. In this thesis, all metrage is referred to in the format 00.000, which includes full kilometre pegs.

1.6.2 Track Logs and Track Faults

Track logs are obtained by running a vehicle along the lines, which records ‘faults’ as changes track geometry. Different types of faults describe different types of discrepancies in the track geometry. The applicable track faults are defined below:

Top Faults refer to discrepancies in relative vertical geometry of the track.

Line Faults refer to curved or sinuous discrepancies in horizontal track geometry.

Gauge Faults refer to the distance between the tracks, relative to each other, and are therefore concerned with lateral track geometry.

1.7 Thesis Format

Table 1-3 summarises each of the chapters, and also outlines the structure of the thesis. For clarity throughout the thesis, each chapter includes an introductory section and a summary of the main findings.
<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Purpose(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Introduction</td>
<td>Thesis aims and objectives defined with project background and a brief geologic and geomorphological overview. Landslide terminology is set up for reference throughout the remainder of the thesis.</td>
</tr>
<tr>
<td>2</td>
<td>Thesis Methodology</td>
<td>Detailed outline of the thesis approach and methods used to meet objectives. Methods are referred back to throughout the thesis where necessary.</td>
</tr>
<tr>
<td>3</td>
<td>Physical Setting</td>
<td>Specifics on climate, tectonic history, geology and geomorphology of the project area.</td>
</tr>
<tr>
<td>4</td>
<td>Ground Model of the Slovens - Avoca Corridor</td>
<td>Engineering geology model of the rail corridor comprising engineering geomorphology maps, geological maps, cross sections, and three dimensional imagery.</td>
</tr>
</tbody>
</table>
| 5       | Ground Model of The Slovens Creek Viaduct Area | a) Engineering geology model of the viaduct area comprising an engineering geomorphology map, a geological map, deep geotechnical investigation data, cross sections, and three dimensional imagery.  

b) Stability analysis of the western abutment. Slide models consist of data derived from core obtained from deep geotechnical investigation. Sensitivity analyses are conducted alongside limit equilibrium, and pseudo-static analyses.  

c) Provide the engineering geological basis for risk assessment. |
| 6       | Conclusions | Concise summary of the main thesis findings, and recommendations for future work. |
2 Thesis Methodology

2.1 Introduction

This chapter outlines the fundamental processes involved in achieving the first three project objectives described in Chapter 1. The first two objectives, mapping and geotechnical investigation, required third party involvement during the steps taken to create the maps of the Slovens-Avoca field area, and to carry out deep geotechnical investigation and monitoring of the western abutment at Slovens Creek Viaduct. These objectives form the basis of this project. The third objective, geotechnical classification and landslide modelling, was achieved through laboratory testing and analyses and by using those results to produce realistic landslide models. Laboratory methodology, and landslide model selection processes are discussed in this chapter.

2.2 Aerial Survey

A detailed base map of the rail corridor was needed for the project. Existing maps of the Castle Hill Basin area were at a scale of 1:50,000 with 20 m contours and did not provide adequate detail for the small field area of this project, which is approximately 3km$^2$. An aerial survey was therefore required, and Synergy Positioning Systems were employed to undertake the aerial survey of the field area using an Unmanned Aerial Vehicle (UAV) on 07 November 2014. This technique was selected in favour of a LiDAR survey because the UAV survey is a cost-effective alternative and produces the same (albeit less detailed) geo-referenced data and imagery as LiDAR, using a technique called photogrammetry.

The contour maps of the rail corridor were created by the author using the data received from Synergy, which included orthophotography (geometrically corrected aerial photography) and 3D point cloud data as .las files; the latter being the same format as those received from a LiDAR survey. Point cloud data were filtered into ground and vegetation points as a part of the service from Synergy. Once this process is complete the data are referred to as ‘classified’. Using the classified point cloud data, and excluding points classified as vegetation or buildings, 2m contour base maps of the rail corridor were generated by the author in geo-referencing software (Surfer11). In the first instance, ArcGIS was explored as a geo-referencing tool, however the convenience of pre-existing ArcGIS shape files for railway infrastructure and water courses was not beneficial, as they were inaccurate compared to data obtained from the UAV survey. In addition, ArcGIS was unable to project the local co-ordinate system used for the survey, which was Mount Pleasant2000, and conversion to any other format proved troublesome. For these reasons, using the aerial orthophotography from the survey to trace the infrastructure and water courses onto the (identically geo-referenced) contour maps became preferable, and was done in Surfer 11.

An example of the classified data as a three dimensional ground model created in Surfer 11 is shown in Figure 2-2, and also included is an example of the aerial orthophotography overlain a three dimensional ground model (Figure 2-2); each ‘block’ seen in the base of both models is the representation of one .las tile received from
The survey. The tiles are bound together by geo-referencing as a ‘mosaic’ in Surfer11. The receipt of .las tiles enabled division of the rail corridor into four main A3 maps (Figure 2-2; Figure 2-3), with approximately two rows of tiles per A3 Map. In order to fit each of the maps on to an A3 page, a scale of 1:3,333 was required.

The accuracy of the UAV survey data allowed for 1m contour spacing, however the maps were too ‘busy’ with 1m contours applied. Therefore, as already mentioned, contours of 2m spacing were applied to the initial base maps, which provided enough precision to accurately map the geomorphological and geological features observed in the field. The final maps produced for the study are presented in the form of 5m contours because there is a large amount of information presented and, with the detail of 2m contours, these became too complex for final presentation.

2.3 Fieldwork and Mapping

Field work was undertaken in three stages using the 2m contour base maps at a scale of 1:3,333. Prior to undertaking any field work, and outlined below, the appropriate permissions from KiwiRail had to be obtained.

In accordance with KiwiRail health and safety regulations, a short one day course in track safety was completed at ENGEIO. The course is named Individual Train Detection (ITD) and is provided for the purpose of staying safe when working near the rail corridor. In addition to this, a Permit to Enter had to be obtained prior to each site visit, and the area manager was to be informed of the intended work dates.

Preliminary investigation of the field area took place in December 2014 using the University of Canterbury (UC) Cass Field Station as a base. The purpose of the week long trip was to solidify background and desktop research, become acquainted with geomorphological features observed in 3D orthophotography, collect samples and begin mapping whilst becoming familiar with the rail corridor geology.

For the purpose of completing detailed engineering geomorphology and geology maps, a secondary stage of field work was carried out over a three week period in January 2015. A handheld GPS was taken to add way points of geological contacts, outcrops, and seepages.

Engineering geomorphology mapping consisted of mapping infrastructure, such as metrage, culverts and retaining structures, as well as mapping surface water (seepages). It is worth noting that the mapping of seepages was carried out during a very dry period of summer, and so the extent of surface water mapped could be underestimated. Geomorphological mapping comprised landslides, back-tilted blocks and hummocky ground. Landslide ages throughout the corridor are unknown, and so mapping of the features was done using geomorphic expression and cross cutting relationships to determine their relative age.
Figure 2-1. 3D rail corridor ground model created in Surfer 11 using classified point cloud data from the UAV survey. Slovens Stream is the deepest depression (coloured blue in the southeast), and Slovens Creek Viaduct crosses the stream at 844000. The rail corridor to Avoca is the flat feature west of the stream, which truncates the toe of the western hills. The scale of the $z$ axis is in meters, and there is no vertical exaggeration.

Figure 2-2 Orthophotography overlain the 3D ground model of the rail corridor. Note the MDL cut into the western flanks. Slovens Creek Viaduct is in the southeast (844000, 332000) and Avoca is in the northwest (846800, 330800). Red outlines indicate the area of the main four A3 maps, and their correlation with the .las tiles discussed in text.
Figure 2-3 A reference map for Maps 1 to 6 created for the rail corridor. Note Map 4 covers an area of interest between Maps 3 and 5. Also note map 6 is covered by map 5, however detailed information of site investigation at Slovens Creek Viaduct is applied to map 6 (see section 2.4).
Geological mapping consisted of mapping structural measurements within the 3km² field area, and in some cases geology outside of the specified field and map area was explored to enable comprehension of geological structures found within the field area. The majority of geology within the field area is obscured by vegetation and/or landslide debris, or is not in situ due to anthropogenic dumping of failed materials on the outside of the track. Limited outcrops of in-situ lithologies were found in railway cuttings and in Slovens Stream; only one formation contact was observed.

Following the fieldwork, the geological and geomorphological data were digitised. Figure 2-3 is a reference map to illustrate the location of each of the six numbered maps of the rail corridor. Each map number (1-6) has a geological map, an engineering geomorphology map, and a hazard zonation map associated.

The final stage of fieldwork was deep geotechnical investigation in the western abutment of the Slovens Creek Viaduct. The investigation site is in a mapped landslide. The deep geotechnical investigation site was identified and decided upon during the work carried out in stages one and two of fieldwork. Rock mass rating (RMR) and Geological Strength Index (GSI) were calculated at viaduct outcrops for use with the Hoek-Brown criterion in the landslide modelling stage (Section 2.6).

2.4 Boreholes, Inclinometer, and Piezometer

Speight Drilling Limited was employed to drill two boreholes in May 2015. Sonic wire line technology with water injection was used, and both boreholes are within the apparent landslide on the western viaduct abutment (Figure 2-4).

Borehole one (BH1) is located approximately 50m northwest of the western viaduct abutment, and consisted of cored drilling to 36m plus installation of inclinometer casing. Inclinometer casing was installed to a depth of 32m with one set of grooves parallel and the other normal to the direction of failure. BH1 was successfully completed on 14 May 2015. Overall core recovery was good, at 90% or above, and a potential failure surface was found at 27.1m – 27.3m. The majority of logging was carried out on site in accordance with the New Zealand Geotechnical Society (NZGS) guidelines; the core and logs were later reviewed and finalised in the University of Canterbury’s engineering geology laboratory.

Borehole two (BH2) was not cored, and was for the sole purpose of installing a standpipe piezometer. BH2 was drilled 10m southeast of BH1 and was intended to proceed to 28m to enable screening between 24m and 28m, however drilling was terminated at 15m due to caving and visible disturbance of BH1 inclinometer grouting. The piezometer was instead installed to a depth of 14m on 17 May 2015, and has been dipped concurrently with inclinometer site visits (outlined below).
Figure 2–4 A) Drilling rig set up on the western abutment of Slovens Creek Viaduct. The viaduct can be seen behind the digger and container, and is indicated by the white arrows. B) Core being extracted from BH1, note the saturation of the core due to the water injection technique. C) Thomas Formation Tuff from BH1, note the close fracturing and poor quality of the rock.

Geotechnics Limited were employed to monitor the inclinometer, with a total of two readings including the base read. The inclinometer readings form a significant part of the information for this project, from which some conclusions as to the stability of the western abutment can be drawn.
2.5 Laboratory Analysis

2.5.1 Uniaxial Compressive Strength

Uniaxial compressive strength (UCS) testing was carried out to obtain accurate strength parameters for use in landslide modelling (Figure 2-5). UCS data were maximised by incorporating sonic wave velocity prior to each test, and also by applying strain gauges to each sample (Figure 2-5 A, B, C). Sample preparation, testing, and result calculations were carried out in accordance with the ISRM Suggested Methods for Rock Characterisation, Testing and Monitoring (1974-2006).

It was intended to perform an equal amount of UCS tests on the two major lithologies found in the core (Puffer Formation and Thomas Formation Tuff; Figure 2-5 A and B respectively), but the poor condition of the tuff core meant samples could not be cut to size. Few lengths of intact (tuff) core exceeded a 2:1 ratio (required by the above standards) as they were truncated by angular joints or close fracturing (Figure 2-5, D). In addition, the majority of intact lengths of core readily fell apart when handled. All but one attempt at trimming sufficient pieces of tuff core were unsuccessful because of vibration induced fracturing from the diamond saw; one tuff sample underwent the UCS test (Figure 2-5, F).

2.5.2 Point Load Test

The point load test was incorporated to enable a comparable test between the Puffer and Thomas formations in lieu of UCS results. Although not suitable for UCS, tuff core samples suitable for point load testing were abundant. Diametral and axial (Figure 2-6, B and C respectively) point load testing was carried out using the ISRM Suggested Methods for Rock Characterisation, Testing and Monitoring (1974-2006).

2.5.3 Direct Shear (Rock)

Triaxial testing for shear data was preferred in the first instance, however the sonic drilling technique used returned core of diameter 67mm, which was too large for the loading cells in the Engineering Geology Laboratory at the University of Canterbury. Therefore, direct rock shear was carried out using the Portable Rock Shear Box as an alternative. The testing was carried out in accordance with the Portable Rock Shear Box Operations Manual, Version 1.1, supplied by Robertson Geologging Ltd. See Appendix A.

2.5.4 Direct Shear (Soil)

Direct shear testing was carried out on the clay-rich material found at the interpreted failure surface (Figure 2-7, B-D). The most applicable test for the sample was uncertain in the first instance due to the characteristics
Figure 2-5  A) Puffer Formation samples prepared for UCS testing. B) Thomas Formation Tuff prepared for UCS testing. C) sonic wave velocity testing on Puffer Formation sample D) Fracturing of Thomas Formation Tuff highlighted with a black permanent marker. E) Puffer Formation sample undergoing UCS test, note strain gages attached. F) Thomas formation Tuff undergoing UCS test, note strain gauges attached.
Figure 2-6 A) Thomas Formation Tuff sample undergoing a diametral test in the Point Load Tester. B) An example of the sample after a successful diametral test, the core is 67mm in diameter. C) the same sample from photo B used to carry out axial point load tests.

Figure 2-7 A) Failure surface clay dry and brittle (left), and wet and plastic (right). B) remoulded saturated clay ready for direct shear testing. C) Clay following the direct shear test of photo B, the induced shear plane is visible above the ruler. D) Direct shear testing equipment in the Engineering Geology Laboratory at the University of Canterbury.
of the unit. When dry, the clay-rich material appeared too strong for soil and too weak for rock, however when
the sample was wet it displayed soft plastic behaviour (Figure 2-7, A). The in situ moisture content is uncertain
because the sonic drilling technique used water injection, and therefore the core was saturated by the drilling
method, if not already saturated by the in situ conditions. In light of the phreatic surface reading -0.8m in the
standpipe piezometer some 10m away, the final decision was to saturate the sample, as it was found on site in
the drill hole, and to proceed with the direct shear soil test as opposed to direct rock shear (Figure 2-7 B-D) to
obtain Mohr-coulomb parameters cohesion ($\mathbf{c}_c$; kPa) and phi ($\phi$; degrees) for use in the landslide modelling
phase.

2.6 Landslide modelling

The topographic profile for the viaduct landslide model was created from the contour map in Surfer 11. The x
and y coordinates of the profile were exported as a text document from Surfer 11 and imported to Rocscience
software (Slide 6.0). This process provided accuracy in slope topography. Subsurface geometry of the model
was constrained as best as possible by extrapolation of subsurface information from BH1 and outcropping
gеology at the surface.

Limit equilibrium and pseudo-static analyses utilised piezometric data and strength data obtained during
laboratory testing of BH1 material, which includes the shear strength of the clay-rich material of the interpreted
failure surface, and the uniaxial compressive strength (UCS) of the landslide debris and bedrock. The limit
equilibrium and pseudo-static ground models also use geological strength index data collected in the field to
develop generalised Hoek-Brown strength parameters for the appropriate materials within the model.
Sensitivity analyses were also conducted alongside limit equilibrium and pseudo-static analyses. The details
of the slope stability analyses are discussed in Chapter 5 of this thesis.
3 Physical Setting

3.1 Introduction

This chapter details the physical setting of the Castle Hill Basin, and in particular the rail corridor in the eastern extension of the basin. Physical properties such as climate, topography, and geological structure collectively determine the geomorphology of the rail corridor. The introduction to this chapter provides a broad summary of tectonism and climate, as it relates to the Castle Hill Basin prior to focusing on the Castle Hill Basin itself.

The continental crust of New Zealand comprises many complex structural and lithological heterogeneities - particularly in the South Island - resulting from a series of complex tectonic events (Ghisetti & Sibson 2006). Tectonostratigraphic terrane, such as the Rakaia Terrane of the Castle Hill Basin (Figure 1-5), were accreted along the Gondwana margin during intermittent collision related to the Tuhua and Rangitata Orogenies (370-330Ma and 142-99Ma, respectively). Locally interrupted by tilting and uplift in the late Eocene, and regionally modified by mid-Oligocene basaltic eruptions, the Cretaceous-Tertiary succession of the Castle Hill Basin records a transgressive-regressive sequence between the Rangitata and Kaikoura tectonic events, and the deposits remain as in-faulted outliers between elevated ranges of Rakaia Terrane as a result of the Kaikoura Orogeny (Gage 1970).

New Zealand’s complex topography and geographic location, relative to the Southern Hemisphere mid-latitude westerly wind belt, results in significant temporal and spatial climate variability. The National Institute of Water Atmospheric Research (NIWA) state the average annual rainfall throughout New Zealand is between 600 and 1600 mm, with mean annual temperatures ranging from 16°C in the North Island to 10°C in the South Island. Topography, such as that of the Southern Alps, interacts with the prevalent westerly winds resulting in localised climate variation throughout the South Island, particularly within the mountains and basins of the Canterbury foothills (Ackerley et al., 2012).

3.2 Climate

Annual precipitation in the Waimakariri catchment (Figure 1-1) ranges from 600 mm per year in low-lying plains to 6400 mm in areas of high altitude, with an average annual temperature of 6.5°C (Barrell et al., 2011; NIWA, 2014). Figure 3-1 from Barrell et al. (2011), presents rainfall data collected by the Institute of Geological and Nuclear Sciences (GNS) during the period 1950 to 1981, and shows annual rainfall to be between 1200 and 1600 mm in the Slovens to Avoca corridor. Recent rainfall measurements from Environment Canterbury (ECan) for the period October 2013 to October 2014 are similar to the GNS data, with annual rainfall of 1017 mm and 1020 mm at Grasmere and the Waimakariri-Esk River junction, respectively (Figure 3-1).
On 26 January 1952, the Slovens-Avoca corridor (and surrounding area) was subject to a period of prolonged rainfall that resulted in flooding, landsliding, and damage to railway infrastructure. The railway was washed out in three places between Springfield and Arthurs Pass. There were two slips at Slovens Creek, and a washout at Annat (NIWA, 2014; Barrell et al., 2011). This event highlights the effects of rainfall on slope stability and infrastructure performance.

Figure 3-1 Illustration of annual precipitation in the central South Island. Note the location of the Castle Hill Basin indicated by the red rectangle, and the locations of Lake Grasmere and Waimakariri-Esk River junction at the red arrows. Figure from Barrell et al. (2011).

3.3 Cretaceous-Tertiary Rocks of the Castle Hill Basin

Cretaceous-Tertiary rocks of the Broken River area and Castle Hill Basin overlie basement Rakaia Terrane and record a transgression-regression sequence, as well as mid-Oligocene submarine volcanism (Figure 3-2). The stratigraphic sequence in the Castle Hill Basin, and in the eastern extension to Avoca, is described in the following sections starting from the basement.
3.3.1 Rakaia Terrane

The Rakaia Terrane is part of the Torlesse Composite Terrane of the eastern province (King 2000), and forms the basement rock for this area (Figure 1-5). The Rakaia Terrane is Triassic, and comprises greywacke, argillite, and conglomerate in part (Gage 1970).

3.3.2 Eyre Group

Two formations of the Eyre group are found in the Castle Hill Basin, the Broken River Formation and the Iron Creek Formation (Figure 1-6). Separated by the angular unconformity and following peneplanation of Rakaia Terrane rocks, the shallow marine Late Cretaceous Broken River Formation rests on Rakaia Terrane. The angular discordance (6° dipping to the southwest) is persistent across the Castle Hill Basin (Figure 3-3; McLennan & Bradshaw, 1984). Gage (1970) describes the Broken River Formation as:

“...uneven stratified, current bedded, white quartz sandstone, dark grey carbonaceous silt and sand, and lensoid beds of sub-bituminous coal”

Browne and Field (1985) suggest that the Broken River Formation marks the beginning of the abovementioned transgression. The Iron Creek Formation (Eocene-Oligocene) comprises two members, the lower Avoca Sand member and the Iron Creek Greensand (McLennan, 1981).

3.3.3 Porter Group

Deep marine Porter Group limestones, marls, and tuffs of Oligocene age succeed the Iron Creek Formation (Figure 3-3). The early Oligocene Coleridge Formation is found in the west of the Castle Hill Basin, and comprises massive light grey silty limestone (some faint stratification) and calcareous grey mudstone (Gage 1970). The Porter Group probably records the maximum of the transgression, which was interrupted regionally by sub-marine volcanism during Late Oligocene deposition of the Thomas Formation (Gage 1970).

In the eastern extension of the Castle Hill Basin, the Puffer Formation rests unconformably on Rakaia Terrane, and consists of four members comprising calcareous glauconitic sandstone and argillaceous limestone. The Puffer and Coleridge Formations are approximately the same age, and therefore share the same stratigraphic horizon. However, compositional differences, such as colour and particle size, allow for them to be distinguishable from one another (Figure 3-3; Gage 1970).
Figure 3-2  A) Site location relative to the South Island, the Castle Hill Basin is indicated by the red square. B) Zoomed in to the red square inset of image A. Note the ‘Brechin Burn Outlier’ discussed in text to the northeast of the top right corner of the red square inset, and also note major active and inactive faults that define the Castle Hill Basin. C) The Castle Hill Basin as outlined by the red square inset of image B. Note Avoca in the eastern extension of the Castle Hill Basin.
**Stratigraphic succession as found in the western area of Castle Hill Basin**

<table>
<thead>
<tr>
<th>Formation</th>
<th>Age</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Enys Formation</td>
<td>Miocene to Pliocene</td>
<td>Pale green fine sandstone with 30cm-2m beds of mod. rounded greywacke pebble and cobble conglomerate.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pale green, locally crossbedded medium to very fine sandstone with thin mudstone and mudstone clast layers.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blue grey fine sandstone with concretionary mollusc rich shell beds, thin lignite layers and lenticular bedded interlaminated sandstone and mudstone.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(total approximately 1000m) Disconformity and paraconformity.</td>
</tr>
<tr>
<td>Thomas Formation</td>
<td>Late Oligocene</td>
<td>Limestone: yellow green bryozoan-echiniderm-mollusc biosparite. Locally cross bedded and tuffaceous where resting on tuff.</td>
</tr>
<tr>
<td></td>
<td>Ld-w.</td>
<td>Tuff: Fine to coarse green to brown basic lithic tuff. Locally crossbedded and fossiliferous.</td>
</tr>
<tr>
<td>Coleridge Formation</td>
<td>Early-Oligocene</td>
<td>Lwh. Grey clayey micritic limestone with Zoophycus, interbedded with fine white sand.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blended Contact.</td>
</tr>
<tr>
<td>Iron Creek Formation</td>
<td>Late Cret-Eocene</td>
<td>Dark green richly gluconitic medium sandstone. One or more shell beds with numerous Conchothyra inoceramus and Ostrea at or near the base.</td>
</tr>
<tr>
<td></td>
<td>Mh-A.</td>
<td>Sharp Contact.</td>
</tr>
<tr>
<td>Broken River Formation</td>
<td>Coal Measures</td>
<td>White, green and brown coarse to fine cross bedded sandstone, mudstone and muddy coal.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Angular unconformity.</td>
</tr>
<tr>
<td>Triassic Rakia Terrane</td>
<td>Greywacke, argillite and conglomerate.</td>
<td></td>
</tr>
</tbody>
</table>

**Stratigraphic succession as found in the eastern extension of the Castle Hill Basin**

<table>
<thead>
<tr>
<th>Formation</th>
<th>Age</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Enys Formation</td>
<td>Miocene to Pliocene</td>
<td>Pale green fine sandstone with 30cm-2m beds of mod. rounded greywacke pebble and cobble conglomerate.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pale green, locally crossbedded medium to very fine sandstone with thin mudstone and mudstone clast layers.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blue grey fine sandstone with concretionary mollusc rich shell beds, thin lignite layers and lenticular bedded interlaminated sandstone and mudstone.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(total approximately 1000m) Disconformity and paraconformity.</td>
</tr>
<tr>
<td>Thomas Formation</td>
<td>Late Oligocene</td>
<td>Tuff Late Oligocene, Ld-w. Fine to coarse green to brown basic lithic tuff. Locally crossbedded and fossiliferous.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sharp Contact.</td>
</tr>
<tr>
<td>Puffer Formation</td>
<td>Early-Oligocene</td>
<td>Lwh. Pale green to grey silty micritic limestone or glauconitic calcareous sandstone, with Zoophycus, interbedded with fine to medium sand. Glauconitic throughout.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Angular unconformity.</td>
</tr>
<tr>
<td>Triassic Rakia Terrane</td>
<td>Greywacke, argillite and conglomerate.</td>
<td></td>
</tr>
</tbody>
</table>

**Note:**

*Thomas Formation Limestone is absent from the eastern extension of the Castle Hill Basin (at Avoca)*

The Puffer Formation and Coleridge Formations share their stratigraphic horizon and are found in the east and west of the Castle Hill Basin, respectively.

The Iron Creek Formation and Broken River Formations are absent in the eastern extension of the Castle Hill Basin. Previous names, assigned by Gage (1970) in brackets.

---

*Figure 3-3 Stratigraphic column taken and adjusted from Bradshaw (1975).*
The Thomas Formation conformably rests on both the Coleridge and Puffer Formations in the west and east of the Castle Hill Basin, respectively. The Thomas Formation comprises limestone members and tuff members, however the Thomas Formation limestone members are absent at Avoca (Figure 3-3).

3.3.4 Motunau Group

The Enys Formation includes conglomerate, coals, shells, carbonaceous mudstone, mudstone and pale green siltstone. Basal shallow marine sediments grade upward into estuarine (possibly lacustrine and fluvial) deposits, marking the end of the transgression-regression cycle that this tertiary sequence records. The Enys Formation lies disconformably on the Porter Group and is overlain, unconformably, by Pleistocene glacifluvial and glacial deposits (Figure 3-3), which are discussed in Section 3.5.

3.3.5 Previous Formation Names

Some previous geological work was done in the Castle Hill Basin, of which a table is presented in Gage (1970), but it was Gage himself who produced the first complete account of stratigraphy. Gage assigned a number of formation names and terminology, which were slightly adjusted by McLennan in 1981. Recent mapping by GNS as a part of the QMap series has applied different terminology for some of the formations in the Castle Hill Basin, and in the eastern extension at Avoca; these are summarised in Table 3-1.

The scale of the GNS QMaps is 1:250 000, resulting in some generalisations being made with regard to the formation names. The in-faulted Brechin Burn outlier, comprising the Brechin Formation and Esk Formation (discussed in Newman and Bradshaw, 1981), is associated with the Castle Hill Basin outlier by related infaulting along the Esk Fault. Figure 3-2 B illustrates the position of the Brechin Burn Outlier relative to the Castle Hill Basin and Esk Fault, and similarities between the Brechin and Esk Formations, with the Enys and Puffer Formations respectively, are summarised in Tables 3-2 and 3-3. The similarities found would readily justify and explain any generalisations made at the scale of a regional synthesis.

For the mapping scale used in this project, the Formation names applied by Gage in 1970 are used for the Castle Hill Basin, and include the adjustments made by McLennan in 1981 (Table 3.1).
Table 3-1 Summary of formation name changes and the terms used in this thesis.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Late Pleistocene – Quaternary</td>
<td>Glacial and Glacifluvial Deposits</td>
<td>Glacial and Glacifluvial Deposits</td>
<td>Glacial and Glacifluvial Deposits</td>
<td>Glacial and Glacifluvial Deposits</td>
</tr>
<tr>
<td>Miocene-Early Pliocene</td>
<td>Enys Formation</td>
<td>Enys Formation</td>
<td>Brechin Formation</td>
<td>Enys Formation</td>
</tr>
<tr>
<td>Oligocene</td>
<td>Puffer, Coleridge and Thomas Formations</td>
<td>Puffer, Coleridge and Thomas Formations</td>
<td>Esk Formation, Thomas Formation Limestone and Thomas Formation Tuff, respectively</td>
<td>Puffer, Coleridge and Thomas Formations</td>
</tr>
<tr>
<td>Late Cretaceous</td>
<td>Iron Creek Greensand</td>
<td>Iron Creek Formation</td>
<td>Iron Creek Formation</td>
<td>Iron Creek Formation</td>
</tr>
<tr>
<td>Late Cretaceous</td>
<td>Broken River Coal Measures</td>
<td>Broken River Formation</td>
<td>Broken River Formation</td>
<td>Broken River Formation</td>
</tr>
<tr>
<td>Triassic</td>
<td>Torlesse Group</td>
<td>Torlesse Group</td>
<td>Rakaia Terrane</td>
<td></td>
</tr>
</tbody>
</table>

Table 3-2 Summary of similarities between the Brechin Formation and Enys Formation, at Brechin Burn and Castle Hill Basin, respectively.

<table>
<thead>
<tr>
<th>Brechin Formation</th>
<th>Member Description</th>
<th>Age</th>
<th>Enys Formation</th>
</tr>
</thead>
<tbody>
<tr>
<td>B4</td>
<td>Conglomerate, Silty Sand</td>
<td>Miocene to Pliocene</td>
<td>Upper</td>
</tr>
<tr>
<td>B3</td>
<td>Mudstone</td>
<td>Miocene</td>
<td></td>
</tr>
<tr>
<td>B2</td>
<td>Sandstone, Silty, Carbonaceous</td>
<td>Lower</td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>Sandy Limestone</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3-3 Summary of similarities between the Esk Formation and Puffer Formation, at Brechin Burn and Castle Hill Basin, respectively.

<table>
<thead>
<tr>
<th>Esk Formation</th>
<th>Member Description</th>
<th>Age</th>
<th>Puffer Formation</th>
</tr>
</thead>
<tbody>
<tr>
<td>E7</td>
<td>Basaltic Tuffs</td>
<td>Oligocene</td>
<td></td>
</tr>
<tr>
<td>E6</td>
<td>Glaucconite Quartzarenite</td>
<td></td>
<td>P4</td>
</tr>
<tr>
<td>E5</td>
<td>Micritic Limestone</td>
<td>Early Oligocene</td>
<td>P4</td>
</tr>
<tr>
<td>E4</td>
<td>Massive Greensand</td>
<td>Early Oligocene</td>
<td>P3</td>
</tr>
<tr>
<td>E3</td>
<td>Concretionary Greensand</td>
<td>Late Eocene</td>
<td>P2</td>
</tr>
<tr>
<td>E2</td>
<td>Pillow Lava, Breccias, and Tuffs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E1</td>
<td>Basal Greywacke Conglomerate</td>
<td></td>
<td>P1</td>
</tr>
</tbody>
</table>
3.4 Geological Structure of the Castle Hill Basin

The major faults of the Castle Hill Basin are shown in Figure 3-4. The Cretaceous-Tertiary cover has been compressed into complex surficial periclinal folds of highly variable trend, plunge and profile, where faulting and uplift of the rigid Rakaia Terrane basement has induced strong local non-uniform stresses (Bradshaw 1975). Great variability occurs within single folded structures of the Castle Hill Basin, therefore Bradshaw (1975) suggests that faulting of the Rakaia Terrane is not older, but actually contemporaneous with folding, and such variation within single structures is likely due to development of folds across fault boundaries. Following an in-depth study of the folding of Tertiary cover in the Castle Hill Basin, Bradshaw (1975) suggests the structural development of the basin to have occurred in the following sequence:

- Initially, the Broken Hill block was uplifted in the east, and contributes to the gentle westerly dip and angular discordance throughout the basin. Thereafter, northeast-trending reverse faults of the Craigieburn Fault (Figure 3-4), comprising a southeasterly downthrow, developed at the base of the present day Craigieburn Range as a result of compressional stress. Over time, north to northwest-trending faults, such as the Flock Hill Fault (Figure 3-4), joined to the northeast trending reverse faults. The combination of motion on these faults, strike slip and reverse dip slip respectively, is considered to be the reason for particular complexities observed in the folding patterns in the northwest of the basin.

- Bradshaw (1975) postulates the Cheeseman Fault, which separates the uplifted Craigieburn Range from the Castle Hill Basin, was the next fault to rupture. The Cheeseman Fault is a fault zone comprising northwest-, north-, and northeast-trending faults, each with an easterly downthrow. According to Bradshaw, the final fault development of the Castle Hill Basin was the Torlesse Fault Zone, with uplift of the Torlesse Range.

- An angular unconformity persists across the Castle Hill Basin, between Rakaia terrane and the late Cretaceous-Tertiary cover, and is mentioned but not addressed in Bradshaw’s (1975) account of structural development, although it is addressed by McLennan and Bradshaw (1984). Here, “the removal of some 200 – 300m of late Cretaceous- early Tertiary rocks” (McLennan and Bradshaw, 1984 p. 301) is attributed to tectonism, and possibly to the incipient uplift of the Puketeraki Range to the east (Figure 3-5).
Figure 3-4 Major faults and ranges of the Castle Hill Basin. Figure taken and annotated from GNS online QMap

Figure 3-5 Representation of the angular unconformity in the Castle Hill Basin. Note the Puffer Formation in the east and Coleridge Formation in the west. Also note truncation of the Castle Hill Basin Formations by the Esk Fault, and uplift of Rakaia Terrane. Figure from McLennan and Bradshaw (1984).
3.5 Quaternary Geology and Geomorphology of the Castle Hill Basin

3.5.1 Gage (1958)

The paper produced by Gage in 1958, can be regarded as the definitive paper on the glaciations of the Waimakariri Valley. According to Gage (1958), the Waimakariri Valley records five periods of major ice advance (Table 3-4). Differences in the morphology, surface gradients, elevation, weathering, and distribution, enabled glacial and glaciofluvial deposits to be distinguished. The Avoca Glaciation, part of the Waimaungan Glacial Stage, was thought to be the most extensive of the five ice advances. The Avoca Glaciation was not subdivided by Gage, however he does note that there were most likely multiple advances. Gage postulated that the Waimakariri Glaciation was separated by an interglacial stage and subdivided into four events comprising the Woodstock, Otarama, Blackwater and Poulter advances. Figure 3-6 depicts the spatial distribution of the Avoca Glaciation and subsequent advances of the Otiran Glacial Stage in the east of the Castle Hill basin, as mapped by Gage (1958).

Named after the location of the thickest remaining deposits, Gage infers that the Avoca glaciation occupied the whole of the Waimakariri catchment at its maximum. The Torlesse and Puketeraki ranges restrained the glacier somewhat, so that its course was directed through (what is currently known as) the Waimakariri/Staircase Gorge. Evidence suggests the Avoca Glaciation reached as far as Sheffield on the Canterbury Plains, and ice was thick enough to leave deposits on No Mans Land (1116 masl; Figure 3-6). Deposits of the Avoca glaciation can be observed in Slovens Creek where coal sand laminae and thinly bedded silt unconformably lie beneath Blackwater-II outwash gravels.

The Woodstock advance of the Otiran Glacial Stage is named after the location where deposits are best preserved, which is Woodstock Station. There is uncertainty surrounding the extent this advance reached at its maximum, however it is concluded by Gage that the Woodstock Advance did not flow above the Avoca Plateau, nor did it occupy the Castle Hill Basin. Gage (1958, p. 138) refers to the advance as ‘an early sub-stage of the Otiran Glacial Stage’.

In the work of Gage, the Otarama advance was the final advance to reach the southern end of Staircase Gorge. The Otarama advance filled the Waimakariri Valley in its entirety, spanning from Mount White (east of the Waimakariri River) to the St. Bernard Range (west of the Waimakariri River) with an ice distributary between Broken Hill, the St. Bernard Range, and through the Broken River Valley (Figure 3-6).

Gage (1958) also notes that, separated by a brief recession, Blackwater-I and Blackwater-II deposits record two distinct advances. It is hypothesised that ice streams of the Blackwater advance occupied all main northern tributaries entering the Waimakariri Trough, and filled its upper reaches. In addition,
Figure 3-6 Glaciations of the Waimakariri Catchment as mapped by Gage (1958).
one of two major distributaries followed the path of Slovens Creek, where Blackwater-I and Blackwater-II outwash terraces are well preserved. Extensive outwash deposits can also be found in Slovens Stream, as illustrated in Figure 3-6.

The final major advance postulated by Gage (1958) is the Poulter advance, of which significant terminal moraine deposits lie at the mouth of the Poulter River, hence the name assigned. Joined to the Waimakariri Glacier at its maximum, the Poulter Glacier was thought to have then receded.

Gage provides a relative chronology based on geomorphology and position of deposits in the valley. Recently, a paper by Rother et al. (2015) provides a chronology using $^{10}$Be cosmogenic dating. The absolute dates have been determined at ten sites in the Waimakariri Catchment (Table 3-4; Figure 3-7).

### 3.5.2 Rother et al. (2015)

Table 3-4 summarises the advances and relative ages as mapped by Gage (1958), and compares these to the absolute dates found by Rother et al. (2015). Note that the Otarama advance of Gage (1958) is now dated to have been part of the Early Blackwater advance during the Last Glacial Maximum (LGM; 26-17ka), and thus is much younger than previously thought.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Poulter I &amp; II</td>
<td>LGM</td>
<td>16-17</td>
<td>LGM</td>
<td>17-18</td>
<td>(Late)Late Blackwater</td>
</tr>
<tr>
<td>Blackwater I, II, and III</td>
<td>24-26</td>
<td>19-21</td>
<td>(Mid)Late Blackwater</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Otarama</td>
<td>Early LG or Waimean</td>
<td>60-70 or 135</td>
<td>24-26</td>
<td>Early Blackwater</td>
<td></td>
</tr>
<tr>
<td>Woodstock</td>
<td>Penultimate</td>
<td>250</td>
<td>Penultimate</td>
<td>-</td>
<td>Woodstock</td>
</tr>
<tr>
<td>Avoca</td>
<td>Antepenultimate or Penultimate</td>
<td>&gt;250 (?$^{10}$) LGM</td>
<td>25 (?$^{10}$) Early Blackwater</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In addition, Late Blackwater can be subdivided into Mid- and Late- Late Blackwater (Table 3-4). From the cosmogenic dates obtained in Rother et al, there is no clear distinction between the previously named Poulter I and II, and Blackwater II, and III advances, therefore distinction by name can be removed and they can be considered (Late)Late Blackwater and (Mid)Late Blackwater, respectively (Table 3-4).

Moreover, absolute dates from the Avoca Plateau (Site 2; Figure 3-7) show deposits to be of LGM (Early Blackwater; dated at 25,635±798 ka) as opposed to the previously thought, penultimate or antepenultimate Avoca advance. Finally, based on the temperature dependent model of Rother et al. (2015), it is possible that the Early Blackwater Advance (previously named Otarama) had briefly extended beyond the Otarama moraine to the edge of the Canterbury Plains (Site 4, line C; Figure 3-7).
There are limitations to the study of Rother et al. (2015), in that more cosmogenic dates are required in conjunction with pollen studies, such as those carried out by Moar and Gage (1972). Nonetheless, the study of Rother et al. (2015) has recognised the work of Gage (1958) is essentially correct, however the LGM was possibly more extensive than previously thought and, in terms of ice fluctuation, certainly more complex. It is worth noting that the outwash terraces of the Slovens-Avoca corridor, which overlie the Tertiary strata, are clearly related to the LGM, and limited to the early LGM and ice retreat associated with the Early Blackwater Advance.

Figure 3-7 Extent of the LGM, as modelled by Rother et al., 2015. Note dated sites and the possible full extent of the early Blackwater advance (previously Otarana).

36
3.6 Geology of the Rail Corridor

3.6.1 Structure in the rail corridor

The rail corridor lies in a complex structural depression. The Avoca Fault (west) and Esk Fault (east) are roughly parallel to each other, and to the railway line itself (Figure 3-8). The downthrown eastern and western quadrants of the Avoca and Esk Faults, respectively, meet in the centre and form a linear depression up to 1500 m wide. The Torlesse Fault Zone cross cuts both the Avoca and Esk Faults and is approximately 1.0 km south of Slovens Creek Viaduct.

With a total structure length of approximately 35 km, the steeply inclined (75°) Esk Fault is currently considered inactive at Avoca, although the Northern Esk Fault is active some 17 km away in the northeast (Figure 3-2, B). At Avoca, Forsyth et al. (2008) have noted reverse faulting on the Esk Fault during the Pliocene and Quaternary, with total reverse slip ranging from 1 to 10km. It is suggested by McLennan and Bradshaw (1984) however, that given the pre-Oligocene angular unconformity across the basin and postulated uplift in the east, the Esk Fault may have been active in the early Tertiary and not only in the Quaternary (Figure 3-5). The Esk Fault is particularly well expressed on the true left side of Slovens Stream, whereby preferential incision the stream is aligned to the fault, resulting in a steep face of Rakaia Terrane near the junction of Slovens Stream and Broken River.

The Avoca Fault is approximately 6km in length (Figure 3-8), and therefore somewhat smaller than the Esk Fault. It is obscured by Blackwater glacial outwash surfaces for the most part. Currently deemed inactive, a range of 0.1 – 1km of reverse slip is assigned to the fault, with the activity dated to the Pliocene and (?) early Quaternary (Forsyth et al., 2008). Despite the Quaternary cover, distinct linear features can be observed on foot and in aerial photography, and other faults active in the last 20ka may be present.

Figure 3-8 Faults within, and surrounding, the rail corridor. Note the syncline marked as a curved white line with inward pointing arrows southeast of the 'Fault'.

An active portion of the Torlesse Fault Zone truncates both the Esk and Avoca Faults with an oblique trend and a normal sense of slip (Figure 3-8).

A small unnamed fault, approximately 1km in length, is of a similar trend to the Torlesse Fault and bisects the Avoca and Esk Faults (Figure 3-8). This unnamed fault is recognised by Gage (1970) and McLennan (1981) as a normal fault.

A fold structure is observed in the Slovens-Avoca corridor, however the exact location of this folded structure is uncertain. Gage (1970) suggests a folded syncline in the lower end of the rail corridor at the mouth of Slovens Creek (Figure 3-8), whereas McLennan (1981) suggests that the axis of the syncline is parallel to the centre of Bryce Gully. Gage (1970) argues that the structural complexity of the Tertiary strata Avoca is due to the gravitational sloughing of cover beds, which occurred during the uplift of the Torlesse Ranges.

3.6.2 Lithology of the Rail Corridor

Of the Tertiary sequence outlined in Section 3.3, only the basement Rakaia Terrane, Puffer Formation, Thomas Formation tuff, and the Enys formation are found in the rail corridor (Figure 3-2; Figure 3-3).

**Rakaia Terrane**

Rakaia Terrane encompasses the eastern abutment of Slovens Creek Viaduct (Figure 3-9 A), and is separated from Tertiary strata by the Esk Fault. At the western abutment, the Rakaia Terrane rocks are well indurated closely jointed greywacke. Beds of indurated argillite and dark grey siltstone are present near Avoca. The Torlesse Supergroup is intensely crushed (at cm scale) and sheared on the up-thrown block of the Esk Fault, true left side of Slovens Stream, and is slightly to moderately weathered at the surface (Figure 3-9 B, C, D).

An abrupt decrease in the elevation of Craigieburn Road, some 800m south of Avoca, denotes one location of a transition between indurated Rakaia Terrane and the relatively soft Puffer Formation. The formation contact is sharp and can be viewed in Slovens Stream. The contact is projected to cross the railway, and is constrained by small outcrops of both the Puffer Formation and Rakaia terrane rocks in a cut batter of the MDL, some 150m south of the 72km metrage mark (refer to Map 2).

**Puffer Formation**

Gage (1970 p. 519) defines the Puffer Formation as “massive argillaceous or silty limestone and glauconitic calcareous sandstone”. The Puffer Formation outcrops at Slovens Creek Viaduct, and is overlain by ‘slumped’ Thomas Formation at the base of the steeply inclined viaduct outcrop (Figure 3-10 A, B, and C). McLennan and Bradshaw (1984) postulate that the Puffer Formation differs from the Coleridge Formation as a result of being closer in proximity to pre or earliest Oligocene erosion and uplift near the Puketeraki Ranges, as discussed in Section 3.4. Uplift in the eastern extension of the
Castle Hill Basin is hypothesised on the basis of a decrease in the magnitude of angular discordance toward the west, and a decrease in the coarseness of grains from east to west (Figure 3-3; Figure 3-5). Stripped from Rakaia Terrane during erosion, the basal conglomerate of the Puffer Formation (member P1; below) comprises rounded greywacke clasts, which not found in the Coleridge Formation of the west. In addition, the Puffer Formation comprises a higher argillaceous content than the Coleridge Formation (Gage, 1970; McLennan, 1984). Further to the work of Gage (1970), McLennan (1981) divided the Puffer Formation into 4 members, as follows:

Figure 3-9  A. Eastern abutment of Slovens Creek Viaduct (photograph is looking to the south) and portal of Tunnel 16 in Rakaia Terrane uplifted by the Esk Fault; B. Tunnel portal and railway for scale of photo C; C. Slightly to moderately weathered Rakaia Terrane northeast of the tunnel portal; D. Angular clasts of Rakaia Terrane from weathering and erosion of joint blocks.
Figure 3-10: A, Looking southeast toward the Thomas Formation Tuff (brown) 'slumped' at the base of the Puffer Formation (pale greyish green) outcrop, west of Slovens Creek Viaduct; B, Looking west-northwest at Slovens Creek Viaduct. Note the persistent discontinuities within the Puffer Formation, parallel to the incline of the outcrop (indicated by the orange dashed line); C, west-southwest view of the Thomas Formation Tuff 'slumped' at the base of the Puffer Formation outcrop. Note the variation in bedding orientations of the Thomas Formation Tuff (indicated in white); D, weathered discontinuity on the face of the Puffer Formation outcrop. Note the 'flaky' surface related to the silt and argillite content. Compass for scale.
1) Member P1 varies between a clast-supported pebble conglomerate with 10% to 40% sand, and a bimodal very coarse to very fine sand. The consolidated member is grey or pale brown with some glauconite and carbonate cement. Remains of fossils such as Panopea and Ostrea are also common.

2) Member P2 is a fossiliferous carbonate-cemented glauconitic litharenite. The member is mainly pale greyish-green and indurated, with crossbedding and concretionary bands between 0.2m and 0.4m thick.

3) Member P3 can be found in Puffers Stream, in the norther part of the field area (Figure 1-6), and is a soft glauconitic sand that is massive in structure and brownish green in colour. Member P3 shows two noticeable dark green bands of highly glauconitic sediments at the base, and becomes indurated by carbonate cement at the top.

4) Member P4 is the only member outcropping in the rail corridor. Member P4 is described as indurated calcilutite, which is white to pale greenish grey and is silty (Figure 3-10 D). The base of the member is moderately glauconitic, with glauconite grains and foraminifera found throughout. At the western end of the Slovens Creek viaduct, this member is well exposed as a slightly weathered, pale greenish cream face, with widely-spaced joints and an average dip of 60° to the northeast (Figure 3-10 A).

Approximately 1.4km due south of the viaduct outcrop, on the south bank of Broken River, the corresponding member P4 can be observed successively overlying the younger members (P1-P3), with P4 at an elevation of approximately 650 masl. At the viaduct abutment, the lower contact of P4 is obscured, however the upper contact with Thomas Formation tuff appears sharp at 485 masl.

Petrographic microscopy shows the Puffer formation to comprise very fine-fine sub-angular quartz grains and glauconite pellets, with minor clays, detrital clinopyroxene, biotite, and feldspars (Figure 3-11 C). Voids and veins are filled with chlorite or calcite (Figure 3-11). The description of indurated calcilutite, provided by McLennan (1981), is very detailed and correct, however for ease of terminology the Puffer Formation Member P4 will simply be referred to as limestone for the engineering purpose of this study.

**Thomas Formation Tuff**

Gage (1970, p. 521) defines the Thomas Formation as “a sequence of hard buff or cream-coloured limestones alternating with basaltic tuff and conglomerate”. As mentioned previously, the types, colour and bedding orientations of the Thomas Formation tuff units are highly variable. McLennan (1981) assigned three members to the Thomas Formation Tuff, where it had previously been considered ‘impracticable’ by Gage (1970) due to lateral variation and intermittent exposure of the sequence.
Nonetheless, McLennan (1981) found the first (and lowest) tuff member to be coarse, poorly sorted, and sandy specifically near the top (Figure 3-10 C).

A second type of tuff shows occasional red and green beds, but is primarily pale brown through to black. The beds of this member are agglomeratic to bedded (McLennan, 1981; Figure 3-12 A, B, D). A third type of tuff is massive and black or dark grey (Figure 3-12 B). Angular volcanic clasts and basaltic minerals, such as glass and clinopyroxene, are present (Figure 3-12 B; 3-13 B, C; McLennan, 1981).

The Thomas Formation is weak and weathered at the surface and some units show shrinkage cracks (Figure 3-12, C), which suggests the presence of clay minerals. Microscopic inspection confirms clays, such as illite, are present, however x-ray diffraction would be required to determine clay percentages. Microscopic study suggests an approximate mineral assemblage of quartz, plagioclase, clinopyroxene, amorphous volcanic glass, and (some) calcite (Figure 3-13).

Enys Formation

Distinctly different from any other formation in the rail corridor, Gage (1970, p.527) defines the Enys formation as “shell-beds succeeded by alternating blue green silts and sands, concretionary in places
Figure 3-12 A, Thomas Formation Tuff exposed on the true left of Slovens Stream, approximately 150m northeast of metrage mark 70.369 (refer to Map 4 or 5). Note the variation in orientation of the bedding in the Tuff; also note the green bands described in text; B, Thomas formation outcrop found southwest of the railway line at metrage mark 70.369. This is the same member as in photograph A, and is situated almost directly opposite, however the beds are near vertical in this location. Also note the third member (massive and black or dark grey) above the orange dashed line; C, Thomas Formation Tuff displaying shrinkage cracks caused by the clay content. This feature is also found at metrage mark 70.369; D, zoomed in image of an ‘agglomeratic’ bed within the outcrop of photograph B.
and crowded with molluscan shell fragments, grading upwards to estuarine sediments composed of grey sands with carbonaceous silt laminae”.

In Slovens Stream, coal seams of up to 1m thick appear as thinly laminated muddy lignites within a sandy silt unit near the base (Figure 3-14 C), and are succeeded by a basalt conglomerate with a sharp contact. Grading upwards, a muddy sandy silt of grey to pale green occurs. This unit is calcareous with 0.4m bands of packed molluscs (Figure 3-14, A). The remainder of the calcareous silt is generally massive.

The only contact with the Thomas formation noted by Gage (1970) and McLennan (1981) is faulted and was observed in Slovens Creek, near the mouth of Bryce Gully. This contact appears to have been obscured by landslide material and could not be located. To the west of the railway, in the lower part of Bryce Gully, silts and lignites can be observed in the stream bed. Further up the gully a large portion of the grey green silt has been eroded out and, as opposed to the typical V shape carved by water, the gully form is much more subdued (Figure 3-14 B). The outcrops here have undergone significant erosion, taking on a pattern that can best be described as a ‘badlands’.

The Enys formation is overlain unconformably by Blackwater I and II glacifluvial deposits, clearly visible as the grey mass truncating the Enys beds in Bryce Gully (Figure 3-14 A).
3.7 Quaternary Geology and Geomorphology of the Rail Corridor

3.7.1 Geomorphology

The Slovens Stream valley, in which the Slovens-Avoca corridor lies, was carved out by ice during the last glaciation. As discussed in Section 3.5, the Slovens Creek lobe of the Waimakariri Glaciation was
a major distributary (Figure 3-6; Figure 3-7), which retreated at approximately 19ka, during fluctuations of the Waimakariri Glaciation (Gage 1958; Rother et al., 2015).

Adjacent to the rail corridor, glacial landforms in the west include kettle-hole lakes, kame terraces, and outwash terraces. Kame terraces can be found in the upper reaches of Slovens Stream, and approximately 2km to the southeast, along the path of Slovens Stream, ablation moraine fields are present (Figure 3-6). Post-glacial deposits, such as peat and flood plain alluvium, occupy Slovens Stream itself for the most part (Figure 3-6; Gage 1958).

Glacial outwash terraces lie either side of the rail corridor. To the east, (Mid) Late-Blackwater outwash terraces up to 620masl overly Rakaia Terrane, which has been uplifted by activity on the Esk Fault (Figure 3-6; Figure 3-8). To the west, (Mid) Late-Blackwater deposits overly the Tertiary geology of the rail corridor, and are up to 600masl (Figure 3-6). The lower formation contact of the glacifluvial deposits is traceable in the hillside by way of vegetation growth; that is, where the surface water readily percolates glacifluvial deposits, and drains at the surface of Tertiary deposits, vegetation growth is increased and defines a linear trace. The linear trace is attributed to the lower contact of the glacifluvial deposits with Tertiary strata (Figure 3-15).

Figure 3-15 Photograph to show the growth of vegetation below the postulated contact of the glacifluvial outwash terrace (highlighted by the red dashed line). Photograph is looking toward the western slope of the Slovens-Avoca corridor, the Torlesse Range is in the background.
3.7.2 Landslides

Along the length of the Slovens-Avoca corridor, approximately 3km, at least 90 percent of the surface has been subject to mass movement, with particular landslides more significant than others. The landslides of the Slovens-Avoca rail corridor are clearly younger than the last glaciation, and it would appear that Slovens Creek has been downcutting since the last glaciation. The extent of the deep seated landslides in the corridor appear to extend to Slovens Stream, and in some cases have caused a meander in the Stream; that is, due to the incision of Slovens Stream material has failed and obstructed the initial path, and has therefore redirected the flow. In addition, it should be noted that where surface water percolates through the gravels into the weak Thomas Formation Tuff, increased water content provides favourable conditions for landslides, especially within the clayey layers of the Thomas Formation.

Within the scope of this study, it has not been possible to date the landslides, apart from being younger than the Waimakariri Glaciation (17ka). In this study, the relative age of landslides are determined using cross cutting relationships, geomorphic expression, and vegetation cover. The following two chapters characterise the landslides, and provide detailed engineering geomorphology maps as a part of the ground model for the rail corridor (Chapter 4), and separately of the Slovens Creek Viaduct (Chapter 5).

3.7.3 Engineered and Non-Engineered Fill of the Rail Corridor

Non-engineered Fill

Non-engineered fill has been mapped beneath and alongside the Slovens-Avoca rail corridor. In this study, non-engineered fill comprises clay and gravels, which are derived from landslides, dumping of landslide debris, and clearing the track following slip events.

Engineered Fill

Engineered fill is not mapped in this study, however engineered fill refers to the ballast that exists beneath the tracks. The railway ballast is designed in accordance with KiwiRail specification 140, which refers to various New Zealand Standards (NZS). In general, the ballast is a nominally sized granular material that meets the NZS with regard to hardness, quality, and angularity, and is free from harmful substances or dust.

3.8 Summary

- Rakaia Terrane is extensive and denotes the basement rock of the Castle Hill Basin, and Slovens-Avoca Corridor.
- Periodic uplift and faulting, namely to the east of Slovens-Avoca corridor along the Esk Fault, has resulted in Tertiary strata lying in faulted contact with Rakaia Terrane throughout the Castle
Hill Basin. There are additional faults, such as the Avoca and Unnamed Fault, which appear to disrupt surfaces in the valley.

- Of the Tertiary sequence found in the Castle Hill Basin, only the Puffer and Thomas Formations of the Porter Group, and the Enys Formation of the Motunau Group are present in the Slovens-Avoca Corridor. Late Cretaceous-Eocene Formations have been stripped from the eastern extension of the Castle Hill Basin as a result of uplift and erosion in the early-Oligocene.

- Due to being obscured by vegetation, outcrops of Tertiary strata are limited. The Puffer Formation is well exposed at the western abutment of Slovens Creek Viaduct (Figure 3-10). The Thomas Formation Tuff is best exposed at the abovementioned location, and at metrage mark 70.369, both in Slovens Stream and to the west of the railway line (Figure 3-12). The Enys formation is best observed in Bryce Gully, metrage mark 70.722, and in Slovens Stream at the mouth of Bryce Gully (Figure 3-14).

- Glacial deposits of the Waimakariri Glaciation are extensive across the Castle Hill Basin, and (Mid) Late Blackwater outwash terraces, as dated by Rother et al. (2015), lie either side of the rail corridor at elevations up to 620masl. The basal contact with the Enys Formation is observable in Bryce Gully (Figure 3-14 A).

- Slovens Creek Valley was carved out by the Slovens Creek lobe during the Waimakariri Glaciation, and landslides within the valley post-date that glaciation, however the age cannot be further constrained.

- Landslides occupy approximately 90 percent of the slope to the west of the Slovens-Avoca corridor. The majority are found within, or are related to, the clayey, weak Thomas Formation Tuff, and are mapped and characterised in the following two chapters.
4 Ground Model of Slovens-Avoca Rail Corridor

4.1 Introduction

This chapter presents a ground model for the Slovens-Avoca rail corridor using the geology (‘A’ series) and engineering geomorphology (‘B’ series) maps compiled for this project. Maps 1A and B through to 5A and B are covered in this chapter with map excerpts throughout, and the unabridged maps are located at the back of this thesis (refer to Figure 2-3 for map layout). Maps 6A and 6B are presented in Chapter 5 as part of the detailed site investigation and stability modelling carried out for the Slovens Creek Viaduct western abutment.

‘A’ series geological maps, numbers 1-5, present the rail corridor geology and utilise the stratigraphy of Chapter 3, Section 3.6.2. The maps include rail corridor infrastructure, such as metrage and cut batters. Two cross sections for the Slovens-Avoca rail corridor are presented, with the lines of section shown on ‘A’ series maps. The line of section A-A’ can be found on Map 2A; the line of section B-B’ can be found on both 3A and 4A because of overlap in the map coverage (Figure 2-3).

‘B’ series engineering geomorphology maps detail landslides along the rail corridor, which are grouped into numbered areas to allow for succinct description and discussion in the text. The numbered areas are outlined on three-dimensional hill shade models shown in each section of this chapter. It is intended that the appropriate map is examined alongside the text of each map section.

The final section of this chapter presents a generic ground failure model for the mapped landslides along the rail corridor, and discusses the processes that have led to numerous failures adjacent to the Slovens-Avoca rail corridor. In the first instance however, engineering geology descriptions for the soil and rock of the rail corridor are provided in section 4.2.

4.2 Engineering Geology Descriptions

Geological formations in the rail corridor typically comprise weak rock or soil. The engineering soils identified in the rail corridor are generally the product of weathering and slaking of the less resistant rocks, such as the clay-rich Thomas Formation Tuff. The Puffer Formation comprises an engineering soil, which is the friable sand of member P3, however the properties of this soil are related to the depositional environment, rather than being the result of weathering. Where there is soil and rock within a formation, whether due to weathering or deposition, two descriptions are provided for that formation. The first description describes the rock, and the second describes the soil, as indicated in brackets at the end of each description.
4.2.1 Rakaia Terrane Rocks – Torlesse Supergroup

Rakaia Terrane rocks are present in the rail corridor both near Avoca, where argillite beds are found, and near Slovens Creek Viaduct, where there is a faulted contact with Tertiary strata as described in Chapter 3, Section 3.6.2. The engineering geology description of the Torlesse Supergroup is below.

*Unweathered to moderately weathered, light to dark grey and light purplish grey, moderately thin subhorizontal to subvertical bedded SANDSTONE. Strong to moderately strong; closely spaced joints, narrow to tight. [Rakaia Terrane, TORLESSE SUPERGROUP]*

4.2.2 Puffer Formation

The four members of the Puffer Formation are discussed in Chapter 3, Section 3.6.2, however only members P3 and P4 are found outcropping in the Slovens-Avoca rail corridor. Engineering geology descriptions are given below for P3 and P4, and as already discussed member P3 requires a soil description given the friable sand, and the very weak nature of the sandstone.

*Moderately weathered, light green, massive SANDSTONE. Weak to very weak; joints are widely spaced and non-systematic. Friable. In part, glauconitic. [PUFFER FORMATION, P3 - rock]*

*Fine to coarse SAND; light greenish grey, massive. Moderately dense, dry. [PUFFER FORMATION, P3 – soil]*

*Slightly to moderately weathered, light brownish white to light green, very fine massive LIMESTONE. Moderately strong; joints are steep, non-systematic, and closely to widely spaced. [PUFFER FORMATION, P4]*

4.2.3 Thomas Formation

The Thomas Formation Tuff is divided into three members by McLennan (1981), as discussed in Chapter 3, section 3.6.2. The Thomas Formation comprises clay-rich units throughout, and requires a soil description in addition to the rock description, given the plastic and cohesive behaviour of parts of the unit (Figure 3-12C).

*Slightly weathered to moderately, dark brownish red, massive argillaceous TUFF. Very weak; very closely spaced joints, narrow to open. [THOMAS FORMATION Lowest Member - rock]*

*Moderately weathered, dark greenish black, moderately thin subvertically bedded ‘agglomerate’ TUFF. Moderately strong; very closely spaced subhorizontal discontinuities. Angular basalt clasts up to 300mm, with some carbonate cement. [THOMAS FORMATION Second Member - rock]*
Highly weathered to completely weathered, blackish green to light green and reddish brown, thinly to moderately thickly bedded TUFF. Very weak to extremely weak, moderately widely to widely spaced joints. [THOMAS FORMATION Third Member- rock]

Argillaceous SILT, dark brown or dark greenish grey, soft, wet, medium to high plasticity, sensitive. [THOMAS FORMATION TUFF– soil]

4.2.4  Enys Formation

The Enys Formation differs from the marine Puffer and Thomas Formations because it grades upward into estuarine deposits, as described in Chapter 3, Section 3.6.2. The lower silty units of the Enys Formation, found in the bed of Slovens Stream and the lower part of the Bryce Gully Stream, have a lower sand content than the upper units, which are estuarine deposits. The latter are found near the head of Bryce Gully, and display a badland erosional pattern (Figure 3-14A).

Moderately weathered to highly weathered, blue grey, trace mottled orange and speckled black, laminated SILTSTONE. Weak to very weak. Minor carbonaceous material. [Lower Bryce Gully Enys Formation Siltstone]

Highly weathered, light greyish green, moderately thick to very thick bedded SILTSTONE. Weak to extremely weak; non-systematic discontinuities. [Upper Bryce Gully Enys Formation Siltstone]

Moderately weathered, brownish grey to greyish green, massive SANDSTONE. Weak-very weak; widely spaced discontinuities. [Upper Bryce Gully Enys Formation Sandstone]

Weathering and rapid slaking of the Tertiary rocks in the Slovens-Avoca rail corridor has led to the accumulation of regolith materials on the slopes. Therefore the landslides described in the following text are referred to as ‘earth’ or ‘debris’ slides as per the landslide terminology of Cruden and Varnes (1996; Chapter 1, Section 1.5).

4.2.5  Quaternary Glacifluvial Deposits

Quaternary glacifluvial deposits known as the Blackwater Outwash Terraces overlie the Tertiary strata of the rail corridor, and are described below.

Sandy fine to coarse GRAVEL with minor silt and clay; light greyish brown, bedded. Loosely packed to medium dense; dry; well graded; bedding, subhorizontal, thick; subangular to subrounded greywacke, fine gravel to boulders; sand, fine to coarse; silt and clay, slightly plastic. [BLACKWATER OUTWASH TERRACES].
4.3 Map 1

4.3.1 Map 1A - Geology

The area of Map 1A is largely underlain by Rakaia Terrane rocks, however the Puffer Formation is found either side of the rail corridor, where it is overlain by Quaternary gravels. The sandy soil unit of the Puffer Formation, member P3, outcrops on the true left bank of Slovens Stream approximately 200m northeast of metrage mark 72.380, and again on the true left bank some 50m upstream. The Puffer Formation is not mapped in such close proximity to Avoca by Gage (1970) or McLennan (1981), however their work does show the Puffer Formation underlying the same glacifluvial outwash surface as in the east of Map 1, but only in Puffers Stream approximately 300m to the southeast. Puffers Stream is not included in the field area, but is on Gage’s (1970) map and is shown in Figure 1-6. Given the presence of the Puffer Formation under the Quaternary outwash surface to the east of the rail corridor, Slovens Stream and Puffers Stream (Figure 4-2), it is assumed that the sandy soil unit of the Puffer Formation third member (member P3) is also present beneath the Quaternary gravels on the slope to the west of the rail corridor (Area 1, Figure 4-1; Figure 4-2). The assumption is made on the basis that the Puffer Formation was continuous across Slovens Valley prior to fluvial incision Slovens Stream.

4.3.2 Map 1B – Engineering Geomorphology

Water

Channelled surface runoff from the valley and alluvial fan south of Area 1 (Figure 4-1; Figure 4-2), causes the ground to be visibly wet beneath the ballast of the railway line, and on Craigieburn Road. A culvert is in place at metrage mark 72.380, which directs the surface water to Slovens Stream, however there is additional flow approximately 30m north on the same alluvial fan. This path of water is not properly drained by the culvert, and the water is entering the rail corridor north of the culvert, and north of the cut batter. Track logs do not indicate any disruption at track level, and the presence of fluvial fans, which comprise highly permeable gravels, suggest the ground is stable for this section of track despite water being present at track level.

Landslides of Area 1

Hummocky ground to the west of the railway (Area 1, Figure 4-1; Figure 4-2) is attributed to failure of the Puffer Formation. With the assumption that the underlying material is the soil member P3 of the Puffer Formation, as discussed in Section 4.3.1, the geomorphic expression suggests the failure to be successive rotational earth slides, or simply a complex earth slump (Figure 1-7). Given the underlying geology, and that single rotational failures, or slumps, are common in homogenous engineering soils (Cruden and Varnes, 1996), the latter is probably more likely. Although there are no cross-cutting relationships to determine a relative age for this landslide, the failure has occurred between two fluvial
(possibly glacifluvial) fans as shown in Figure 4-2. The fans appear to overlie, or truncate, the hummocky ground, and therefore suggest that the failure is older than the aggradation of the fans.

Figure 4-1 Three-dimensional photography of Map 1. See the model below for the relief of Landslide Area 1 (outlined by the green oval), which is over darkened by hill shade in the photography.

Figure 4-2 Shaded relief model of Map 1. Note the outwash fans and hummocky ground to the west of the railway referred to in the text. Also note the diagrammatic interpretation of Puffer Formation P3 geology beneath the Quaternary terrace as described in the text. The direction of Puffers Stream is indicated, and although it is not within the mapped field area, it can be found on Gage’s (1970) map (Figure 1-6, this thesis).
4.4 Map 2

4.4.1 Map 2A – Geology

As discussed in Chapter 3, Section 3.6.2, a contact between Rakaia Terrane rocks of the Torlesse Supergroup and the Puffer Formation is found in Slovens Stream, approximately 200m south-southeast of metrage mark 72.000. The contact exposed in Slovens Stream is unconformable and sharp, with a strike of 140° and a dip of 50° to the southwest, and may be the result of a minor fault. A cut batter, 50m north of metrage mark 71.681, exposes both the Torlesse and the Puffer Formation in the northern and southern ends, respectively. The outcrops here provide a good constraint for the projection of the Torlesse- Puffer Formation contact from Slovens Stream across the railway line, however the contact itself is obscured by vegetation in the cut batter. The Puffer Formation, exposed in the southern end of the cut batter, comprises a carbonate bed approximately 0.3m thick, from which a similar strike and dip to the contact in Slovens Stream was measured (146/50SW). Torlesse Supergroup rocks at the northern end of the cut batter were too weathered to obtain accurate structural measurements, however the structural measurements of Torlesse rocks in Slovens Stream show the strike and dip to be highly variable, ranging from 070/85E to 100/60S. Variation in the orientation of the beds within the Torlesse Supergroup is attributed to faulting and folding (Gage, 1970) resulting in small block failures typically ≤10m³.

Approximately 400m downstream of the Torlesse-Puffer Formation contact, there is an obscured contact between the Puffer Formation and the Thomas Formation Tuff. Although obscured, the formation contact can be constrained by a change in the lithology of outcrops on the banks of Slovens Stream, and can be projected to be between metrage marks 71.510 and 71.386 where there is a Thomas Formation Tuff outcrop west of the railway line, with a strike of approximately 150 (Map 2A).

4.4.2 Engineering Geomorphology - Map 2B

Water

Water in the section of track covered by Map 2 is managed by three culvert drainage systems at metrage marks 71.681, 71.510 and 71.386. At metrage mark 71.681 the culvert is adjacent to what appears to be an engineered drainage structure. The structure comprises two vegetated oblong ‘piles’ of gravel, and an excavated ditch behind to drain and direct water from the slope into the culvert (Figure 4-3). It is likely that landslide deposits have been excavated and placed into oblong piles to improve drainage for the pronounced landslide on the adjacent slope (Figure 4-4).

The culvert at metrage mark 71.510 drains the area of the slope in close proximity to the contact between the Puffer Formation and the Thomas Formation Tuff. Approximately 120m south another culvert at
Figure 4-3 Aerial photograph of the engineered drainage structure referred to in the text. The 'X' denotes the location of the culvert, while the number provides the metrage. The location of this structure relative to the remainder of the area of Map 2 is indicated in Figure 4-4 below.

Figure 4-4 Aerial orthophotography of the area mapped for Map 2. Note the location of the photograph of Figure 4-3, and metrage mark 70.681 indicated by the yellow arrow. The tension cracks at the surface of the Quaternary gravels (Figure 4-1) are clearly visible, as is the hummocky geomorphology that extends to that level.
metrage mark 71.386 drains water from the slope mainly comprising Thomas Formation tuff. As the lithological transition into the clay-rich Thomas Formation Tuff occurs, more drainage is required due to an increase in groundwater and surface run off given the relative impermeability of the clay-rich tuffaceous materials.

Track logs show discrepancies in the sinuosity or curvature of the track, known as line faults (Chapter 1, Section 1.6.2), at the approximate metrage of 70.700, and near the drainage structure. Two line faults are recorded on 20 July 2014, and one on 20 March 2014. Given the engineered structure at this location, the line faults are likely due to minor changes in the ground, which is probably landslide debris (gravels, sand, and clay), as a result of erosion from improperly drained water at the culvert and drainage system.

**Landslides of Area 2**

Area 2 (Figure 4-5) comprises two types of mass movement. The first is considered a ‘multiple rotational retrogressive debris-earth slide’. The underlying geology of the slope is reflected in the geomorphology of this failure, where the friable sandy member of the Puffer Formation underlies the very hummocky geomorphic expression in the lower part of the slope beneath the Thomas-Puffer Formation contact, and the relatively cohesive clay-rich Thomas Formation Tuff displays a smooth ‘blocky’ geomorphic expression (Figure 4-5). The smooth blocky form of the main headscarp and minor scarpds within the slide, and above the Thomas-Puffer contact (Figure 4-5), is attributed to back-tilting rotated blocks (Figure 4-6).
Figure 4-6 Cross section A-A' of Slovens - Avoca Rail Corridor showing an interpreted failure surface for the multiple rotational landslide of Area 2. The geometry of the landslide is schematic and is based on the geomorphic expression, an idealised rotational landslide model, and the ratio $D/L$, given by Cruden and Varnes (1996) as discussed in text. Note the back tilting of the upper blocks indicated by the black arrows in the upslope direction.
A retrogressive landslide is defined as the ‘surface of rupture extending in the direction opposite to the direction of movement of the displaced material’ (Cruden and Varnes, 1996, p. 47). Based on the minor scarp, tension cracks, and retrogressive nature of the slide, potential surfaces of rupture are illustrated in Figure 4-6.

The depth and form of the failure surface, or surface of rupture, is not known for this landslide, however reasonable approximations have been made based on the geomorphic expression, assumed geology, an idealised model for rotational failure, and an empirically-derived ratio for rotational failure in engineering soils from Cruden and Varnes (1996). The ratio given is $D_r/L_r$, where $D_r$ is the depth of surface of rupture, defined as the maximum depth of surface of rupture below original ground surface (Figure 4-6), and $L_r$ is the length of surface of rupture, defined as minimum distance from the toe of the surface of rupture to the crown (Figure 4-6). Calculated values from the ratio $D_r/L_r$ would typically range from 0.15 to 0.33 for a rotational failure in engineering soil (Cruden and Varnes, 1996). Figure 4-6 illustrates where the measurements were taken, and in this case the ratio $D_r/L_r$, is equal to 51m over 225m, which gives a value of 0.22. This suggests the postulated geometry of the multiple rotational landslide in Figure 4-6 is appropriate.

The extent of the toe, or zone of accumulation (Figure 4-6), of this slide is also uncertain due to the previously discussed anthropogenic alteration of landslide deposits at metrage mark 71.681 (Figure 4-3). Large greywacke boulders, up to 1m diameter, suggest landslide deposits reach at least as far as mapped on Map 2B, however it is possible that this map provides an underestimate of the true extent of this particular slide because the deposits have been excavated and reworked. As discussed in Section 4.4.1, the Puffer Formation bedrock is observed in the cut batter 50m north of metrage mark 71.861, which is immediately east of the toe of the multiple rotational landslide. Structural measurements (approximately 150/50SW) obtained from the Puffer Formation in the cut batter are consistent with measurements taken from the Puffer Formation on the true left bank of Slovens Stream, and therefore landslide deposits are not considered to be present in that cut batter.

The overlying debris flow in Area 2, detailed in Figure 4-5, is younger than the rotational slide, with the source area in the Quaternary glacifluvial deposits. The main path and depositional area of this debris flow are visible in aerial photography (Figure 4-4; Figure 4-5). A subdued expression, and similar amount of vegetation cover as the remainder of Area 2, suggest this landslide is only slightly younger than the rotational slide. Landslides of area 2 are considered dormant, defined as ‘an inactive landslide that can be reactivated’ (Cruden and Varnes, 1996), because the underlying geology is weak clay-rich tuff. Given the measures taken to manage water already present, prolonged rainfall could increase overland and groundwater flow and act as a trigger during exceptionally wet periods.
Landslides of Area 3

Area 3 comprises a group of three landslides (Map 2B; Figure 4-5). The first slide of Area 3 is the most northern, which is approximately 100m southwest of metrage mark 71.681, and its recognition as a debris flow is based on the geomorphic expression and the identified source area. The source is the Quaternary gravels, and compared to the other landslides within Area 3, this debris flow has a relatively large source area. The runout of this flow certainly extended across the railway line, and probably reached as far as the location of Craigieburn Road. Mounds of landslide debris comprising gravel and boulders derived from the Quaternary deposits are found on both sides of the track. The youngest slide of Area 3 appears to predate rail construction, and as determined by cross-cutting relationships, this slide is interpreted as the oldest. Therefore it is most likely that the landslide debris of the mounds was excavated and placed there in order to lay the railway infrastructure on a flat surface, rather than the mounds being the result of clearing the track following a recent landslide.

The landslide immediately to the south is in the middle of those in Area 3, and is located immediately west of metrage mark 71.510. This failure is considered to be a composite debris flow-earth slide. The source of the debris-flow component appears to be the Quaternary gravels, and the earth slide component has occurred near the postulated formation contact between the Puffer Formation and the Thomas Formation, with the headscarp in the clay-rich Thomas Formation. The geomorphic expression of this debris flow earth-slide is less subdued than the landslide further south, and this slide is therefore considered younger.

The final, southernmost landslide of Area 3 is west of metrage mark 71.386. This failure is again classed as a composite debris flow-earth slide, with the slide component again occurring in the clay-rich Thomas Formation Tuff. This landslide is the least subdued of Area 3, and is therefore considered the youngest. Geomorphic expression suggest that the primary failure and the debris flow of this slide predate construction of the rail, however track logs suggest that the earth slide component within the Thomas Formation is active, for reasons discussed below.

At metrage mark 71.400, the track logs show a total of five lateral geometric changes (three on 31 July 2014, one 20 March 2014, and one 30 July 2013), which are gauge faults, and could be attributed to the earth slide moving east toward Slovens Stream. The midland line is orientated approximately north at this location, and so eastward movement of the earth-slide, as mapped, could laterally deform the track. In addition, three changes in vertical track geometry (two 20 March 2014; one 30 July 2013), which are known as top faults, are recorded. Assuming the top faults record an upward vertical motion, the expected movement at the toe of a rotational slide (outward and upward) can be recognised from the lateral and vertical track deformation.
Figure 4-7 Aerial orthophotography of the Map 3 area. Note the increased vegetation growth in the lower parts of the slope.

Figure 4-8 Outline of landslide areas on Map 3 referred to in text. The line of cross section B-B’ is discussed in Sections 4.5.2 and 4.6.1.
4.5 Map 3

4.5.1 Map 3A - Geology

The Enys Formation dominates as the bedrock geology for most of the Map 3 area. The obscured Thomas-Enys Formation contact is well constrained by two small outcrops west of the rail corridor, between metrage marks 71.301 and 71.240, and by the presence of the Enys Formation mudstone in the bed of Slovens Stream.

Gage (1970) and McLennan (1981) record a faulted contact between the Enys Formation and Thomas Formation on the true right bank of Slovens Stream, approximately 50m south of the mouth of Bryce Gully (Figure 4-7; Figure 4-8), and also note that this is the only Thomas-Enys Formation contact observable in the eastern extension of the Castle Hill Basin. This fault and contact has been obscured by landslide debris since the work of McLennan and Gage, and was therefore not observed in the present study. Approximately 20m south of the mouth of Bryce Gully, the basal shell bed and conglomerate units of the Enys Formation are exposed on the true left bank of Slovens Stream (Figure 3-14A), and dip between 55° and 85° to the east (Map 3A; Figure 4-12). Within the stream bed of Bryce Gully, numerous beds of carbonaceous mudstone and siltstone are exposed, and consistent measurements show these beds to be dipping between 40° and 60° to the west (Map 3A; Figure 4-12). The opposing orientation of beds within the Enys Formation, on the true left of Slovens Stream compared to within Bryce Gully, is attributed to a fault-related fold, which is discussed in conjunction with cross section B-B’ (Figure 4-12) in section 4.6.1.

At the head of Bryce Gully, the contact between Quaternary strata and the Enys Formation is sharp and well exposed at approximately 555masl. The contact is clearly identified by seepage and a subsequent line of increased vegetation growth on the hillside, which is persistent to the north of Map 3 with a gradual increase in elevation of approximately 15m (Figure 3-15; Figure 4-7).

4.5.2 Map 3B - Engineering Geomorphology

Water

Five culverts drain the slopes in this section of track, and there is a significant increase in the amount of vegetation growth in the area of Map 3 compared to the Map areas already reviewed (Figure 4-1; Figure 4-4; Figure 4-7). Water from the culverts at metrage marks 71.301 (draining the flow from Matagouri Gully) and 71.240 (groundwater flow across Craigieburn Road), leave the surface of the road wet in summer and muddy in winter. The culvert at metrage mark 71.041 drains water from the unnamed gully, and surface water is not present during the summer months.
Bryce Gully is the most significant stream in the western slopes of the Slovens-Avoca Corridor, and is drained by the culvert at metrage mark 71.722. It is worth noting that there is a high sediment load from the upper catchment of Bryce Gully, and the diameter of some glacial greywacke boulders (up to 2m) exceeds the width of the culvert at approximately 1m.

Figure 4-9 Landslide Area 4 extracted from Map 3B. The landslides are numbered on this diagram for the sole purpose of clarity during discussion in text, and the full size map (Map 3B) should be referred to for scale and legends. North is at the top of the figure.

Landslides of Area 4

Area 4 comprises four landslides. The two northernmost landslides, immediately west of metrage mark 71.240 (Ls1 and Ls2; Figure 4-9), are a debris flow (Ls1), with gravels as the source material, and a rotational debris-earth slump near the toe of the debris flow (Ls2). The headscarp of the debris-earth slump is within the Enys Formation, while the surface of rupture and the lower part of the debris-earth slump is within the Thomas Formation. The debris flow appears as the older of the two given that the debris-earth slump truncates the postulated runout area of the debris flow (Figure 4-9). The rotational slump is of particular interest here, as it occurs in the clay-rich Thomas Formation Tuff and in the area surrounding metrage mark 71.240, where drained water leaves the surface visibly wet at Craigieburn Road. The toe of the slide is adjacent to Craigieburn Road, and track logs show multiple gauge faults (three on 31 July 2014, one on 20 March 2014, and one on 16 October 2013), and top faults (two on 20 March 2014, and one on 16 October 2013) at this location. The track faults at this location are similar to those observed near the southernmost landslide of Area 3, and west of metrage mark 71.386 (Section 4.4.2), which also occurs in Thomas Formation Tuff and is considered the result of an active rotational
slide. The clay-rich lithology, presence of water, and geomorphologic expression of this slide suggest that the track faults are again the result of an active rotational landslide in the Thomas Formation Tuff.

The southernmost landslides in Area 4 (Ls3 and Ls4, Figure 4-9), northwest of metrage mark 71.041, are classed as a translational debris-earth slide (Ls4) and a debris flow (Ls3). The translational debris-earth slide is apparent in the aerial photography where the original spur has moved downslope towards the unnamed gully (Figure 4-7), and the lithology of this slide is both the Quaternary gravels and the Enys Formation. The debris-earth slide is more subdued in geomorphic expression than the debris flow, and therefore it is considered older. The source area of the debris flow is in the Quaternary gravels, and this debris flow is considered to have occurred as a failure consequent to the debris earth slide due to the removal of support.

**Landslides of Area 5**

Area 5 includes all landslides between the unnamed gully and Bryce Gully stream, and between metrage marks 71.041 and 70.722 (Figure 4-7; Figure 4-8; Figure 4-10). The northernmost landslide of Area 5, and adjacent to metrage mark 71.041 (Ls1, Figure 4-10), displays a pronounced lateral scarp (Figure 4-7), and given the geology and geomorphic expression is considered to be a dormant rotational debris-earth slump. This slump, or single rotational slide as defined by Cruden and Varnes (1996), comprises both gravel and fine soil, and is clearly cross cut by the adjacent slide to the south (Ls2, Figure 4-10). This is reflected in the relative ages assigned to Map 3B. The younger slide to the south and west-southwest of metrage mark 71.000 (Ls3, Figure 4-10) is considered to be a composite debris flow-earth slide, with a single rotational earth-slide at the toe. The composite slide has a debris flow component

![Figure 4-10 Landslide Area 5 extracted from Map 3B. The landslides are numbered on this diagram for the sole purpose of clarity during discussion in text, and the full size map (Map 3B) should be referred to for scale and legends. North is at the top of the figure. Note the three sliding blocks within the large landslide outlined in red, and their active toe slumps.](image-url)
derived from the Quaternary gravels, and in some parts the gravel ‘debris’ visibly overlies the earth slide component, which is within the Enys Formation. The slide at the toe (Ls3, Figure 4-10) cross-cuts the composite slide (Ls2, Figure 4-10) and displays a pronounced crown and headscarp, the form of which suggests that Ls3 (Figure 4-10) is a rotational earth-slump. This assumption is based on the description of characteristics of such a slide by Cruden and Varnes (1996). The earth-slump (Ls3) is considered to be younger than the composite slide it truncates (Ls2), and older than the overlying slide (Ls4, Figure 4-10), which is moving towards Bryce Gully.

The group of eight slides on the true left bank of Bryce Gully, which is made up of seven smaller slides within a large slide (outlined by the red dashed line Figure 4-10), move towards Bryce Gully Stream and are closely related to the slide immediately north (Ls4, Figure 4-10). The largest failure of this group is considered to be a part-translational part-rotational debris-earth slide, and its headscarp is at the spur of the gravels, with the toe extending to Slovens Stream (red dashed line, Figure 4-10). The translational component of the slide is inferred where the original spur appears to have slid downslope and in the direction of Bryce Gully (Figure 4-7) due to incision of the stream. The expression of the slope that has been exposed following this slide is almost planar, and so is characteristic of a translational slide (Cruden and Varnes, 1996). The rotational component of the slide is suggested because the large landslide certainly reaches as far as Slovens Stream, however based on structural measurements and geomorphology (Figure 4-13) the surface of rupture could not do so if it was only planar or slightly concave. The landslide must therefore have a convex failure surface in part (Figure 4-13).

Within the large slide outlined in red of Figure 4-10, subsequent translational sliding has occurred as a series of three back-tilted sliding blocks, which are sliding south-southeast towards the stream of Bryce Gully (Figure 4-10). Due to active incision of Bryce Gully stream, the tilted translational blocks are considered younger than the large slide they overlie (Figure 4-10), but older than the subsequent failure immediately north (Ls4, Figure 4-10) because the orientation of the failure immediately north suggests it to also be the result of the incision of the Bryce Gully Stream. Ls4 is therefore considered to be younger because failure on the true left bank of Bryce Gully is regressing, as a result of fluvial incision.

The majority of landslides on or related to the northern flank of Bryce Gully are considered dormant or suspended; that is, a slide that has moved within the last 12 months but is not currently active. However, the toe of the sliding blocks adjacent to the stream in Bryce Gully are being eroded by fluvial incision (Figure 4-10). On Map 3B these slides have been assigned an ‘undifferentiated’ relative age, but these slides are considered the youngest of Area 5 and are active toe-slumps, which contribute to the high sediment load of Bryce Gully, and could lead to blockage of the culvert at metrage mark 70.722 (Figure 4-10).
Figure 4-11 Orthophotography showing the area of Map 4 and part of Map 3. Note the vegetation growth is linear.

Figure 4-12 Illustration of numbered landslide areas referred to in text.
Figure 4-13 Cross section B-B’. Note the schematic planar surface of the translational part of the landslide at metrage mark 70.722, also note the rotation part of the landslide extending to Slovens Stream.
4.6 Map 4

4.6.1 Map 4A – Geology

As shown in Figure 2-3, there is some overlap between Maps 3 and 4, and also of Maps 4 and 5. Map 4 was created to cover an area of interest on one map, being the length of the unnamed fault and the retaining structure, rather than these features being divided over two maps. The Enys Formation is the underlying geology on the northwest side of the unnamed fault and in Bryce Gully, as discussed in Section 4.5.1, and the Thomas Formation Tuff lies in faulted contact with the Enys Formation to the southeast of the unnamed fault. The unnamed fault is now obscured by landslide debris, however has been observed by both Gage (1970) and McLennan (1981). A linear feature is visible in the slope behind the retaining structure, and this is interpreted as the fault trace. Gage (1970) has assigned the sense of slip of the fault to be normal, however he has not provided any further detail. This study agrees with the work of Gage, and also finds a fault-related fold to be evident in the structural measurements taken from the basal conglomerate, and the basal shell beds of the Enys Formation, on the true left of Slovens Stream (Section 4.5.1; Figure 4-13). Figure 4-13 shows the unnamed fault steeply dipping with a normal sense of slip, and also shows the folding of the Tertiary strata as indicated by the basal bedding of the Enys Formation. The sense of slip on the Avoca Fault is also normal, as assigned by Gage (1970), however the Avoca Fault dips to the east, whereas the unnamed fault dips to the west. Both faults show a very steep dip, which is interpreted as the result of compression that has forced the Tertiary strata to fold adjacent to the main Esk Fault.

4.6.2 Map 4B – Engineering Geomorphology

Water

Culverts and metrage marks north of 70.598 are addressed in section 4.5.2. A seepage is present in the bank west of the culvert at metrage mark 70.598, and originates at the trace of the unnamed fault. There do not appear to be any particular seepages flowing into the culverts at metrage marks 70.456 and 70.427, however water is visible at the surface in the excavated ‘ditch’ on the southwest side of the track. The linear vegetation growth that appears at the basal contact of the Quaternary gravels with the Tertiary strata is still persistent in Figure 4-11, as it was in Figure 4-7, and is indicative of where water infiltrated through the Quaternary gravels seeps onto the slopes due to the permeability contrast.

Landslides of Area 6

Area 6 (Figure 4-12) comprises ten landslides, located on the true right bank and in the head of Bryce Gully, and to the west of metrage marks 70.598 and 70.722 (Figure 4-14). Again, the ‘undifferentiated’ landslides in this area are the youngest, and are classed as active debris-flows or earth-slides. Quaternary gravels at the top of the slope provide the source for the debris flow components, and silts and sands of
the Enys Formation form the earth slides. These landslides are considered active because the slopes are denuded, and Bryce Gully is still evolving by fluvial incision promoting slope instability.

On the true right bank of Bryce Gully, Ls1 of Figure 4-14, two rotational failures have occurred. These single rotational slides are suggested to be successive rotational slides, rather than multiple rotational slides where the surfaces of rupture would coalesce as described by Cruden and Varnes (1996). The smaller landslide appears to have occurred on a separate surface of rupture, and is considered younger because it is due to active fluvial incision at the toe.

Figure 4-14 shows two debris flows, Ls3 and Ls4, west and southwest of metrage mark 70.598, respectively. The source areas for these debris flows are within the Quaternary gravels, and their runout is projected to the true right bank of Slovens Stream, where the unnamed fault is now obscured. At the toe of the debris Ls3, west of metrage mark 70.598, a single rotational earth-slump failure Ls2 (Figure 4-14) has occurred, and the geomorphic expression is a pronounced depression in the slope, with the toe of the slump at Slovens Stream. Gravels and displaced wire fence lines on the steep true right bank of Slovens Stream indicate that the landslides in this area are active. There is a large timber retaining structure in place that has recently been renewed because the previous structure failed due to bowing, and the failure of the tie backs. Photographs of the retaining structure are shown in Figure 4-15. In photograph B, Figure 4-15, the structure appears to be bowing on the true left bank of Slovens Stream, however given the previous failure the retaining structure was replaced in this fashion purposefully.
Landslides of Area 7

Area 7 (Figure 4-12; Figure 4-16) comprises a ‘widening’ translational debris-earth slide, which occurs in the Thomas Formation tuff. ‘Widening’ is defined by Cruden and Varnes (1996) as the lateral spread of the surface of rupture, which is in the direction perpendicular to the direction of mass movement. The translational slide is widening on the north-northwest lateral scarp, and this is reflected in the relative ages assigned to the incremental movement of that lateral scarp on Map 4B (Figure 4-16). The widening translational debris-earth slide is overlain by an active debris flow, the source material of which is the Quaternary gravels. The gravels have been redeposited throughout the area mapped for this slide.
4.7 Map 5

4.7.1 Map 5A – Geology

Thomas Formation Tuff is the main lithology in the area of Map 5. The Tertiary strata of the Slovens-Avoca corridor lie in faulted contact with the uplifted Torlesse rocks, which are located east of the Esk Fault. The Puffer Formation limestone, member P4, is in situ at the western abutment of Slovens Creek Viaduct, and is in contact with the Thomas Formation tuff, which appears to be ‘slumped’ at the base of the relatively strong Puffer Formation outcrop. This particular area (Area 10, Figure 4-18) is further detailed as a part of the Slovens Creek Viaduct ground model in Chapter 5, Section 5.2.1.

On the western side of the railway, structural measurements from bedding of the Thomas Formation Tuff at metrage mark 70.369 were taken (Figure 3-12B), and show the bedding of the Thomas Formation tuff to consistently dip to the southeast at angles of 60° to 80°. The structural measurements of bedding taken on the true left and true right banks of Slovens Stream, approximately 70m east of metrage mark 70.369, are consistent with those west of the railway line, with a dip of approximately 70° to the southeast. South of metrage mark 70.000, and north of metrage mark 69.174, the dip of bedding in the Thomas Formation is shallower than those at 70.369, with dips in the range of 20°–40°. In addition, the bedding between 70.000 and 69.147 dips in the opposite direction to that near 70.369,
Figure 4-17 Orthophotography for the areas of Map 5 and Map 6. Map 6 is discussed in Chapter 5. Note the increased vegetation growth of Ls4, Landslide Area 9, indicated by the orange arrow, and discussed in the text.

Figure 4-18 Outline of numbered landslide areas for Map 5 and Map 6, as referred to in text. Area 7 was discussed as Map 4, and area 10 will be discussed in Chapter 5 as part of the Slovens Creek Viaduct ground model.
which is to the northwest. The change in dip between the two locations is attributed to a folded synclinal structure near the 70.000 metrage mark, as illustrated on Map 4A and in Figure 4-20.

A syncline structure is recognised in the work of Gage (1970), and the measurements taken in this study are consistent with his suggested location for the structure. The curvature of the Puffer Formation limestone at the viaduct, and the presence of the Puffer Formation on the true left of Slovens Stream adjacent to the Esk Fault, support the hypothesis that a complex fold structure exists. As discussed in Chapter 3, Section 3.6.1, the Slovens-Avoca rail corridor lies in a tectonic depression, and is on the downthrown side of each of the Avoca and Esk Faults. Therefore compression, drag induced by faulting, and the tectonic downthrowing of the Tertiary strata in the Slovens-Avoca would explain the curvature and steep dip (approximately 60°) of member P4 at the viaduct.

4.7.2 Map 5B – Engineering Geomorphology

Water

The culvert at metrage mark 70.369 drains at least two seepages, as illustrated on Map 5B. The water is well channelled to the true right bank of Slovens Stream through a ditch. Despite this the ground west of the railway line in Landslide Area 8 (Figure 4-18) is wet, and water pools in the ditch. Little to no water was observed at metrage mark 70.177 during mapping in the summer months, and the ground is relatively dry. Culverts and seepages at metrage marks 69.174, 69.660, 69.639 are discussed as part of the Slovens Creek Viaduct ground model, and are addressed in Chapter 5.

Landslides of Area 8

Area 8 consists of two separate landslides (Figure 4-19A). The first is a debris flow channelled through bedrock outcropping southwest of metrage mark 70.369, and is considered older than the adjacent slide based on geomorphic expression. The larger landslide, south-southeast of metrage mark 70.369, comprises a gravelly debris flow and an earth slump (Figure 4-19A). The debris flow comprises Quaternary gravels, and the earth slump is a failure within the clay-rich Thomas Formation Tuff. Gravels are exposed at the headscarp of this landslide, and it is postulated that the slide is retrogressive and successive. That is, the debris flow of gravels is a consequent failure to the rotational slump of the Thomas Formation downslope, and the two components of this large landslide do not share the same surface of rupture. A contact between landslide deposits (gravels, sands, and clay) and Thomas Formation Tuff is visible on the right bank of Slovens Stream at approximately 480masl (10m below the ground surface; Figure 4-19C), and in the ditch northeast of metrage mark 70.369 (Figure 4-19B). The large volume of this slide probably caused the meander in Slovens Stream, and as applicable to all landslides in the Slovens-Avoca rail corridor, the extent of this failure developed over time (i.e. as a series of events).
Landslides of Area 9

Area 9 (Figure 4-17; Figure 4-18; Figure 4-20) comprises a complex series of multiple rotational earth slides and translational earth slides, which occur in the Thomas Formation Tuff, and collectively share the same extensive head scarp. A pronounced lateral scarp is visible west of 70.177, and denotes the northwest extent of the oldest slide in Area 9 (Ls1, Figure 4-20). Within Ls1 there are two back-tilted blocks, which are attributed to a retrogressive multiple rotational landslide. The regressive nature of this failure has caused subsequent downslope movement of a block from the headscarp at 540masl. At approximate metrage 70.100 (i.e. within Ls1), track logs indicate disruption to the track during the period 07 July 2011 to 16 October 2013, with three line faults, one top fault, and one gauge fault recorded. Given the extent of Ls1, and the toe at the level of Slovens Stream, it would not be unreasonable to suggest that this slide had been moving extremely slowly, causing discrepancies in track geometry due to removal of support by fluvial incision at the toe. Despite the possibility of having
been active in 2013, Ls1 is classed as the oldest on Map 5B because of vegetation cover and complex relationships with other landslides in Area 9, which collectively share the same headscarp, suggesting that Ls1 was the earliest to occur.

Adjacent to Ls1, Ls2 (Figure 4-20) is a similar style of rotational failure, and is again classed as a retrogressive multiple rotational landslide. Two back-tilted blocks are present near the toe of the slide, and west of the cut batter (Figure 4-20). The younger landslide (Ls3; Figure 4-20) to the south is considered to be a multiple translational slide, and is also retrogressive. The translational component refers to the blocks of Thomas Formation that have tilted backwards and moved downslope, and regressive refers to the blocks having moved as a result of removal of support.
The southernmost landslide of Area 9 (Ls4, Figure 4-20), appears older than the translational slide, and also comprises a back-tilted block at approximately 515masl as well as an earth slump, which is indicated by the headscarp within Ls4. Increased vegetation cover on this particular slide in Area 9 indicates that the cause for the earth slump is excess water, such as that infiltrated through the Quaternary gravels near the top, and groundwater seepages near the base.

The landslides associated with Slovens Creek Viaduct have been placed in Area 10, and are discussed in Chapter 5 as a part of the Slovens Creek Viaduct stability assessment.

4.8 Ground Failure Model

Since deposition of the Quaternary glacialfluvial deposits during ice retreat from the Last Glaciation, Slovens Stream has been progressively incising the Quaternary surface and underlying Tertiary strata. At present, the volume of water flowing down Slovens Stream is insufficient to have carved out the Slovens Valley, i.e. it is too small, however the valley would have been sufficiently formed following ice retreat to the Bernard Saddle (Figure 3-6) at 17ka, and the abandonment of the Slovens Valley meltwater channel.

In conjunction with faulting and uplift, carving by water has led to the present day geomorphology of the rail corridor. Weathering, erosion, loss of support, water regime change, and human interference are considered the main causal factors of landsliding in the Slovens-Avoca rail corridor. The effects of these causal factors are exacerbated by the high relief of the slopes to the west of the rail corridor, which are mantled with regolith and glacialfluvial deposits. Figure 4-21 shows a schematic and generic ground failure model for landslides identified within the Slovens-Avoca rail corridor, and is discussed below.

Stage 1 depicts the Quaternary surface and undifferentiated Tertiary bedrock prior to incision of Slovens Steam, and as it was following the retreat of the Slovens Creek lobe of the Last Glaciation by approximately 20ka. A dashed ‘v’ shaped line is used to represent the progressive incision of Slovens Stream over time. As a process, and following incision of Slovens Valley as a meltwater channel, incision by Slovens Stream, in conjunction with water percolated though the Quaternary gravels, has resulted in lateral retreat of the Quaternary surface due to weakening and failure of the generally ‘soft’ underlying Tertiary strata of the Slovens Valley. In the southern end of the Slovens–Avoca corridor the lateral migration of Slovens Stream, near Slovens Creek Viaduct and Broken River, is controlled by the uplifted Torlesse of the Rakaia Terrane, and only a small part of the outwash terraces remain to the west of the rail corridor.

The impermeability of the Tertiary strata, particularly the clay-rich Thomas Formation tuff with an approximate permeability ($K_v$) less than $10^{-7}$ m/s, relative to the permeable Quaternary gravels ($K_v$ approximately 0.1-1m/s) has caused a perched water table near the basal contact of the Quaternary
Figure 4-21 Schematic diagrams to illustrate slope failure development in the Slovens-Avoca rail corridor.
gravels with the Tertiary strata, and throughout the rail corridor this boundary is clearly indicated by a line of vegetation growth as a result of groundwater seepage or discharge. This boundary is not well defined however, where the sandy member of P3 is the bedrock on Map 2, Landslide Area 2, and this is directly related to the greater permeability of the sand (\(K_v\), \(10^{-4}/10^{-5}\)m/s) relative to the silty Enys Formation or clay-rich Thomas Formation Tuff.

Stage 2 in Figure 4-21 shows the topography of the rail corridor at an intermediate stage, which is between the primary stages of the incision of Slovens Stream and the present topography. The crosshatching of the upper materials in the slope is not to imply failure, but to indicate that there is a zone of Tertiary strata that has an increased unit weight due to higher water content, and consequently developing clays due to the weathering of susceptible minerals. With regard to slope failure, the increased unit weight and weakening of the strata within this zone increases the driving forces acting in the downslope direction, and once great enough the resisting forces will be overcome, which will lead to failure. At this intermediate stage, the development of shear failure surfaces are indicated by discontinuous red lines in Figure 4-21. In addition, and as a result of progressive failure development, the Quaternary surfaces are undercut by the headscarp of failures within the underlying Tertiary strata, and tension cracks, such as those observed in Landslide Area 2 (Map 2B), form in the overlying Quaternary surface.

Stage 3 of Figure 4-21 represents the present day form and topography of the Slovens-Avoca corridor. The materials that are ‘crosshatched’ in Stage 2, are now landslide debris due to the full development of shear failure surfaces and subsequent mass movement downslope. Mass movement and slope recession is concentrated on the west side of the rail corridor due to presence of often weak tertiary bedrock, when compared to the indurated Torlesse Supergroup that is present to the east of Slovens Stream, particularity near the mouth at Broken River and Slovens Creek Viaduct.

Stages 1-3 of Figure 4-21 establish the generic and overall process of ground failure that has resulted in extensive landslides throughout the Slovens-Avoca rail corridor from approximately 20ka to the present day.

4.9 Summary

- Puffer Formation member P3 is sandy and weak, which results in a successive multiple rotational failure in Landslide Area 2, and regresses into the Thomas Formation Tuff. The Thomas Formation tuff is smooth in geomorphic expression in this location which suggests back-tilted blocks are present in the multiple rotational landslide. There are tension cracks in the Quaternary gravels, which are induced by downslope failure.
- The clay-rich Thomas Formation Tuff is prone to slaking, and this is exacerbated by the presence of tens of meters of permeable Quaternary gravels unconformably overlying generally
weak Tertiary bedrock (mudstone, tuff, sandstone). Progressive incision and lateral erosion by Slovens Stream into the Tertiary strata has led to the development of multiple retrogressive slides or slumps. In some cases failure of the Thomas Formation Tuff is blocky, such as in Landslide Area 9 near Slovens Creek Viaduct.

- Active rotational slump failures in the clay-rich Thomas tuff are interpreted at metrage 71.400, 71.200 where gauge faults, top faults are recorded in the section of track. Track disturbance at metrage 70.100, landside Area 9, has not been recorded since 2013, and therefore this slide activity is considered dormant.

- Bryce Gully is actively evolving and the sediment load is high. There are active landslides associated with the incision of the stream in Bryce Gully, however do not pose immediate threat to the rail corridor. Despite this, there is possibility that the high sediment load could affect the drainage at metrage mark 70.722, and indirectly affect track operations.
5 Ground Model for Slovens Creek Viaduct Western Abutment

5.1 Introduction

Chapter 5 presents a ground model for Area 10, as defined in Chapter 4 Figure 4-16, using Map 6A (geological) and Map 6B (engineering geomorphology) at a scale of 1:2,000. Chapter 5 employs a similar approach to Chapter 4 by describing each Map before incorporating detailed geotechnical data. A larger scale (1:2000) is used on Map 6A and 6B compared to Maps 1-5A and B, given the aims of Chapter 5 and the geotechnical data available. The engineering geological model of the Slovens Creek Viaduct western abutment incorporates data from each, cored borehole one (BH1), the piezometer for borehole two (BH2), and the inclinometer installed in BH1 (Figure 5-1). From BH1 data, it is clear that the western abutment of Slovens Creek Viaduct is located in landslide debris, with a clay-rich failure surface on bedrock some 27m below ground level. This information, in conjunction with geological structures identified on Map 5A, has been used to suggest a subsurface configuration for the underlying bedrock (Puffer Formation) at the western abutment, and the clay-rich failure surface, which is presented in cross section (C-C’ in Figure 5-1).

Using borehole data and the geometry of the cross section, a ground model has been defined for limit-equilibrium and pseudo-static analyses. Both circular and block critical slip surface search methods were applied to the ground model for the western abutment of Slovens Creek Viaduct. The eastern abutment is not considered in the ground model because the ground there is identified as Torlesse Supergroup, and is considered stable. Limit equilibrium and pseudo-static analyses utilised piezometric data and strength data obtained during laboratory testing of BH1 material, which includes the shear strength of the failure surface clay, and the uniaxial compressive strength (UCS) of both the landslide debris and bedrock. The limit equilibrium and pseudo-static ground models also use geological strength index data collected in the field to develop generalised Hoek-Brown strength parameters for the appropriate materials within the model.

Sensitivity analyses have been carried out for the limit-equilibrium and pseudo-static analyses of the western abutment. As noted in Chapter 4, the Slovens-Avoca corridor is susceptible to failure induced by water, and so sensitivity to the location of the water table was examined. In addition, sensitivity to the adopted Mohr-coulomb parameters of the clay failure surface were evaluated. Pseudo-static analyses included sensitivity to seismic loading given that there are multiple active faults in the area, such as the Torlesse Fault and the Porters Pass Fault.

Prior to pseudo-static sensitivity analyses, the peak ground acceleration (PGA) for various Ultimate Limit State (ULS) design return periods, defined by an equation given in NZS1170.5:2004, were
calculated and have been used as a calibration technique to find and compare specific PGA values for pseudo-static analyses in the Slovens Creek Viaduct area. The PGA that would yield slope failure is greater than the yield seismic coefficient \( k_y \) times gravity (i.e. PGA ≥ \( k_y g \)), and the yield seismic coefficient is the value of the seismic coefficient when the Factor of Safety (FoS) of the pseudo-static analysis is equal to 1.0. The main purpose of that section is to provide indication of how railway infrastructure could be affected by seismic events of various return periods defined by ULS design standards for the area.

Figure 5-1 Image from Map 6A to illustrate the location of BH1 (inclinometer), BH2 (piezometer), culverts, Slovens Creek Viaduct, Craigieburn Road, and the unnamed road beneath Slovens Creek Viaduct. Note the Puffer formation in green, and the Thomas Formation in grey on the western abutment (E2) of Slovens Creek Viaduct.

5.2 Map 6 – Slovens Creek Viaduct Area

5.2.1 Map 6A - Geology

Area 10 comprises Torlesse Supergroup rocks of the Rakaia Terrane east of the Esk Fault, which lie in faulted contact with each the Thomas Formation Tuff and the Puffer Formation to the west of the viaduct. The Tertiary strata are partially overlain by Quaternary gravels to the west of the railway line. The geology of the Slovens Creek Viaduct end one abutment is greywacke of the Torlesse Supergroup (Rakaia Terrane), which is slightly to moderately weathered on exposed surfaces (Figure 3-9). Despite superficial weathering the greywacke is strong, and an engineering geology description is provided in Chapter 4, Section 4.2.1.
Figure 5-2 A) Photograph of the main large landslide and subsequent outcrop of Puffer formation P4 at the western abutment of Slovens Creek Viaduct, which is described in the text. Note the landslide debris of the Thomas Formation ‘slumped’ at track level below the P4 outcrop, and note the minor landslides superimposed downslope of the main slide and viaduct. Photograph taken from the eastern abutment on the northern side of the viaduct. B) A photograph of the main landslide from the southern side of the viaduct, on the eastern abutment. Note the area in red, which is zoomed into in photograph C. C) Close up image of the layers of landslide debris referred to in text. Image as oriented in photograph B. D) Image to illustrate the pooling of water on the unnamed road beneath the viaduct, which is indicated by the blue arrow and referred to in text. Image is oriented as per photograph B. E) Image of the retaining structure beneath the viaduct, and referred to in text. Image is oriented as per photograph A, and indicated by the black arrow in each.
The Puffer Formation is in situ throughout Area 10, with a large outcrop of member P4 southwest of the western abutment showing dips of approximately 60° to the northeast (Figure 5-2A). Member P3 can also be found outcropping in Area 10, approximately 200m northeast of the eastern abutment on the true left bank of Slovens Stream at an elevation of 440masl. Member P3 of the Puffer Formation is found on the true left of Broken River, which is not within the mapped area, however can be projected to intersect the rail corridor from approximately 300m southwest of metrage marks 69.714 and 69.660 (Map 6A, Figure 5-1). Engineering geology descriptions for the Puffer Formation are provided in Chapter 4, Section 4.2.2, and Puffer Formation member P4 is further described in Section 5.3 of this chapter as part of the subsurface geology.

The dip of the bedding within the Thomas Formation tuff at Slovens Creek Viaduct western abutment is highly variable (Map 6A). Structural measurements show the dip to range between 20° and 80° degrees to the southwest. This particular location is considered part of the landslide forming the ground for the western abutment, therefore variation in dip is attributed to mass movement. Engineering geology descriptions for the Thomas Formation Tuff are given in Chapter 4, Section 4.2.3, and in Section 5.3 of this chapter as part of the subsurface geology.

5.2.2 Map 6B - Engineering geomorphology

Water

The western abutment of the Slovens Creek Viaduct is wet, and the ground is persistently wet to the west of the railway line where seepages from the slope are clearly observed during the summer months (see Map 6B). Three drainage and culvert systems at track level dewater the steep slope at the western abutment, which is southwest of the MDL (Figure 5-2A), and they are located at metrage marks 69.174, 69.660, and 69.639 (Figure 5-1). The culvert at metrage mark 69.174 drains a part of Landslide Area 9, where there is a well-vegetated landslide (Ls4; Chapter 4, Section 4.7.2) as a result of the persistent groundwater. A ditch has been excavated for most of the length of Area 10, between metrage marks 69.174 and 69.639, which directs seepages away from parts of the slope that are not immediately drained by a culvert. The culvert at metrage mark 69.660 directs seepage down the bank to the northeast of the railway line (Location F, Figure 5-3). A constant flow of groundwater is drained by this culvert, and is shown in Figure 5-4F. A drainage system at metrage mark 69.639 comprises pipes and a water tank, and is at the western end of the Slovens Creek Viaduct. Groundwater is drained from the slope through a pipe, which is pooled into a water tank prior to being allowed to discharge under the viaduct. Figure 5-2D shows water pooled on the unnamed road beneath the viaduct, where the discharged water from the water tank system exits. It is worth noting that there is a retaining structure under the viaduct (Figure
5-2A, D). Persistently wet ground beneath the viaduct may be causing, or exacerbating, failures in the already unstable redistributed materials at this location, which is shown in the images of Figure 5-2.

The screen of the piezometer placed in BH2 is at 12m-14m depth, and shows the water table to be at approximately 0.8m below ground level (mbgl). Drilling was conducted with water injection, which resulted in visible disturbances at the surface, and in BH1 some 10m away, as discussed in Chapter 2, Section 2.4. Figures 5-3 and 5-4 illustrate the multiple locations where seepages of drilling fluid upwelled. The drilling fluid is water contaminated by cement of BH1 and appears as the grey opaque liquid seen in Figure 5-4 A, D and E. The groundwater directed through the culvert at metrage 69.660 (Figure 5-4F) is cloudy and grey as a result of being contaminated by the drilling fluid, which demonstrates hydraulic connectivity within the ground of the western abutment. The drilling fluid seepages move downslope, as indicated by the arrows on Figure 5-3, however the drilling fluid is pooled in images 5-4 B and C due to a slight depression in the surface.

Given the technical problems that were encountered while drilling BH2, there was concern regarding the usefulness of the piezometer. It was thought that water may be being held in the pipe as a result of issues during construction, rather than the water level in the piezometer being that of the phreatic surface.

Figure 5-3 Aerial photo to indicate the spatial distribution of drilling fluid seepages, and the approximate location of each photograph in Figure 5-4, which in turn provides an indication of scale for Figure 5-4.
Figure 5-4 Photographs of drilling fluid seepages, which upwelled during implementation of the piezometer in BH2. See Figure 5-3 for the location of each photograph A-F, and an indication of the scale.

Figure 5-5 Piezometer recovery rate following ‘blowout’ experiment as described in the text.
In order to be certain that the actual groundwater level was being measured by the piezometer, the water within the stand pipe was ‘blown’ out and the recharge rate was measured. Figure 5-5 gives the results of this test, which confirms that the water table is high, and the piezometer readings are accurate. The high level of the water table measured by the piezometer is considered to be due to a perched water table above the clay failure surface at 27.1m. Piezometer readings show the phreatic surface to remain at 0.8±0.02m below ground level since piezometer installation.

**Landslides at Slovens Creek Viaduct**

Within Area 10 there are at least six significant landslides, and of the six landslides discussed (Ls1-Ls6; Figure 5-6), five occur within the most extensive landslide (Ls6), which is the main Slovens Creek Viaduct landslide of the western abutment. The ground at the eastern abutment (Torlesse Supergroup, west of the Esk Fault) is considered stable, with no landslides observed, and is therefore not considered further in this Chapter. On the hill south of Slovens Creek Viaduct western abutment, there are numerous debris-earth flows layered upon each other (Figure 5-2 A-E; in particular C), which comprise tuff of the Thomas Formation, limestone of Puffer Formation P4, and Puffer Formation Member P3. These landslides are the youngest, and are considered to be active.

The two landslides adjacent to Slovens Stream (Ls1 and Ls2; Figure 5-6), and northeast of culverts and metrage marks 69.174 and 69.660 respectively, are considered earth slumps within landslide debris. The landslide debris mainly comprises tuff of the Thomas Formation. Ls1 and Ls2 are within the area that comprises redistributed landslide material, which has built up from clearing the railway line of failed materials. The slumps of Ls1 and Ls2 are therefore considered to be secondary to the main failure...
(Ls6, Figure 5-6), and have occurred as a result of the ground being weak redistributed landslide material. In addition, the slumps of Ls1 and Ls2 are most likely exacerbated by saturation from groundwater, which is drained by the culverts at metrage marks 69.174 and 69.660 (Figure 5-1; Figure 5-6). Water from the culverts at metrage marks 69.174 and 69.660 is known to be directed down the slope north east of track level, and seepages are observed on the unnamed road approximately northeast and east-northeast of these culverts, respectively (Figure 5-6). Based on the direction of the flow at each the culvert of metrage mark 69.174, and the culvert of metrage mark 69.660, seepages at unnamed road level (Figure 6-6; Map 6B) correspond to the locations at which the eventual culvert discharge would reappear as groundwater seepages. This suggests that the slumps of Ls1 and Ls2 (in part) occur as a result of being saturated by the discharge of culverts at metrage marks 69.174 and 69.660, which are poorly drained beyond the major infrastructure of the MDL.

BH1 was drilled approximately 50m east of metrage mark 69.147 (Figure 5-1; Figure 5-6), and into the main landslide of the end two abutment, which comprises blocky landslide material overlying Puffer Formation member P4 (Ls6; Figure 5-6). This landslide is extensive, and the toe is mapped as far as Slovens Stream. The geomorphic expression of Ls6 (Figure 5-2A; Figure 5-6) is different from those observed in the remainder of the Slovens-Avoca rail corridor, and this is interpreted as being due to the subsurface geometry and greater strength properties of the underlying bedrock (Puffer Formation member P4) relative to other Tertiary strata and bedrock of the rail corridor. This implies that the ‘soft’ Thomas Formation has slid down the relatively ‘hard’ and steeply dipping Puffer Formation P4, which is not found in the remainder of the corridor. The Puffer Formation is a steep denuded outcrop dipping at approximately 60°, and is approximately 225m in length. The headscarp of Ls6 located at 540masl and 490masl, at the western and eastern extents, respectively (Figure 5-2A, B; Figure 5-6). The orientation of the denuded face gently curves from 130° in the west to approximately 100° in the east, as indicated by the structural measurements shown on Map 6A (Figure 5-1).

The Thomas Formation is slumped below the Puffer Formation outcrop, and is at 505masl and 480masl at the western and eastern extents respectively (Figure 3-10A; Figure 3-10C; Figure 5-2A). The upper surface of the Thomas Formation is flat in geomorphic expression, and this is attributed to excavations that took place as a part of the railway landslide management and mitigation scheme of 1952, as discussed in Chapter 1, Section 1.2.3. This particular landslide was troublesome for a number years, and slips were persistent, particularly after rainfall. To remediate the frequent failures, the top of the slumped material was excavated. Anthropogenic modifications to the slope make the failure type slightly more difficult to determine, however it is thought that this slide comprises multiple failed blocks, such as those discussed in Landslide Area 9 (Chapter 4, Section 4.7.2).

Debris-earth topple is a possible failure mechanism for Ls6, given the steep incline of Puffer Formation P4, and could account for some of the steep dips in the bedding of the displaced blocks of Thomas Formation Tuff. Some beds within the displaced blocks dip up to 80° towards the denuded face of the
Puffer Formation (southwest; Figure 5-2A), which is most likely too steep to be caused purely by rotational failure. Debris-earth topple however, is thought unlikely when compared to landslides already identified along the Slovens-Avoca corridor, particularly those within the Thomas Formation Tuff, which are commonly slumps, or multiple rotational failures. Therefore, the viaduct landslide is classed as a part translational-part rotational multiple block failure, such as in Figures 4-6 and 4-13 (Chapter 4), with more recent block movements disturbing and almost overturning already failed blocks.

5.3 Subsurface Geology

Subsurface geology of BH1 comprises non-engineered fill, displaced tuff of the Thomas Formation, and Puffer Formation member P4, which are separated by a clay-rich horizon, approximately 0.2m in thickness at 27.1m below ground level, interpreted as the failure surface. Figure 5-7 provides a summary log of BH1 and is discussed below. The full logs and photographs of each core box are given in Appendix D.

Clay-rich fill is found to approximately 3m depth (Figure 5-7). The non-engineered gravel fill is brownish grey, with firm, moist, medium plasticity clay, and coarse angular gravel (ballast). Below the non-engineered fill is the displaced tuff of the Thomas Formation, and given its presence above interpreted the failure surface it is considered to be landslide debris (Figure 5-7). Strength and weathering of the Thomas Formation Tuff throughout the core is variable due to multiple crush and shear zones, which are interpreted as inter-block shears. These zones have developed as a result of multiple failed blocks being subject to shear within the sliding mass. The crush and shear zones typically contain bluish grey clay, calcite, and very angular fragments of tuff ranging from 10-70mm in size. Bedding is found in the core beginning at approximately 13.6m, and dipping between 30° and 90° from the vertical core axis. Bedding is not persistent but is present to approximately 20m depth. Some of the bedding is shown in Figure 5-8B and C, at approximately 13m and 15m, respectively. Images A-D (Figure 5-8), show that there is little consistency in the size and distribution of shear zones, the presence of bedding and the orientation of such, or within the non-systematic jointing. The landslide debris is calcareous throughout, and in most cases mm scale calcite appears as veins within the core.

As the failure surface within BH1 is approached (Figure 5-8E), the degree of shearing and polishing of angular fragments of tuff increases. Some angular fragments up to 5mm appear similar in appearance to a high grade coal, which is the result of the lustre developed by polishing. Figure 5-8D shows the core from depth 22.2m – 25.6m, which precedes the core of depth 25.6- 28m in photograph E (Figure 5-8); the clay failure surface is in photograph E (Figure 5-8) at 27.10m -27.3m. As in photograph D (Figure 5-8), clay shear surfaces are not limited to the failure surface immediately above the Puffer Formation bedrock, and are intermittent between 23.4m and 27.3.
5-7 Summary log of BH1. Note the clay failure surface at 21.1m, which is detailed in Figure 5-9.
Figure 5-8A) Photograph of core box 4, 6.7m to 9.1m. Note the poor condition of the core due to weakness of the subsurface landslide materials. B) Photograph of core box 6, 11.5m to 13.9m. Note bedding is present in the core from 13.6m. C) Photograph of core box 7, 13.9m to 16.3m. Note the bedding terminates at approximately 14.6m. D) Photograph of core box 11, 23.4m to 25.8m. Note the clay within the crush and zone at 23.5m to 24.2m. E) Photograph of core box 12, 25.8m to 27.9m. Note the crush zones as the clay failure surface is approached at 27.2m depth. The Puffer Formation P4 begins at 27.4m depth. F) Photograph of core box 13, 27.9m to 30.1m. Note the good condition of the core, which is directly related to the strength properties of Puffer Formation P4.
Figure 5-9 Detailed log of core box 12 (Figure 5-8E) at depth 25.8m to 27.9m, and including the clay failure surface.
Throughout the length of core comprising landslide debris from 3.0m to 27.3m, there are slight variations in strength and density, as well as a minor variation in hue due to different degrees of weathering, particularly at inter-block shear zones, and variations in the type of tuff. Member P4 of the Puffer Formation was encountered at approximately 27.3m below ground level in BH1, and was found to be unweathered to slightly weathered and strong. Joints within the Puffer Formation (P4) consistently dip at an approximate angle of 50° to the core axis, and are moderately widely spaced with smooth stepped surfaces. Closely spaced horizontal joints are considered to be disturbance caused by drilling. The core was not orientated, and so the direction in which these joints are dipping is unknown. The Puffer Formation exposed above track level shows a steep dip of 60°, however because Puffer Formation P4 is found in BH1 at 27.3m, it is known that the dip observed at track level must shallow out with depth. The shallowing at depth, and the curvature measured by strike changes at the surface (Map 6A), as discussed in Section 5.2.1, is attributed to the synclinal structure near metrage mark 70.000, which is discussed in Chapter 4, Section 4.7.1.

5.4 Engineering Geology Model

Figure 5-10 shows the engineering geology model of the Slovens Creek Viaduct western abutment, as interpreted from the geomorphology, structural measurements, and subsurface geology. The subsurface geometry of the Puffer Formation bedrock and the failure surface is interpreted, but the engineering geology model is constrained by (1) the location of Slovens Stream, and the geomorphic interpretation that failure is due to incision of Slovens Stream at the toe; (2) the outcrop and structural measurements of the Puffer Formation (P4) at track level; and (3) the location of the Puffer Formation (P4) at 27.3m within BH1.

Figure 5-10 is similar to the schematic ground failure model of Chapter 4, Figure 4-21, with regard to the geomorphological interpretation. Geomorphology throughout the Slovens-Avoca rail corridor suggests that the Quaternary gravels extended across the entirety of the Slovens Valley. Similarly, the Slovens Creek Viaduct model (Figure 5-10) indicates two such surfaces at 20ka and 15ka to show the progressive incision of meltwater and Slovens Stream since the Last Glaciation, despite the absence of Quaternary gravels in the line of section selected (C-C”; Figure 5-1). In this location however, the lateral migration of Slovens Stream is limited by the indurated sandstone (greywacke) of the Torlesse Supergroup to the east. Moreover, Slovens Stream has incised deeply into materials that have been sheared and weakened by the Torlesse Fault. By way of preferential erosion, faulting, and limited lateral migration, the incision of Slovens Stream at Slovens Creek Viaduct is deeper than at any other point of Slovens Stream between Avoca (at 525masl) and Slovens Creek Viaduct (at 430masl), and the deeper incision is assumed to have contributed to the significant landslides in Area 10.
Figure 5-10 Cross section C-C’ of the Slovens Creek Viaduct western abutment showing an interpreted failure surface for the large landslide (Ls6) of Area 10. The geometry of the landslide is an interpretation based on the geomorphic expression, the clay failure surface identified in BH1, and structural measurements.
In addition, structural measurements from the Puffer Formation (P4) at track level of the western abutment, and from the Thomas Formation Tuff alongside the rail corridor, are consistent with a folded syncline structure at approximately 70,000 metrage, as already discussed. The fold structure is reflected in Figure 5-10, where the western extent of the Puffer Formation is folded to an approximate angle of 60°, and shallows out at the inferred location of the syncline axis (Figure 5-10). The syncline is a complex structure, however in its simplest form, and for the purposes of the cross section, the axis is considered to be approximately aligned north-south and dipping towards the north.

5.5 Limit Equilibrium Analysis

5.5.1 Background

Limit-equilibrium analysis is one of the oldest approaches in slope stability that uses the Factor of Safety, or FoS (Chiwaye, 2010). The FoS is defined as the ratio of resisting forces (strength) to driving forces (loading), and where the output of the ratio is one, the slope is considered to be in the state of limit-equilibrium. A value greater than one corresponds to a stable slope, and an unstable slope would return a value less than one (Chiwaye, 2010).

Limit equilibrium analyses for this study were carried out using SLIDE software from Rocscience Inc. SLIDE is a 2D limit equilibrium slope stability programme for evaluating the safety factor or probability of failure for circular or non-circular failure surfaces in soil or rock slopes. SLIDE analyses the stability of slip surfaces using vertical slice limit equilibrium methods (Rocscience, 2015), which are discussed below. It should be noted that although probability of failure statistics are available in SLIDE, they were not used in this study.

The fundamentals of vertical slice limit equilibrium methods involve many assumptions, equations and theories, each of which is well documented and summarised by Abramson et al (2001). Of the methods available, the Bishop simplified and Janbu’s corrected methods of slices are considered most suitable for this study, and were therefore used in all analyses. Bishop’s method is more suited to circular failure mechanisms which can occur in homogeneous rock or soil masses. For inhomogeneous rock or soil masses, Janbu’s corrected method is more appropriate, since it can deal with non-circular failure surfaces (Chiwaye, 2010). The methods are run simultaneously in each analysis to ensure that the most appropriate method is applied.

5.5.2 Input parameters

Strength Data

Laboratory and field strength data were collected in order to provide input parameters for limit-equilibrium modelling. The collection of data, and methods used in the laboratory, were discussed in Chapter 2, and further details are provided in Appendices A and B. Data were obtained from a series of
tests carried out on the core from BH1, and the related laboratory and field results are summarised in Table 5-1.

Table 5-1 Input parameters for limit equilibrium model as determined by laboratory and field work.

<table>
<thead>
<tr>
<th>Strength Type</th>
<th>Unit weight (ρg)</th>
<th>UCS (kPa)</th>
<th>mb</th>
<th>s</th>
<th>a</th>
<th>Cohesion (L ; kPa)</th>
<th>Phi (φ)</th>
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<td>Thomas Formation (landslide Debris) Generalised Hoek-Brown</td>
<td>20 22</td>
<td>5000</td>
<td>0.401</td>
<td>4.5^5</td>
<td>0.585</td>
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<td></td>
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<td>400</td>
<td>29°</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Puffer Formation Generalised Hoek-Brown</td>
<td>25 27</td>
<td>67000</td>
<td>3.082</td>
<td>0.035</td>
<td>0.501</td>
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</tr>
</tbody>
</table>

Critical slip surface search methods

SLIDE provides three search methods for circular slip surfaces, and two methods for non-circular slip surfaces. Circular search methods (grid search, slope search or auto refine search) are best for a homogenous slope, or for a slope without well-defined layers of weakness, whereas a model with well-defined weak layers is best suited to non-circular search methods, such as block search methods. Although this is the case, a circular search method with ‘composite surface options’ (i.e. the ability to search for composite circular/non-circular surfaces) turned on will also suffice for a model with well-defined weak layers, such as the clay failure surface in the model for this study. Both circular and block search methods are suitable for a model comprising multiple materials without a well-defined layer of weakness (Rocscience, 2015).

For limit-equilibrium analyses in this study, both the grid search and auto refine circular critical slip surface search methods were used, in addition to the block search method. Grid search and auto refine search methods are automated in SLIDE with little user input, although for the grid search method the slip centre is defined. The auto refine method is the only method that is actually ‘searching’ for the critical slip surface. The auto-refine method uses a simple but effective algorithm to retain the results of each iteration, and builds upon its ‘knowledge’ to refine the search (Rocscience, 2015). By using this process, the auto refine search finds the surface with the lowest FoS. For block failure, the slip surfaces are user defined. In this study multiple block failure surfaces were added to the ground model, both on the clay failure surface of BH1, and within the landslide debris.

Statistical analyses

In this study, sensitivity analyses were run alongside each circular and non-circular critical slip surface search. The sensitivity analysis considers each random variable defined for the model between the mean and relative mean to examine the effect that variable has on the FoS. The relative mean is simply a
number by which the parameter is varied relative to the mean, for example when 0.15 is the mean and 0.10 is the relative mean, the variable is analysed for the range 0.05 and 0.25 (i.e. 0.15±0.10) with a normal distribution. The results of the analysis can be presented in a number of different types of graph, and in this study the results of the sensitivity analyses were analysed in a Sensitivity Plot. Simply put, a steep curve on the Sensitivity Plot shows the model is sensitive to the random variable or parameter, such as cohesion (kPa), phi (degrees) or groundwater, and a flat slope shows that the model is not sensitive to that parameter. SLIDE carries out the sensitivity analysis of each parameter one at a time, and so a relationship between the random variables is not computed. Sensitivity analysis partially overcomes some of the uncertainties within limit equilibrium analyses, which are discussed in Section 5.5.3.

**Groundwater**

Utilising the groundwater level measured by the piezometer, a water table is assigned to the model passing track level at 0.8m below the surface, which is the only known point. The groundwater parameters are user-determined in SLIDE. For the model in this study the groundwater method is used. The groundwater method utilises the $H_u$ coefficient to determine the pore pressure within each slice of the model. The $H_u$ coefficient is defined as:

$$
\mu = \gamma_w h H_u
$$

Where:

- $\mu$ = pore pressure
- $\gamma_w$ = the pore fluid unit weight
- $h$ = the vertical distance from the base of a slice to a water surface
- $H_u$ = the $H_u$ coefficient for the soil type

The $H_u$ coefficient is simply a value between 0 and 1, where 0 represents a dry soil and 1 represents maximum pore pressure, however where a value between 0 and 1 is user-defined, it is only applicable to a horizontal groundwater surface. Because the model in this study has an inclined groundwater surface across some slices, the automatically calculated $H_u$ value was selected so that the correct $H_u$ value could be calculated for each slice comprising an inclined water surface. The method is performed based on the assumption that there is a straight equipotential line between the base of the slice concerned and the groundwater surface, as illustrated in Figure 5-11.

The $H_u$ groundwater method is capable of approximating head loss due to seepage, in the absence of detailed data such as seepage analysis results. This is particularly useful in this study because the maximum ground water level defined for sensitivity analyses is above ground level near the ditch southwest of the track of the western abutment, and this is taken to imply seepage at track level.
5.5.3 Assumptions and limitations

Laboratory and Field Data

The clay of the failure surface was remoulded prior to direct shear testing, and the sample is therefore disturbed. The value for cohesion ($c$) obtained from the direct shear test was approximately 410kPa, which seems excessive. Holtz et al (2011), however, state that the strength of very stiff clay can vary between 200 and 400kPa, and in hard clays can be in excess of 400kPa. The clay of the failure surface is very stiff, as documented in borehole data (Appendix D), and the numbers obtained from the direct shear test are comparable to values provided by Holtz et al (2011). The data are therefore considered appropriate despite remoulding. In addition, the internal angle of friction, $\phi$, could also be overestimated. Because the clay is in a zone of shear, angular fragments of surrounding material were present within the clay. Large fragments were removed, however some probably remained within the sample, and may have caused a consequent increase in the measure of the internal angle of friction obtained in the laboratory, which was found to be approximately 29°. Although this is the case, a value of $\phi$ between 20° and 30° is acceptable for clay (Holtz et al, 2011).

As discussed in Chapter 2, UCS and shear strength data, used in the computer generated ground models for the slope at the western abutment, are based upon a few samples from within BH1 with the exception of GSI data for use in generalised Hoek-Brown strength criterion, which was collected in the field. This means that it is being assumed that the strength properties of the 67mm diameter samples are applicable to the entire sliding mass. Rock and soil masses are naturally inhomogeneous, and it is not likely that this is the case, however the use of field observations within the generalised Hoek-Brown criterion helps to account for behaviour of the rock mass on a larger scale.

Limit equilibrium analysis

Limit-equilibrium is a simplistic method of slope stability analysis, and limitations lie within the fundamental assumptions for vertical slice limit equilibrium methods (see Abramson et al (2011)).
Some corrected methods of analysis, such as Janbu’s corrected method which accounts for inter slice shear whereas Bishops simplified method did not, help to compensate for the simplicity of limit equilibrium analysis (Rocscience, 2015). In addition, the limit equilibrium method has a major disadvantage, when compared to numerical analyses, in that that there is no information on how the slope might behave or deform. That is, limit-equilibrium only identifies the onset of failure based on mechanical measures, without taking into account the effects of stress distribution or progressive failure. Moreover, the limit equilibrium method assumes input parameters are known with certainty, however the uncertainty can be accounted for by ensuring the FoS is greater than 1.0 (Chiwaye, 2010).

5.5.4 Output/Results

A graphical representation of the limit equilibrium model output for each slip surface search method, alongside the respective sensitivity plot, is presented in Figure 5-12. Original full size images of each are given in Appendix E. Table 5-2 summarises the resultant FoS for each slip surface search method, in addition to the parameters used in the sensitivity analysis, and the lowest FoS for variables to which the model is sensitive. The models are the same orientation and topography as C-C’ in Figure 5-10, however only the south-western half of the slope is modelled.

Circular - Grid Search Method

The circular grid search method returned a FoS of 1.638 under normal conditions (i.e. the mean value), with sensitivity to the water table only. The FoS (y-axis) is reduced to its lowest value of 1.340 when the water table (x-axis) is at its highest percentage of the range, which is defined by the relative mean. This relationship is demonstrated by the negative correlation and steep incline of the red curve on the accompanying graph in Figure 5-12A. The curve for each of the parameters cohesion and phi is flat in the Sensitivity Plot. This shows the slip surfaces are not sensitive to cohesion or phi of the clay failure surface, and this is because the circular surfaces are not on or within the clay surface. Circular slip surfaces are statistically defined, and generally occur with the landslide debris (tuff of the Thomas Formation) in the lower northeast of the slope as a result of this analysis (Figure 5-12A and B).

Circular – Auto Refine Search Method

The circular auto refine search method, with composite surfaces turned on, returned a lower FoS than that of the grid search method. The FoS under normal conditions is 1.552. As for the grid search method, the auto refine method returned sensitivity to the water table only, with the lowest FoS of this relationship being 1.300. Excluding FoS values, the Sensitivity Plot for the auto refine search method is the same as for the grid search method. The curve that describes water table sensitivity is steeply inclined with a negative FoS correlation, and so FoS decreases with an increase in groundwater level.
Figure 5-12  Limit-equilibrium models. Full size images are in Appendix E, this figure is intended as a summary overview. Each model is accompanied by a sensitivity plot. Each sensitivity plot has the same axes, where the FoS is on the y-axis, and the percentage of the random variables are on the x-axis. The red line of each sensitivity plot in this figure represents the water table. A) Circular grid search method result. Note the negative correlation between the water table and the FoS. B) Circular auto refine search method. Note the similarity to the grid search method, and the slip surface identified on the lower eastern portion of the slope C) Non-circular block search method. Note the sensitivity to the water table (red) in addition to cohesion (blue).
Table 5-2 Summary of results for FoS and lowest FoS from sensitivity analysis. Variables for sensitivity analysis are provided

<table>
<thead>
<tr>
<th>Sensitivity Parameters</th>
<th>Phi (ϕ) (clay failure surface)</th>
<th>Cohesion (kPa) (clay failure surface)</th>
<th>Water Table (m)</th>
<th>Sensitive to; lowest FoS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Search Method</td>
<td>FoS Mean</td>
<td>Relative mean</td>
<td>FoS Mean</td>
<td>Relative mean</td>
</tr>
<tr>
<td>Circular (Grid)</td>
<td>1.638</td>
<td>25</td>
<td>300</td>
<td>100</td>
</tr>
<tr>
<td>Circular (Auto Refine)</td>
<td>1.552</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Non-circular (Block)</td>
<td>2.681</td>
<td>200</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

The curves remain flat for both cohesion and phi, which shows the model is not sensitive to these variables.

Non-Circular - Block Search Method

As discussed in Section 5.5.1, the block search method is probably the most appropriate for the model in this study because there is a well-defined weak layer present. Although this is the case, the FoS for the surface defined at the failure clay surface returned a FoS in excess of 20.000, and a block surface defined above the failure surface, within the landslide debris, returned the lowest FoS of 2.681. Figure 5-12C shows the sensitivity plot of the block search method next to the ground model. As described by the curves of the sensitivity plot, the block search method displays sensitivity to both the water table and cohesion, with the lowest FoS values being 2.567 and 2.647, respectively. Rather than the negative correlation observed for the FoS-water table relationship, the FoS-cohesion curve shows a positive correlation. This is because, as the value of cohesion is increased, the FoS also increases. The curve of cohesion is not as steep as the water table curve, and therefore the model is not as sensitive to cohesion as it is to groundwater. All previous trials of the block search method showed sensitivity to cohesion in the Sensitivity Plot, as does the block search presented in Figure 5-12C. When the cohesion of the clay surface in the model was reduced to a mean value of 300 kPa, and a relative mean of 100 kPa, a FoS of up to 25 was returned. Cohesion was modelled with a mean of 200 kPa, and a relative mean of 100 kPa in the block search method to see if the FoS would be significantly affected by the value of cohesion.
5.6 Pseudo-static Analysis

5.6.1 Background

Pseudo static analysis is the simplest form of seismic slope performance evaluation, and depending on the strength and pore pressure properties of the slope, is usually over or under conservative as a consequence (Jibson, 2011). As limit-equilibrium analysis cannot account for stress deformation or progressive failure, pseudo-static analysis cannot account for seismically-induced cyclic shaking or transient stress conditions. In most cases, as is the case in this study, the pseudo-static earthquake load is modelled using only the horizontal component. This is because when using the relevant equations for the pseudo-static FoS (Eq 5-2; Eq 5-3), the vertical component tends to equate to near zero, and is therefore negligible (Jibson, 2011). Because the vertical component is ignored for the purposes of this study, the effect of pore water pressure cannot be accounted for in the pseudo-static analysis (Rocscience, 2015).

5.6.2 Assumptions and limitations

Assumptions and limitations of laboratory and field data are the same for pseudo-static analysis, and the limit equilibrium analysis. In addition, pseudo-static analysis assumes that the seismic load is constant (i.e. static). Seismic shaking is not static, rather it is cyclic, and so significant limitations lie within this assumption. Moreover, because the vertical component of seismic shaking is assumed to be zero, the effects of pore water pressure cannot be modelled, however the pore water component of the clayey tuff materials are likely negligible in themselves due to the low permeability of the clay-rich tuffaceous landslide debris ($K_v <10^{-7}$ m/s).

5.6.3 Input parameters

Input parameters for the pseudo-static analyses are the same as for limit-equilibrium analyses (Section 5.5.1) but with the addition of a seismic coefficient. The seismic coefficient is essentially a reduction factor for the peak ground acceleration (PGA) that helps to overcome some of the limitations of the simplistic static analysis.

The pseudo-static FoS (or FS) is defined by the equation:

\[ FS = \left| \frac{(W \cos \alpha - kW \sin \alpha) \tan \phi}{W \sin \alpha + kW \cos \alpha} \right| \]

Where:

FS = the pseudo-static Factor of Safety (FoS)

\( W = \) the weight per unit length of slope

\( \Phi = \) the friction angle
A = the slope angle

\( \) = the pseudo-static (seismic) coefficient

The seismic coefficient is defined as:

\[
Eq. \ 5-3 \quad k = \frac{a_h}{g}
\]

Where:

\( a_h \) = the horizontal acceleration

\( g \) = acceleration due to earth’s gravity

Most pseudo-static models use a seismic (or pseudo-static) coefficient ranging from 0.1-0.3 (Rocscience, 2015). The yield seismic coefficient \( (k_y) \) is equivalent to the value of the seismic coefficient \( (k) \) at the point where the FoS is equal to 1.0. The yield seismic coefficient \( (k_y) \) is also equivalent to the PGA that would cause seismically induced failure, which is reported as \( k_y \)g (Jibson, 2011). Therefore, the yield seismic coefficient, \( k_y \), is required to find the PGA that would induce failure, and any PGA that exceeds \( k_y \)g, is considered to cause seismically induced slope failure. There are multiple approaches to find \( k_y \), and in this study sensitivity analyses have been employed. The mean and relative mean seismic coefficients of 0.15±0.15 were chosen to find \( k_y \), since previous models with \( k = 0.1±0.1 \) did not find \( k_y \) because the model did not achieve as FoS less than or equal to 1.0. The yield PGA found in pseudo-static modelling should be put in context for the area of Slovens Creek Viaduct, therefore Ultimate Limit State design criteria of NZS 1170.5:2004 can be used as a comparative measure.

5.6.4 Ultimate Limit State

The purpose of this section is to put the output values of pseudo-static modelling in context, and in turn to provide an indication as to whether or not damage to infrastructure of the Slovens Creek Viaduct western abutment could occur as a result of a ULS design earthquake. It is worth noting that NZS 1170.5:2004 is not intended for use in slope stability or for bridges, and that the resultant \( C(T) \) values are being regarded only as indicative PGAs that could be expected in the area and soil type of Slovens Creek Viaduct western abutment. Table 5-3 provides the values used for each input parameter and the results from equation 5-4 below, as given by NZS1170.5:2004:

\[
Eq. \ 5-4 \quad C(T) = C_h(T)ZRN(T,D)
\]

Where:

\( C_h(T) \) = the spectral shape factor determined from Clause 3.1.2

\( Z \) = the hazard factor determined from Clause 3.1.4

\( R_u \) = the return period factor from Clause 3.1.5, but limited so that \( ZR \) does not exceed 0.7
\( N(T,D) \) = the near fault factor determined from Clause 3.1.6

Table 5-3 Values of \( C(T) \) calculated with the above equation from NZS 1170.5:2004. \( C(T) \) is analogous to PGA.

<table>
<thead>
<tr>
<th>( C(T)(g) )</th>
<th>Hazard Factor (Z)</th>
<th>Return period</th>
<th>( R_u )</th>
<th>( N(T,D) )</th>
<th>( C(T)(g) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/20</td>
<td>0.20</td>
<td>1</td>
<td>0.189</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/25</td>
<td>0.25</td>
<td>1</td>
<td>0.236</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/50</td>
<td>0.35</td>
<td>1</td>
<td>0.330</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/100</td>
<td>0.50</td>
<td>1</td>
<td>0.472</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/250</td>
<td>0.75</td>
<td>1</td>
<td>0.708</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/500</td>
<td>1.00</td>
<td>1</td>
<td>0.944</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/1000</td>
<td>1.30</td>
<td>1</td>
<td>1.227</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/2000</td>
<td>1.70</td>
<td>1</td>
<td>1.605</td>
</tr>
<tr>
<td>2.36</td>
<td>0.4</td>
<td>1/2500</td>
<td>1.80</td>
<td>1</td>
<td>Invalid</td>
</tr>
</tbody>
</table>

Figure 5-13 Hazard Factor Map for the South Island. Figure taken from NZS 1170.5:2004.
As discussed in the methods of Chapter 2, section 2.6, the spectral shape factor is a value selected from Table 3.1 in NZS 1170.5:2004, and the value is based on the site soil subclass \((C_h)\) as well as period \((T)\). For the Slovens Creek Viaduct western abutment, the site subsoil class is \(C\) with period \(T=0\), and so from Table 3.1 of NZS 1170.5:2004 \(C_hT\) is equal to 2.36g.

The hazard factor, \(Z\), is taken from the Hazard Factor map for the South Island (Figure 5-13), which is provided in NZS 1170.5:2004. The western abutment of Slovens Creek Viaduct is approximately located as shown by the orange triangle in Figure 5-13, and the hazard factor, \(Z\), is therefore taken as 4.

As defined by NZS 1170.5:2004, the near-fault factor is equal to 1 because the site location is more than 20km distance from any of the major faults listed, such as the Hope Fault or Alpine Fault.

For the return period 1/2500, \(ZR\) is greater than 0.7, and as specified in Clause 3.1.5 of the NZS 1170.5:2004, the \(C(T)\) value is therefore invalid. The ULS event is equal to the PGA of a 1/500 return period and is highlighted in red in Table 5-3. For infrastructure to survive a ULS event the yield PGA \((k_y g)\) from the sensitivity plot of pseudo-static model should be greater than or equal to 0.944g. PGA is analogous to the \(C(T)\) of Table 5-3.

### 5.6.5 Output/Results

Results of the pseudo-static analyses are graphically presented in Figure 5-14, and summarised in Table 5-4. The mean (and relative mean) for each variable in sensitivity analyses are also detailed. For comparison, the FoS values of pseudo-static analyses are given alongside the FoS found in limit-equilibrium analyses, as well as the seismic yield coefficient \((k_y)\), and corresponding PGA \((k_y g)\).

**Circular Grid Search method**

Results of the pseudo-static circular grid search method returned a FoS of 1.138 in normal conditions (i.e. the mean values of Table 5-2 and \(k = 0.15\)). The seismic coefficient remains as 0.15±0.15 as the mean and relative mean respectively for all circular slip surface search pseudo-static analyses.

Sensitivity plots (Figure 5-14A; Appendix E) of the grid search analysis show the ground model to be more susceptible to changes in the seismic coefficient when compared to the water table, and the yield coefficient, \(k_y\), is 0.2074 and equal to a PGA of 0.2074g. This model suggests therefore, that any PGA greater than 0.2074g could result in ground failure of the lower north-eastern part of the slope at the western abutment. As for the circular grid search in limit-equilibrium analyses, the grid is only looking for circular slip surfaces, and therefore the sensitivity plot shows the ground model is not sensitive to cohesion or phi.

**Circular Auto Refine Search method**

The circular auto refine search method shows the ground model is sensitive to the water table and to the seismic coefficient. The FoS value returned under normal conditions returned an FoS of 1.117.
Figure 5-14 Pseudo-static models. Full size images are in Appendix E, this figure is intended as a summary overview. Each model is accompanied by a sensitivity plot. Each sensitivity plot has the same axes, where the FoS is on the y-axis, and the percentage of the random variables are on the x-axis. The red line of sensitivity plots A and B in this figure represents the seismic load, the water table is dark blue. Note the slip surface identified on the lower eastern portion of the slope. The seismic load is blue dark blue in the sensitivity plot of C, and the water table is red. A) Circular grid search method result. B) Circular auto refine search method. Note the similarity to the grid search method C) Non-circular block search method.
Again, sensitivity to the seismic coefficient is greater than that of the water table, and \( k_y \) is equal to a PGA of 0.216g. In addition, no sensitivity to cohesion or phi of the failure surface clay was detected in this analysis, because the failure surfaces of the auto-refine method are circular, and the clay-rich failure surface is not. As has been the case for all circular critical slip search methods, the low FoS is on the lower north-eastern part of the slope to the northeast of the MDL, and does not include the part of the slope where the MDL and other infrastructure are situated.

Table 5-4: Summary of limit equilibrium and pseudo static analysis results. The PGA of the yield coefficient is also given.

<table>
<thead>
<tr>
<th>Analysis Type</th>
<th>FoS (mean)</th>
<th>FoS (mean)</th>
<th>( k_y )</th>
<th>PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Limit Equilibrium</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circular Grid Search</td>
<td>1.638</td>
<td>1.136</td>
<td>0.207</td>
<td>0.207g</td>
</tr>
<tr>
<td>Circular Auto Refine</td>
<td>1.552</td>
<td>1.117</td>
<td>0.216</td>
<td>0.216g</td>
</tr>
<tr>
<td>Non-Circular Block</td>
<td>2.681</td>
<td>1.205</td>
<td>0.400</td>
<td>0.400g</td>
</tr>
</tbody>
</table>

**Non-Circular-Block Search Method**

The block search model found the lowest FoS values on the same surface as in the limit equilibrium analyses, which is the user-defined surface immediately above the clay-rich failure surface. Compared to circular slip methods the pseudo-static analysis required a variation of mean and relative mean of the seismic coefficient to find \( k_y \) because an FoS of 1.0 could not be obtained with 0.15±0.15. The seismic coefficient and relative mean used was instead 0.3±0.2. For the pseudo static analysis however, the FoS has been reduced to 1.695. The yield coefficient is \( k_y \) is equal to 0.400g.

5.7 Discussion

5.7.1 Limit-Equilibrium Stability Analysis Results

Limit equilibrium circular slip surface search methods, both grid search and auto refine search, indicated the slope is stable with a FoS greater than 1.0 returned from each, however one particular surface returned the lowest FoS in each. This surface is in the lower portion of the slope, adjacent to Slovens Stream and east of the MDL (Figure 5-12A and B; Figure 5-14A and B). Multiple rotational regressive failures are typical of the tuffaceous materials of the Slovens-Avoca rail corridor, as explored and discussed throughout Chapter 4. A regressive failure on the lower slope to the east of the MDL could be triggered by a seismic event, or adverse weather conditions, and could cause disruption to MDL operations by undercutting if the main scarp of the slide was to regresses to track location. Further numerical modelling of the slope, such as sliding block analyses, could provide a better indication of
how the lower portion of slope would behave during an earthquake. Limit-equilibrium and pseudo-static analysis cannot assess dynamic slope failure, and has therefore not been done in this study.

The block slip surface search method is the only one that allows for user-defined slip surfaces, and therefore is the only model to consider the clay failure surface for which evidence was found in BH1. This method returned a FoS in excess of 20 for slip along the defined clay failure surface. The high FoS is attributed to the low permeability and high cohesion of the clay, as well as the ‘arm chair’ shape of the bedrock and failure surface of the ground model. The FoS was only slightly reduced when the cohesion of the failure surface clay was varied between 100kPa and 300kPa, which suggests the high cohesion of the very stiff clay (400kPa) is not entirely the reason for the high FoS along that surface. The high FoS found in limit equilibrium block search modelling could in part be due to landslide deposits of past slope failure buttressing the toe of the slope, and increasing the level of resisting forces given the ‘arm chair’ shape of the Puffer P4 bedrock (Figure 5-10). In addition to a possible increase in resisting forces, driving forces have also been reduced. As discussed in Chapter 1, Section 1.2.3 remedial measures were taken in 1952 to prevent slips covering the track, and the top of the slope to the west of the rail corridor was excavated. Limit-equilibrium modelling is based upon the ratio of driving to resisting forces, and therefore these two factors are thought to provide a significant contribution to the high FoS found along the clay-rich (interpreted) failure surface. Both circular slip surface and non-circular slip search methods suggest failure at the western abutment of Slovens Creek Viaduct is more likely to occur within the displaced tuff of the Thomas Formation, rather than on the interpreted clay failure surface. Pseudo static analyses found the most critical slip surfaces in the same location as limit equilibrium models, and it is therefore postulated that main viaduct landslide is unlikely to fail on the clay failure surface in future. It is recognised however, that the geometry of the ground model in Figure 5-10 may be incorrect, and that data from further geotechnical investigations are required to properly determine the subsurface geometry and provide accurate conclusions.

5.7.2 Pseudo-static analysis results

As expected, pseudo-static analyses returned a lower FoS overall when compared to limit equilibrium analyses (Table 5-4). The yield seismic coefficient, \( k_y \), was found for each grid and auto refine circular slip surface search methods, as well as the block search method in pseudo-static analyses. The PGA that would induce failure in each the circular (0.207g) and auto refine search methods (0.216g) suggest that the ground of the Slovens Creek Viaduct western abutment would fail if an event of a one in 25year return period was to occur with a PGA of 0.236g. The surface upon which failure would occur however, does not include the portion of the slope comprising the MDL and other railway infrastructure. The portion of the slope in these models is to the east of the railway, which comprises fill and landslide debris from previous failures (Figure 5-12A and B; Figure 5-14A and B). The user-defined block search method, which consists of a failure surface beneath infrastructure of the western abutment, returned a
yield seismic coefficient, or PGA, of 0.400g suggesting that the infrastructure would survive an event of a one in 50 year return period (0.33g), and almost a one in hundred year (0.472g). A ULS event in the Slovens Creek Viaduct area would likely consist of PGA up to 0.944g, and the pseudo-static models therefore suggest that ground of the Slovens Creek Viaduct western abutment would fail, and therefore the effects on infrastructure upon it would most likely be catastrophic.

5.7.3 Groundwater Analysis Results

Both limit-equilibrium analyses and pseudo-static analyses, with a seismic coefficient of 0.15 show that the ground of the western abutment of Slovens Creek Viaduct is sensitive to groundwater level; that is, the slope and infrastructure upon it could be adversely affected if a significant increase in groundwater level was to occur, for example by prolonged rainfall such as in January of 1952 (discussed in Chapter 1, Section 1.2.3). However, the analyses conducted have modelled groundwater level as ‘worst case scenario’, and despite this the FoS returned for each type of analysis is greater than 1.0 (Table 5-2). The analyses are said to consider worst case scenario because the groundwater level of -0.8mbgl, measured by the piezometer, is due to a perched water table situated above the impermeable clay failure surface, and the actual ground water level is considered to be much lower. For the purposes of analyses in this study however, the water level in the piezometer has been assumed as the phreatic surface for the ground of western abutment of Slovens Creek Viaduct, therefore the analyses conducted consider the slope to be saturated. Moreover, where the mean groundwater level has been modelled at 0.8mbgl with a relative mean of 0.8m the slope has been modelled as saturated to track level. Groundwater level to the west of the rail corridor, where the ditch discussed in Section 5.2.2 has been excavated, is considered less than 0.8mbgl due to permanent seepages and pooled water. Given the relative mean of 0.8m, the groundwater is modelled as being above the ground surface in the +0.8m analysis. Therefore, in these models where the lowest FoS returned is 1.3 (Table 5-2) the slope is considered to be stable in steady-state conditions when saturated. Transient flow conditions may have an effect on the slope stability given the low permeability and hydraulic connectivity of the tuffaceous blocky landslide debris, however these parameters have not been considered in this study, and this is recognised as limitation.

5.7.4 Current stability assessment

At the present time, the western abutment of Slovens Creek Viaduct shows no visible evidence for surface instability that could cause immediate disruption to operations of the Midland Line. Geomorphological evidence, such as tension cracks or scarps with little vegetation indicating recent instability of a slope, have not been observed at the western abutment of Slovens Creek Viaduct, and there are no recent reports of disruption to operations or slips covering this section of track on the Midland Line. Similarly, track alignment surveys, which have identified line or top faults in other sections of the Slovens-Avoca rail corridor that appear to be related to instability of slopes adjacent,
have not been identified in the section of track at the western abutment of Slovens Creek Viaduct. However, an inclinometer was installed in the BH1 due to concerns regarding evidence for previous instability of the slope of the western abutment, especially following the 1952 prolonged rainfall event.

An in-depth technical review of inclinometers and interpretation of inclinometer data is available in Stark and Choi (2008), from which a good example of cumulative and non-cumulative inclinometer data is illustrated in Figure 5-15. The data on the right are presented in a profile plot. Tilt plots and profile plots display the same data in a different way, and by doing so strengthen the ability to interpret the data correctly. The data on the left are displayed in a tilt plot, and do not record cumulative displacement, which means a tilt plot will only display a ‘spike’ at the location of movement on any given slip surface (Figure 5-15, left). A profile plot does, however, record cumulative displacement. This means that a profile plot will continue to display the amount of displacement on a failure surface up to ground level if there has been displacement on any given slip surface below it (Figure 5-15, right), and if there has been slip on more than one surface the profile plot will continue to accumulate the displacement and move further away from zero until ground level is reached. A tilt plot will simply record another spike at the location of further displacement.

![Figure 5-15 Example of inclinometer data from Stark and Choi (2008). The tilt plot is on the left only displays a ‘spike’ at the location of the slip surface. The cumulative plot is on the right, and the amount of slip is recorded between the slip surface and ground level, as described in the text.](image)

108
Figure 5-16 Tilt change (left) and profile change (right) plots for the Slovens Creek Viaduct inclinometers. The full Geotechnics reports are available in Appendix F.
Data collected from the inclinometer in this study are in agreement with that of the limit equilibrium and pseudo-static models, and do not indicate movement within the slope during the period 22 May 2015 to 04 August 2015. The full reports of the inclinometer base read and the second inclinometer reading can be found in Appendix F, and the graphed data from each are presented in Figure 5-16. The left-hand side of Figure 5-16 displays the inclinometer tilt plot, including data from the base read (deep red) and second reading (bright red) of the A and B axes, respectively. The A axis azimuth is 067°, and is parallel to the direction of slip, with the B axis is perpendicular to this. The data on the right of Figure 5-16 are compiled in the same manner as the data on the left and include results from the base read (deep red) and second reading (bright red) of the A and B axes, respectively. The inclinometer readings from this study (Figure 5-16) show there is a small anomaly between 22m and 19m depth. The tilt plot shows there is movement to the northeast and southwest, at 19m and 21m respectively, however southwest slip suggests upslope movement. The profile plot does not show that there has been any movement above 19m. If the anomaly displayed in the data was due to mass movement (1) the tilt plot would spike in the northeast direction only (i.e. downslope) and (2) there would be indication of movement above the failure surface shown by the spike of the tilt plot in the accompanying profile plot. Therefore, the anomaly recorded in the second inclinometer reading, relative to the initial base read, is thought to be related to minor buckling of the inclinometer casing, rather than the result of slope movement. Buckling of the inclinometer casing may be due to the known disturbance of the inclinometer grouting that occurred during installation of the piezometer in BH2.

It is noted that the time frame of inclinometer monitoring for this study is short, and so the slide could be moving extremely slowly, which is defined by Cruden and Varnes (1996) as less than 60mm/yr, or the landslide could be dormant. Further monitoring is required to properly determine any movement of the slope, and recommendations for rail corridor management are discussed in Section 5.8.

5.8 Rail Corridor Management Implications

5.8.1 Stability of the Western Abutment of Slovens Creek Viaduct

Observed ground conditions for the western abutment of the Slovens Creek Viaduct comprise a high water table, at 0.8mbgl, and at least 3m of fill or redistributed landslide material both at track level and on the lower slope of the western abutment to the northeast. Inclinometer data, as discussed in Section 5.7.4, confirms there is no downslope movement within the landslide at depth, at least not within the monitoring timeframe of this project (22 May 2015 – 4 August 2015). However, the landslide at Slovens Creek western abutment could be moving extremely slow (defined by Cruden and Varnes (1999) as less than 60mm/yr), and from this study it cannot be confirmed that there is zero movement within the slope at this time. It is suggested a small network of surface monuments is installed to confirm stability, using a fixed datum on the eastern Slovens Creek abutment, as discussed below.
Analysis of the section profile (C-C’) is based on actual determined parameters, and does not indicate potential for movement under static conditions along that profile. However, slope stability models in Sections 5.5.4 and 5.6.5 of this chapter suggest that the most likely portion of the slope to fail is the shallower fill covered segment to the northeast of the rail corridor, referred to as the lower north-eastern part of the slope in various preceding sections within this chapter. Block failure on the deep-seated failure surface identified by core drilling and logging gave a static factor of safety greater than 2.5, whilst the shallower surficial materials, including non-engineered fill, gave values of ~1.5. In the absence of more detailed landslide investigation and analysis, present stability is confirmed under static loading conditions, although this does not negate regular observation and monitoring.

Pseudo-static analysis of Section C-C’ gave much lower factors of safety for block failure (1.2) and circular shallow failure (1.1) using the methodology outlined in Section 5.6, suggesting that under shaking conditions exceeding 0.2g there could be potential for at least shallow circular failures. Given the limitations of the analysis method, and the limited iterations carried out, it is clear that large magnitude or nearby earthquake events could produce slope damage at the western abutment. As KiwiRail has in place standard operating procedures for track and bridge inspections following such events, and management strategies in place, specific additional measures are not indicated.

For movement within the slope at depth, the inclinometer in BH1 should be monitored on a six monthly basis for one year following completion of this thesis. Annual monitoring of the inclinometer should proceed during the year to follow (April 2017- April 2018). Provision is needed for more frequent inclinometer monitoring if movement is detected, and these are likely to be revealed by track inspections showing ground cracking or track displacements. A budget item is recommended for KiwiRail to implement at least annual surveys, and Geotechnics Limited are suggested as the provider of monitoring services unless KiwiRail has its own specialist equipment and personnel.

In addition to measurement of movement at various depths by way of inclinometer monitoring, surface movement monitoring of the slope, for example within the fill and redistributed landslide material on the lower north-eastern slope of the viaduct landslide, should implemented. A network of up to six surface survey monuments is suggested, with a stable datum on the eastern side of the Slovens Creek Viaduct. Survey markers are suggested because evidence was found in the core of BH1 for inter-block crush and shear zones, which indicate that there could be movement within the landslide at various depths and locations, and not necessarily on the clay-rich surface developed on the upper surface of Puffer Formation, unit P), or in the line of section defined by C-C’. Design of the surface monitoring network is beyond the scope of this thesis.

5.8.2 Long-term Rail Corridor Management

Long term management of the Slovens Creek Viaduct western abutment is dependent upon changes observed since the static stability assessment of the Slovens Creek Viaduct western abutment described
Qualitative Risk Assessment Framework

### Measures of Likelihood

<table>
<thead>
<tr>
<th>Level</th>
<th>Descriptor</th>
<th>Description</th>
<th>Approximate Return Period of Occurrence (event: no. years$^1$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Frequent</td>
<td>The event is on-going, or is expected to occur during the next year</td>
<td>1: 1</td>
</tr>
<tr>
<td>B</td>
<td>Probable</td>
<td>The event is expected to occur.</td>
<td>1: 1 to 1: 5</td>
</tr>
<tr>
<td>C</td>
<td>Occasional</td>
<td>The event is expected to occur under somewhat adverse conditions</td>
<td>1: 5 to 1: 20</td>
</tr>
<tr>
<td>D</td>
<td>Remote</td>
<td>The event is expected to occur under adverse conditions</td>
<td>1: 20 to 1: 100</td>
</tr>
<tr>
<td>E</td>
<td>Improbable</td>
<td>The event is expected to occur under high to extreme conditions</td>
<td>1: 100 to 1: 300</td>
</tr>
</tbody>
</table>

### Measures of Consequence (see notes below)

<table>
<thead>
<tr>
<th>Level</th>
<th>Descriptor</th>
<th>Severity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Catastrophic</td>
<td>Evacuation of a significant length of the rail by underslip. Derailment and loss of several carriages is likely to occur. Major emergency works with track closure extending beyond two weeks. Very high remedial works costs.</td>
</tr>
<tr>
<td>2</td>
<td>Critical</td>
<td>Evacuation of a limited portion of the railway line by underslip or blockage of the line over a large extent by debris from overslip (&gt;200 cubic metres). High potential for train derailment. Track closed for up to two weeks for emergency works.</td>
</tr>
<tr>
<td>3</td>
<td>Major</td>
<td>Medium size overslip failures (50 to 200 cubic metres of material) requiring removal. Some potential for train derailment. Track closed for up to three days. Overslip to within 1 m of track centreline, but track itself not affected. Remedial works may not be required immediately.</td>
</tr>
<tr>
<td>4</td>
<td>Minor</td>
<td>Small overslips (up to 50 cubic metres) onto track requiring removal; track closed for less than a half a day. Overslip to between 2 and 4 m of track centreline</td>
</tr>
<tr>
<td>5</td>
<td>Negligible</td>
<td>Small slips adjacent to track requiring removal. Removal of material can be delayed.</td>
</tr>
</tbody>
</table>

### Risk Matrix

<table>
<thead>
<tr>
<th>Likelihood</th>
<th>5: Negligible</th>
<th>4: Minor</th>
<th>3: Major</th>
<th>2: Critical</th>
<th>1: Catastrophic</th>
</tr>
</thead>
<tbody>
<tr>
<td>A - Frequent</td>
<td>M</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>B - Probable</td>
<td>L</td>
<td>M</td>
<td>M</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>C - Occasional</td>
<td>L</td>
<td>L</td>
<td>M</td>
<td>M</td>
<td>H</td>
</tr>
<tr>
<td>D - Remote</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>E - Improbable</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>M</td>
</tr>
</tbody>
</table>

### Risk Level Implications

<table>
<thead>
<tr>
<th>Risk Level</th>
<th>Implications for Risk Management</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category A Track Segments</td>
<td>Category B Track Segments</td>
</tr>
</tbody>
</table>

- **E** Extreme Risk
  - Detailed investigation, design, planning and implementation of treatment options to reduce risk to acceptable levels. Work to be undertaken as a priority.
  - Detailed investigation, design, planning and implementation of treatment options to reduce risk to acceptable levels. Work to be undertaken as a priority.
  - Detailed investigation, design, planning and implementation of treatment options to reduce risk to acceptable levels.

- **H** High Risk
  - Risk acceptability to be considered on case by case basis. Risk may be tolerable at some sites with suitable treatment plan. At other sites, treatment options may need to be investigated to reduce the risk to acceptable levels.
  - Risk may be broadly tolerable provided treatment plan is implemented or, risk may need to be reduced in accordance with ALARP. May require investigation and planning of treatment options.
  - Risk acceptability to be considered on case by case basis. Risk may be tolerable at some sites with suitable treatment plan. At other sites, treatment options may need to be investigated to reduce the risk to acceptable levels.

- **M** Medium Risk
  - Risk may be broadly tolerable provided treatment plan is implemented or, risk may need to be reduced in accordance with ALARP. May require investigation and planning of treatment options.
  - Acceptable. Treatment requirements to be defined to maintain or reduce risk.
  - Acceptable. Treatment requirements to be defined to maintain or reduce risk.

- **L** Low Risk
  - Acceptable. Treatment requirements to be defined to maintain or reduce risk.
  - Acceptable. Manage by normal maintenance procedures.
  - Acceptable. Manage by normal maintenance procedures.

*Figure 5.17 Qualitative Risk Assessment Framework for rail corridor management as provided by KiwiRail.*
in Section 5.8.1. If evidence of slope movement becomes apparent from the inclinometer in BH1, or the survey markers suggested in Section 5.8.1, then drainage options and dewatering of the slope should be considered.

As discussed in various sections of this chapter, the water table of the western abutment of the Slovens Creek Viaduct is high, at 0.8mbgl, and the significance of the hydraulic connectivity of the slope, which became apparent during installation of the piezometer, should not be disregarded. The seepage of drilling fluid observed could be indicative of potential inter-block movements, which could cause disruption to operations of the Midland Line during or following a prolonged rainfall or seismic events. In response to a prolonged rainfall, such as that of 1952 (1 in 100 year return period) or a significant seismic event (1 in 50 year return period; PGA 0.33g), there should be immediate site reconnaissance, including monitoring of the inclinometer and surface survey markers. Depending on the severity of the event track closure should be considered in accordance with the KiwiRail Qualitative Risk Assessment Framework (Figure 5-17).

Dewatering and drainage design are beyond the scope of this study, however it is suggested that up to three drains, approximately 3m below track level (i.e. at the contact between the fill and clay-rich landslide debris) could be installed. Any drain installed within the ground of the Slovens Creek Viaduct western abutment should comprise a collapsible sock of an appropriate length to allow discharge of water to Slovens Stream, rather than within the lower parts of the slope. The lower parts of the slope includes the lower north-eastern slope northeast of the track, where instability has been identified in engineering geomorphological maps (Map 6B), limit-equilibrium models and pseudo-static models (Figure 5-12; Figure 5-14, respectively), and beneath the viaduct itself (Figure 5-2E), where a retaining structure and evidence for water induced instability has also been identified.

5.9 Summary

- The steep Puffer Formation outcrop at the western abutment of Slovens Creek Viaduct is known to shallow out with depth given the presence of the Puffer Formation (P4) at 27.3m depth in BH1. The 0.2m thick layer of clay above the Puffer Formation bedrock is interpreted as the failure surface of a previous landslide, and the materials above that surface (other than fill) are interpreted as be blocky landslide debris. Landslide debris comprises crushed and sheared tuff of the Thomas Formation, which is the result of inter-block shearing within the slope, due to movements that occurred following the incision of Slovens Stream.

- The upwelling of drilling fluid during placement of the piezometer in BH2 suggests hydraulic connectivity throughout the blocky landslide debris. The water table measured at 0.8mbgl by the piezometer is actually considered to be that of a perched water table above the clay-rich failure surface, however during limit-equilibrium and pseudo-static analyses the water level measured by the piezometer has been assumed as the unconfined water table within the slope.
This means the models are ‘worst case scenario’ with regard to saturation of the slope due to groundwater. Water could go up to ground level, even become flowing artesian, and according to the analyses of this study the slope would still be stable.

- Limit-equilibrium modelling results suggest the slope at the western abutment of Slovens Creek Viaduct is stable, and these results are in agreement with the data from the inclinometer. Sensitivity to the level of groundwater was found, however the slope remains stable in the range 0.8m±0.8mbgl. Failure on the clay-rich layer, which has been interpreted as the failure surface of the main slide, is considered unlikely due to the high cohesion of the clay-rich layer (400kPa) in addition to the anthropogenic reduction of driving forces and increase in resisting forces by way of earthworks and removal (i.e. unloading) of up-slope landslide deposits, and the geometry (‘arm-chair’ shape) of the underlying Puffer Formation, unit P4.

- The results of the pseudo-static modelling, in conjunction with sensitivity analyses enabled the PGA that would yield failure to be found. When compared to ULS design return periods defined by NZS 1170.5:2004, circular slip search methods found that yield PGA for the lower eastern portion of the slope is 0.207g or 0.216g, which suggests this portion of the slope would fail in any seismic event greater than a 1 in 20 year return period, which has a PGA of 0.189g. ULS design earthquake is 0.944g for the Slovens Creek Viaduct area, and the block search method found the yield PGA to be 0.400g for a user-defined failure surface above the interpreted clay-rich failure surface, which suggests the slope could fail along that surface in an event of a 1 in 50 year return period.

- The limit-equilibrium and pseudo-static models provide a good indication of the stability of the slope, with the lowest FoS being 1.117 on the lower eastern portion of the slope, which was found with the measured (mean) values for each of the random variables, which were cohesion, phi, water table location, and seismic load (estimated and in pseudo-static analysis only). Being such simplistic analyses however, these models are not entirely reliable. A stronger data set produced from further geotechnical investigation is required to enable more detailed modelling and more reliable models.

- Observed ground conditions for the western abutment of the Slovens Creek Viaduct suggest there is no movement within the landslide at depth, at least not within the monitoring timeframe of this project (22 May 2015 – 4 August 2015), however the landslide could be moving extremely slow, and so slope stability monitoring should be continued in two parts: (1) the inclinometer in BH1 should be monitored on a six monthly basis for one year following completion of this thesis, and annual monitoring of the inclinometer during the year to follow (April 2017- April 2018), with prevision for more frequent inclinometer monitoring if movement is detected and (2) superficial movement of the slope should be monitored with the use survey markers. Survey markers are suggested because evidence was found in the core of...
BH1 for inter-block crush and shear zones, which suggest the landslide could be moving at various depths and locations, and not necessarily on the interpreted clay-rich failure surface.

- Long term management of the stability of Slovens Creek Viaduct western abutment is dependent upon changes observed since the static conditions of Slovens Creek Viaducts western abutment described in Section 5.8.1. If stability of the slope becomes problematic for the operations of the Midland Line, it is recommended drains options with a collapsible sock of an appropriate length to allow discharge of water to Slovens Stream be implemented. Water should not be discharged to lower parts of the slope, where instability has been identified in engineering geomorphological maps (Map 6B), limit-equilibrium models and pseudo-static models (Figure 5-12; Figure 5-14, respectively), and photographs (Figure 5-2E).
6 Summary and Conclusions

6.1 Objectives and Methodology

The objectives of this thesis have been to:

- Assess evidence for an apparent translational landslide and the long term stability of Tertiary strata on the western abutment of the Slovens Creek Viaduct.
- Assess further evidence for slope instability, and long term stability of the rail corridor, in the Tertiary sequence between Slovens Creek Viaduct and Avoca.
- Provide a suitable basis for future geotechnical hazard and risk assessment of the 3km long rail corridor between Slovens Creek Viaduct and Avoca.

The objectives were achieved by:

- Compiling geological and engineering geomorphology maps of (1) the Slovens Creek Viaduct area, and (2) the Slovens – Avoca rail corridor.
- Completing geotechnical investigations at Slovens Creek Viaduct, which included an investigation borehole to 36m, an inclinometer to 32m, and a piezometer to 15m depth.
- Carrying out laboratory work for geotechnical classification of subsurface material and landslide modelling of the Slovens Creek Viaduct western abutment and landslide.
- Identification of active and potentially hazardous landslides along the rail corridor between the viaduct and Avoca, which require additional future hazard and risk assessment.

6.2 Geology and Engineering Geomorphology of the Slovens-Avoca Rail Corridor

- Of the Tertiary sequence found in the Castle Hill Basin, only the Puffer and Thomas Formations of the Porter Group, and the Enys Formation of the Motunau Group, are present in the Slovens-Avoca Corridor. Late Cretaceous-Eocene Formations have been stripped from the eastern extension of Castle Hill Basin as a result of uplift and erosion in the early Oligocene. Slovens Creek Valley was initially carved out by the Slovens Creek lobe during the Waimakariri Glaciation, further incised by meltwater, and subsequently by fluvial processes during postglacial times. Landslides within the valley post-date that glaciation, however the age cannot be further constrained except to record that the majority would be younger than 10ka and several are currently active.
- Puffer Formation member P3 is sandy and weak, which results in successive multiple rotational failures. The clay-rich Thomas Formation Tuff is prone to slaking, and this is exacerbated by
the presence of tens of metres of relatively permeable Quaternary gravels unconformably overlying generally weak Tertiary bedrock (mudstone, tuff, sandstone). There are tension cracks observed in the Quaternary gravels in Landslide Area 2, which have been induced by failure of the downslope buttress of weak Tertiary strata.

- Progressive incision and lateral erosion by Slovens Stream into the Tertiary strata has led to the development of multiple retrogressive slides or slumps in the Slovens-Avoca rail corridor. In some cases failed Thomas Formation Tuff is blocky, such as in Landslide Area 9 near Slovens Creek Viaduct. Active rotational slump failures in the clay-rich Thomas Formation tuff are interpreted at metrage 71.400, and at 71.200 where multiple gauge and top faults are recorded for this section of track.
- Bryce Gully is actively evolving by erosion and the sediment load is high, sourced mainly from Quaternary gravels at the head. In addition, there are active landslides associated with the incision of the stream in Bryce Gully, although these do not pose immediate threat to the rail corridor. There is a possibility that the high sediment load could affect the drainage at metrage mark 70.722, and indirectly affect track operations due to culvert blockage.
- Landslides occupy approximately 90 percent of the slopes to the west of the Midland Line in the ~3km long Slovens-Avoca corridor. The majority of landslides are found within, or are related to, the weak clayey Thomas Formation Tuff, which undergoes relatively rapid slaking and repeated retrogressive landsliding. These slope failures could potentially impact rail operations at times of extreme precipitation due to sliding and/or flowage of debris.

6.3 Western Abutment of Slovens Creek Viaduct

- The steep Puffer Formation outcrop at the western abutment of Slovens Creek Viaduct is known to shallow out with depth, given the presence of the Puffer Formation unit P4 at 27.3m depth in BH1. The 0.2m thick layer of clay-rich material above the Puffer Formation bedrock is interpreted as the failure surface of the main landslide phase. Landslide debris comprises crushed and sheared tuff of the Thomas Formation, the crushing and shearing of which is due to inter-block displacements during slope movements.
- The water table measured at 0.8mbgl in the piezometer is considered to be a perched water table above the clay failure surface, but for limit-equilibrium and pseudo-static analyses the water level measured by the piezometer has been assumed as the unconfined water table within the slope. This means that the slope stability models represent a ‘worst case scenario’ with regard to saturation of the slope due to groundwater. According to the analyses in this study, water could rise to ground level, and even become flowing artesian in steady state, and the slope would still remain stable.
Limit-equilibrium modelling results indicate that the slope at the western abutment of Slovens Creek Viaduct is stable, and these results are in agreement with data from the inclinometer. Sensitivity to the level of groundwater was identified, but the slope was found to be stable within the range 0.8m±0.8m bgl, with the upper boundary of this range equivalent to ground level. Failure on the clay-rich layer, which has been interpreted as the failure surface of the original slide, is considered unlikely due to the high cohesion of the clay-rich layer (~400kPa) in addition to the anthropogenic reduction of driving forces and increase in resisting forces by way of earthworks carried out in 1952.

A ULS design earthquake is computed as 0.944g for the Slovens Creek Viaduct area. Circular slip surface search methods in pseudo-static modelling found the yield PGA for the lower north-eastern portion of the slope to be 0.207g or 0.216g, which suggests this portion of the slope could fail in any seismic event greater than a 1 in 20 year return period (0.189g). This area lies beyond the rail corridor however, and does not represent an immediate threat to operations of the Midland Line.

The pseudo-static block slip surface search method found the yield PGA to be 0.400g on a user-defined surface slightly above the interpreted clay-rich failure surface, which suggests the portion of the slope supporting railway infrastructure could fail along that surface in any seismic event greater than a 1 in 50 year return period (0.330g). As in limit-equilibrium modelling, there was no indication of failure along the interpreted clay-rich failure surface, which is considered to be due to its high cohesion and earthworks carried out in 1952.

Limit-equilibrium and pseudo-static models provide a good indication of the present stability of the slope at the western abutment of Slovens Creek Viaduct, but because these analyses are simplistic the models are not entirely reliable. A stronger data set produced from further geotechnical investigation would be required to enable more detailed modelling and analysis.

Inclinometer monitoring at the western abutment of the Slovens Creek Viaduct (BH1) suggests that there is no movement within the landslide at depth over the short timeframe of observations (22 May 2015 – 4 August 2015). This is consistent with the absence of track faults and ground deformation features, but it is possible that the landslide could be moving extremely slowly. It is therefore considered appropriate to implement a monitoring strategy as part of ongoing track management for the Avoca-Slovens Creek rail corridor.

Slope stability monitoring should be continued in two parts. Firstly, the inclinometer in BH1 should be monitored on a six monthly basis for one year following completion of this thesis, and annually thereafter, with provision for more frequent inclinometer measurements if movement is detected. Secondly, surface movements above and below the track within the landslide should be monitored by a network of survey markers and a fixed datum east of Slovens Creek. Survey markers are suggested because evidence was found in the core of BH1 for inter-block crush and shear zones, which suggest the landslide could be moving at various
depths and locations, and not necessarily on the interpreted clay-rich failure surface. Design of a surface monitoring programme is beyond the scope of this thesis.

6.4 Principal Conclusions

- The Slovens-Avoca rail corridor geology comprises weak Tertiary strata that are prone to slumping and slaking which, in conjunction with the postglacial incision of Slovens Stream, has resulted in numerous landslides affecting slopes adjacent to the 3km long rail corridor.
- Observed slumping of the Thomas Formation Tuff is consistent with gauge and top faults at metrage marks 71.400 and 71.200 in the Slovens-Avoca rail corridor. These slumps have been identified as active, and pose possible threats to operation of the Midland Line by undercutting. The high sediment load of Bryce Gully has also been identified as a possible threat to operations of the railway by blockage of the culvert at metrage mark 70.222.
- The interpreted subsurface configuration of the underlying bedrock (Puffer Formation, P4) at the western abutment of Slovens Creek Viaduct, in conjunction with the unloading of driving forces by way of earthworks in 1952 and high cohesion of the (interpreted) clay-rich failure surface slope stability models, suggest the viaduct slope is unlikely to fail on the interpreted deep-seated (~27mbgl) failure surface in future. This is consistent with geomorphological observations and inclinometer data.
- Modelling suggests that a PGA of 0.400g at Slovens Creek Viaduct could induce failure on a (user-defined block) failure surface within the shallower tuffaceous landslide deposits. If failure was to occur on this surface, the effects to track operations could be critical.
- Further work is required to refine the characteristics and subsurface configuration for the western abutment at Slovens Creek Viaduct, and to evaluate the hazards and risks imposed to operations of the Midland Line due to potential landsliding and/or bridge damage.

6.5 Recommendations for Future Work

Future work in the Slovens-Avoca rail corridor mainly involves careful monitoring of evidence found for slope instability on the adjacent western slopes. Specific recommendations are as follows:

- Particular attention should be paid to the monitoring of the active rotational slump failures identified in the clay-rich Thomas tuff that have been interpreted at metrage marks 71.400 and 71.200. These coincide with multiple gauge faults and top faults in the track logs.
- Further geotechnical investigation, by way of exploration boreholes, is required to improve the reliability of the ground model for the western abutment of Slovens Creek viaduct, and to determine a long term management plan for hazards and risks in this section of the Slovens-Avoca rail corridor. A hazard and risk assessment of the Slovens-Avoca rail corridor and
western abutment of Slovens Creek Viaduct should then be carried out using the engineering geomorphology maps from this study as the basis for hazard zonation maps or similar.

- Long-term management of the stability of Slovens Creek Viaduct western abutment is dependent upon future changes observed by the monitoring measures recommended. If stability of the slope becomes problematic for the operations of the Midland Line, it is recommended that drainage measures are implemented with a system to allow discharge of water to Slovens Stream.

- In the event of a large PGA earthquake event affecting Midland Line operations in the Staircase to Slovens section of the rail corridor, re-measurement of the BH1 inclinometer and the recommended surface monitoring network at the Slovens Creek Viaduct will be required.
References


White, Y., 1998. Life Along the Steel Road, Renwick, NZ.