Deep Lime Stabilisation Applications for Roading,
East Coast, North Island,
New Zealand.

A Thesis
submitted in partial fulfilment
of the requirements for the Degree of
Master of Science
in
Engineering Geology
in the University of Canterbury

by
Ken Donald Mitchell.
University of Canterbury
Frontispiece: Panikau Summit, SH 35.
"....What's the country like up Gisborne way? Not bad. All the farms are sliding into the sea and the roads are no better than this....."

From "Hang on a Minute Mate" by Barry Crump
This study presents a review of literature on two methods of deep lime stabilisation: a] Lime Slurry Pressure Injection (LSPI), and b) Lime Column stabilisation. The methods were then evaluated in terms of their potential to stabilise a candidate site at Panikau Summit, State Highway 35, Raukumara Peninsula, New Zealand. Panikau is located adjacent to a local Cretaceous basement high surrounded by very deformed lower Tertiary marine fine grained sediments. An engineering field investigation of a slope failure at Panikau was performed, including drilling, dutch cone penetrometer soundings, trenching, field mapping and surveying of a peg network.

Slope failure at Panikau is related to the instability of the natural (12°-15°) slopes, and the construction of a highway embankment without adequate site drainage. Shear surfaces identified at 2 m-3.5m by trenching are the failure zone for a slow moving earthflow below the failed embankment. Soils were moist to saturated clayey silts (plasticity index range 18-50) and cobble to boulder size silty sandstone blocks. Several soils had natural water contents in excess of the plastic limit. Smectite (montmorillonite), illite and minor kaolinite were identified were the principal clays present.

Increasing percent hydrated lime and curing time gave greater peak UCS values in remoulded soil specimens. Shear box testing of remoulded natural (c' =12.5 kPa, Ø' =13.5°, c''=0 kPa, Ø''=18°) and 1.5% hydrated lime specimens (c' =8.5 kPa, Ø' =33°, c''=11.8 kPa, Ø''=14.5°) showed marked improvement in shear parameters with addition of lime.

The lime column method has the best potential to stabilise Panikau Summit because of the drainage function of the installed columns. LSPI would require drainage works in addition to the stabilisation programme.
TABLE OF CONTENTS

Frontispiece ........................................ iii
ABSTRACT ........................................... xiii
TABLE OF CONTENTS ................................ iv
LIST OF FIGURES ...................................... viii
LIST OF TABLES .................................... xii

Chapter One: Introduction ... 1
  1.1 Background .................................. 1
  1.2 Site Selection ................................ 1
  1.3 Thesis Objectives ............................ 4
    1.3.1 Methodology ............................. 5

Chapter Two:
Deep Lime Stabilisation,
A Literature Review ......................... 6
  2.1 Introduction ................................ 6
  2.2 Basic Lime Stabilisation .................. 7
  2.3 Lime Column Stabilisation ............... 9
    2.3.1 Introduction ........................... 9
    2.3.2 Lime Column Construction ............. 15
      a) Equipment ................................ 15
      b) Method .................................. 17
      c) Stabilisation Mechanisms ................ 20
d) Application of the Lime Column Method 22
e) Example of the Application of the Lime
   Column Method ................................ 26
  2.4 Lime Slurry Pressure Injection Stabilisation
      (LSPI) ........................................ 28
    2.4.1 Introduction ........................... 28
    2.4.2 Method and Mechanisms ............... 30
    2.4.3 Comparison of Hydrated Lime and
      Quicklime slurries .......................... 34
    2.4.4 Application of LSPI .................... 36
    2.4.5 Examples of the Application of LSPI 38
  2.5 Synthesis .................................... 40
Chapter 3: Site Investigation

3.1 Timing and Aims

3.1.2 Location

3.1.3 East Coast Geological Setting

3.2 Engineering Geological Field Investigation

3.2.1 Site History

a) Site Development

b) Failure History

3.2.3 Geology

3.2.4 Geomorphology

3.2.5 Rainfall

3.3 Surveying

3.4 Subsurface Investigation

3.4.1 Penetrometer Soundings

a) 1967 Pre-Realignment Dutch Cone Penetrometer Investigation

b) 1972 Dutch Cone Penetrometer Investigation

c) Post-Cyclone Bola Dutch Cone Penetrometer Investigation

3.4.2 Trenching

a) Introduction

b) Trench 1

c) Trench 2

d) Trench 3

e) Synthesis

3.4.3 Drilling

a) Programme

b) Borehole Lithologies

3.5 Slope Failure Classification and Mechanism

3.5.1 Classification

3.5.2 Failure Mechanism

3.6 Synthesis
Chapter 4: Laboratory Investigations

4.1 Introduction .................................. 77
  4.1.1 Purpose .................................. 77
  4.1.2 Method .................................. 77

4.2 Natural Soil Parameters of Soils at Panikau Summit .................. 77
  4.2.1 Introduction ................................ 77
  4.2.2 Changes in Atterberg Limits with Increase in Percentage Hydrated Lime ... 79
  4.2.3 X-Ray Diffraction Clay Identification .................. 81
    a) Purpose .................................. 81
    b) Method .................................. 81
    c) Identification ............................ 81

4.2.4 Initial Lime Consumption Testing .......................... 82
    a) Introduction ................................ 82
    b) Method .................................. 82
    c) Results .................................. 84

4.3 Strength Testing .................................. 84
  4.3.1 Introduction ................................ 84
    a) Lime Columns ................................ 84
    b) LSP ....................................... 85
  4.3.2 Sample Preparation Method ......................... 87
    a) Introduction ................................ 87
    b) Discussion ................................ 88

4.3.3 Strength Development with Increase in Percentage Hydrated Lime .......... 89
    a) Method .................................. 89
    b) Results .................................. 89
    c) Discussion ................................ 91

4.3.4 Type of Lime Used ................................ 92

4.3.6 Lime Slurry Glaze Testing ........................ 93
    a) Method .................................. 93
    b) Results .................................. 98
    c) Discussion ................................ 98

4.3.7 Shear Box Testing ............................. 99
    a) Method .................................. 99
    b) Results .................................. 101
Chapter Five:
Summary and Conclusions... 106

5.1 Deep Lime Stabilisation
for Panikau Summit... 106
5.1.1 LSPI Stabilisation... 106
5.1.2 Lime Column Stabilisation... 107
5.1.3 Synthesis... 108

5.2 Conclusions... 109
5.2.1 Literature Review of Deep Lime Stabilisation... 109
5.2.2 Site Investigation... 110
5.2.3. Laboratory Investigations... 111

5.3 Summary... 112
5.3 Recommendations for Further Work... 113

Acknowledgements... 115

References... 117

Appendix 1: Rainfall Data... 122
Appendix 2: Penetrometer Data and Interpretation... 124
Appendix 3: Drilling and Trenching... 140
Appendix 4: Survey... 148
Appendix 5: X-Ray Diffraction Data... 151
Appendix 6: Design Drawings, Panikau Hill Reconstruction... 155
List of Figures

Figure 1.1: Roads District No:5, East Coast, North Island. ......................... 2
Figure 1.2: Location Map ................................................. 3
Figure 1.3: Air Photograph overlay on survey Map Pocket
Figure 2.1: Lime Columns used to reduce differential settlements ................. 11
Figure 2.2: Lime columns used to increase the bearing capacity and shear strength of a foundation. (after Broms, 1984) ............... 13
Figure 2.3: Possible lime column configuration for slope stabilisation. (after Broms, 1984) .................. 13
Figure 2.4: Lime columns used to stabilise a trench excavation in soft ground conditions. (after Broms, 1984) ......................... 14
Figure 2.5: Foundation design using lime columns for a single level structure. (after Broms, 1984) .................. 14
Figure 2.6: The Linden-Alimak LPS 4 lime column machine. (after Broms, 1984) .................. 16
Figure 2.7: The Volvo BM LM 641 lime column machine. (after Broms, 1984) .................. 16
Figure 2.8: Lime column installation equipment, and mixing head. Note the pitched blades. (after Broms, 1984) .................. 18
Figure 2.9: The lime column penetrometer. (after Broms, 1984) .................. 18
Figure 2.10: The screw-plate load device used for in-situ testing of lime columns. (after Broms, 1984) .................. 19
Figure 2.11: The assumed stress-strain relationship in lime stabilised soil. (after Broms, 1984) .................. 19
Figure 2.12: Lime column configuration for an embankment indicating the potential drainage of the foundation area. (after Broms, 1984) .................. 23
Figure 2.13: The effects of preloading a site with lime columns. (after Broms, 1984) .................. 23
Figure 2.14: Site conditions for the multi-lane highway embankment. (after Broms, 1984) .................. 25
Figure 2.15: The Creep Limit curve derived from laboratory and field testing at the embankment site. (after Broms, 1984) .............................................. 25
Figure 2.16: Load distribution between lime columns and the surrounding unstabilised soil. (after Broms, 1984) .............................................. 27
Figure 2.17: Longitudinal section of the embankment site showing calculated settlement and lime column depth. (after Broms, 1984) .............................................. 27
Figure 2.18: "Typical" Lime Slurry Pressure Injection (LSPI) application using a tractor unit (which would be connected to a slurry tank). (after Boynton and Blacklock, 1985) .............................................. 29
Figure 2.19: Seam stabilisation effects of LSPI. (after Boynton and Blacklock, 1985) .............................................. 29
Figure 2.20: Percent Lime/Lime Modification Optimum (LMO) verses Unconfined Compression Strength (UCS) for quicklime and hydrated lime. (after Petry and Lee, 1989) .............................................. 31
Figure 2.21: Change in Optimum Moisture Content of a Texas Soil with different lime type. (after Petry and Lee, 1989) .............................................. 31
Figure 3.1: The Australia-Pacific Plate Boundary, East Coast, North Island. .............................................. 41
Figure 3.2: Site Geomorphology Map Pocket
Figure 3.3: Average Rainfall Year (1968-1991) 41
Figure 3.4: Location of Trenches and Survey Pegs in the Headscarp Area. .............................................. 43
Figure 3.5: Excavator covering Trench 1, with 1972 piezometers in the foreground. .............................................. 45
Figure 3.6: Panikau Summit looking north. Note the non-vertical fence and pole, and remedial tree planting in headscarp area. .............................................. 45
Figure 3.7: Geological Map and Generalised Stratigraphy (after Ridd, 1967). .............................................. 46
Figure 3.8: Photograph from Peg A2 looking north. Note filled backscarp, and vehicle parked on old Panikau intersection. The borehole is recessed in a toby
Figure 3.9: Looking upslope and east toward SH 35. Note the low relief, abundant marsh grass, and the vehicles on the highway below the pine plantation.

Figure 3.10: Photo and line drawing of the left lateral scarp of an earthflow below the highway embankment.

Figure 3.11: Hillside southeast of Peg C2. Note the hummocky ground, and deformed fenceline and power pole.

Figure 3.12: Looking south from the fenceline above Peg C2. Note the standing water and marsh grass. Whangara coast of Pacific Ocean in background.

Figure 3.13: Shrinkage cracks common in summer conditions adjacent to Peg B1.

Figure 3.14: Variation in monthly rainfall totals at NZ Met. Service Station at Wokairau, Whangara.

Figure 3.15: Total peg displacement relative the original survey location for each peg. The lower part is an enlarged version of the top part.

Figure 3.16: Trench 1. Note the dark colour change at the base of the trench. Sliding occurs at change.

Figure 3.17: Trench 1. 3 m. Clean water flow (at point of pencil) in the shear zone. Note contrast from smeared clay above the pencil to granular sandy material below.

Figure 3.18: Graphic Log of Trench 1.

Figure 3.19: Graphic Log of Trench 2.

Figure 3.20: Eastern (road) end of Trench 2. Note the blue colour change in the base of the trench.

Figure 3.21: Sheared zones within Trench 2. a]1.5-1.7m "wedge shears" b] sliksensided basal shear plane.

Figure 3.22: Trench 3. Note the thick organic horizon, and the blue colour change in the base of the trench.

Figure 3.23: Jacro Drill Rig

Figure 3.24: Graphic Log of Trench 3.

Figure 3.25: Borehole moisture content profile.

Figure 4.1: Plasticity Chart for Panikau Summit Soils.
Figure 4.2: Trends in Liquid Limit, Plastic Limit and Plasticity Index with increasing percent hydrated lime. ...................... 80
Figure 4.3: Initial Lime Consumption Test Equipment ................ 80
Figure 4.4: Estimated Creep Limit chart for a single column (after Broms, 1984). ............................................. 83
Figure 4.5: Estimated relative shear strength with curing period (after Broms, 1984) ............................................. 83
Figure 4.6: Types of lime glaze stabilised compression specimens. (after Blacklock and Wright 1986) ......................... 86
Figure 4.7: Lime glaze consolidation and swell specimen. (after Blacklock and Wright 1986) ............................................. 86
Figure 4.8: Unconfined Compressive Strength against strain for 4%, 6%, and 8% hydrated lime. ......................... 90
Figure 4.9: Failed T3/2.1-3.1 natural (recompacted) specimen. ............................................................ 94
Figure 4.10: Failed 6% hydrated lime 4-day specimen. ........ 94
Figure 4.11: Unconfined Strength development over time for increasing percent hydrated lime. ......................... 95
Figure 4.12: 4-Day Water and lime glazed UCS results ........ 95
Figure 4.13: 7-Day lime glaze UCS results ......................... 96
Figure 4.14: 14-Day water and lime glaze UCS results ........ 96
Figure 4.15: Seam-cut glaze specimen UCS results ............. 97
Figure 4.16: ELE Shear Testing Frame, Napier Laboratory ......... 97
Figure 4.17: Shear Box Graphs .......................................... 100
Figure 4.18: Pilcon Shear Vane Undrained Shear Strengths ........ 103
List of Tables

Table 2.1: Types of Lime for stabilisation 10
Table 3.1: 1972 Penetrometer Investigation Summary 60
Table 3.2: 1988 Penetrometer Investigation Summary 61
Table 4.1: Soil Parameter Summary Table 78
Table 4.2: Shear Box Results 100
Table 4.3: Infinite slope Factor of Safety using Shear Box parameters 100
Chapter One: Introduction

1.1 Background

This study came about in response to the need to investigate new methods of stabilisation capable of providing solutions for the wide spread under slip and earthflow features in the East Cape (Roads District No:5) area. The main transportation link in the East Cape area is State Highway 35, shown in Figure 1.1. Many sections of the highway are subject to relatively slow creeping failures and subsidence that often do not warrant major earthworks, but require continual maintenance. In tougher economic times the pressure is on to identify cost effective solutions for these continuing maintenance problems.

The catalyst for the study was a proposal by Evans (1989) to apply the Lime Slurry Pressure Injection (LSPI) technique to a number of sites on State Highway 35 and State Highway 2, including the Panikau Hill Summit (Figure 1.2), the subject of this study. Transit New Zealand (East Coast Office) then made funds available to research the LSPI method as it might be applied in East Coast conditions. The topic was broadened to include the Lime Column method as the study progressed.

1.2 Site Selection

In addition to the literature review of deep lime stabilisation methods, the site at Panikau Summit was selected to be the subject of an engineering geological site investigation to assess its potential as a candidate site for deep lime stabilisation.

The site was selected after considering three criteria;

a) Sites identified in the original report (Evans, 1989)
Figure 1.1: Roads District No:5, East Coast, North Island.
Figure 1.2: Location Map
b) Site history

c) Accessibility

The Panikau Summit site was unique in that the highway had been relocated in 1968-69 on the western side of the summit to escape severe earthflow movement on the eastern side (Figure 1.3). This enabled the timing of the disruption to the road to be defined within a known time period before present. The road construction history is also relatively well documented in Roads District 5 construction and maintenance files held by WORKS Consultancy Services for Transit New Zealand. A significant problem with almost every other site in the district was the lack of historic file material for specific sites. In most cases there is no construction record, and only maintenance records detailing costs for sections of road over an annual or longer period.

Panikau Summit has a relatively large road reserve down slope of the highway which enabled excavation of material for testing without disruption to fencing or grazing land, and allows for surface reshaping downslope if required during any future deep stabilisation trialing. The closeness to Gisborne was also important in terms of project costs as transport and establishment expenses are significant on the East Coast.

The final decision to investigate the Panikau Summit site was made after consultation with University of Canterbury (D.H. Bell, Senior Lecturer in Engineering Geology) and Works Consultancy Services staff (R.J. McKelvey, Investigations Engineer, Napier).

1.3 Thesis Objectives

The primary objectives of this study were:

1) To review and summarise the literature regarding Lime Slurry Pressure Injection and the Lime Column method.
2] To conduct an engineering geological site investigation of the Panikau Summit site to assess its suitability as a candidate for some form of deep lime stabilisation.

3] To evaluate methods of laboratory testing relating to deep lime stabilisation.

1.3.1 Methodology

Initially a library based literature review was undertaken, followed by a search of several scientific and engineering database services. This material was summarised and presented in Chapter Two.

After selecting the site for the field investigation, engineering geological mapping, drilling, trenching, and survey work were initiated. The results are the contained in Chapter Three.

Materials sampled in the field phase were then subject to a laboratory testing programme to identify the nature of the clay minerals present, the soil parameters, shear strength, and response to the application of lime. This is the basis of Chapter Four.

Chapter Five contains a discussion regarding the potential for deep lime stabilisation at Panikau Summit in light of the information gathered in Chapters Two, Three, and Four, and the conclusions from the study, with recommendations for further work.
Chapter Two: Deep Lime Stabilisation, A Literature Review.

2.1 Introduction

Deep Lime Stabilisation is a method of treating poorly performing subgrade materials without major reconstruction works. The method is used in Scandinavia, Asia and the United States of America in areas associated with poor foundation performance in silty clay and clay. Two principal methods are used;

1] Lime Columns (or Deep Mixing Method.)
2] Lime Slurry Pressure Injection (LSPI).

The Lime Column method has been developed technically in the Scandinavian countries where construction on glacial varved clays is common. The method was developed in the 1940s and 1950s, and has been a well documented technique in scientific and proprietary literature from Scandinavia. Testing and evaluation has been well researched by academia and private enterprise as the method has become more common.

Lime slurry pressure injection (LSPI) is a development mainly from the United States of America, and notably from railway and roading organisations in the southern states. The method developed from simple grouting to treat poorly performing subgrade and foundation materials. LSPI has not had the degree of independent evaluation that lime column methods have had, reflecting the way the method has developed as an "in-house" technique by relatively few organisations. The technique has now developed, with increasing mechanisation, to become an accepted ground improvement tool.
2.2 Basic Lime Stabilisation

Lime is a common agent for the modification or stabilisation of soils, and is used in construction to improve workability (handling and compaction), shear strength, and bearing capacity in clay soils. Lime stabilisation is the process of adding either quicklime, or hydrated lime (Table 2.1) to improve the engineering properties of clay soils. The distinction between quick lime and hydrated lime is mainly in the practical differences in applying the material to the site to be stabilised, and the chemical composition.

Quick lime is easier to transport because a smaller volume is needed, but requires water at the site (or within the soil) to hydrate it before stabilisation can occur. The exothermic hydration reaction benefits the early curing of lime-soil mixtures, notably in the drying of soil and increasing the rate of strength gain. Hydrated lime requires a greater volume to be transported, but can be mixed directly to the soil. The nature and location of the site, and transport costs, dictate the most appropriate type of lime to be used.

Since lime reacts with the clay fraction (<2µm) of the soil, soils with high clay content are the best candidates for stabilisation. As a general indication, soils with a plasticity index in the range from 10%-50% are able to be improved by the application of lime (Bell, 1989). The type of clay in the soil is significant to the effectiveness of lime on that soil, with increasing "available" silica in the clay lattice (ie the cation exchange capacity) improving the reaction. The order of decreasing clay reactivity is montmorillonite > illite > chlorite > kaolinite.

Lime provides excess Ca\textsuperscript{2+} cations which replace univalent exchangeable cations and decrease the overall charge on the clay surface. This effectively reduces the
ability for the clay to retain polar charged water molecules in the lattice and the repulsive forces between individual clay particles, enabling flocculation to occur. These chemical changes de-water and increase the effective grain-size of the clay material (and thus increase the permeability). This gives rise to much improved soil workability (amelioration) by virtue of an increase in plastic limit and friability of the material.

The mechanisms of the lime-clay reaction are (Bell, 1989):
1] flocculation
2] cation exchange
3] carbonation
4] pozzolanic reactions

Carbonation is the reaction between lime and atmospheric CO₂ as shown.

\[ CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O \]

The formation of calcium carbonate is not essential to the increase in strength of a stabilised soil, as soils cured in the absence of CO₂ achieved a strength increase. Calcium carbonate in itself is not strong enough to produce the strength increases possible in lime stabilisation (Bell, 1989). Carbonation reactions are not beneficial to lime stabilisation because it uses available lime in a weak cementation product. Lime is kept away from air as much as possible prior to its use on site.

Lime-soil reactions occur only while water is present and able to transport \( Ca^{2+} \), \( OH^- \) and other ionic species in solution (a high pH solution, between 11.5 and 12.4 pH units). Reaction will slow or stop if a soil is too dry initially, or as a result of the drying caused by lime-clay interaction (Bell, 1989). This sometimes necessitates the addition of extra water with lime in the stabilisation process, especially with quicklime which needs water in the
### Table 2.1: Types of Lime For Stabilisation

<table>
<thead>
<tr>
<th>COMMON NAMES</th>
<th>CHEMICAL NAMES</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>Calcium Carbonate</td>
<td>Found throughout New Zealand. Plays a part in stabilisation. It is important as starting point in the manufacture of other materials.</td>
</tr>
<tr>
<td>Ground Lime</td>
<td>CaCO3</td>
<td></td>
</tr>
<tr>
<td>Carbonate of Lime</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Agricultural Lime</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Quick Lime</td>
<td>Calcium Oxide</td>
<td>Formed by burning or roasting limestone at 3 locations in New Zealand - Otorohanga, Havelock North and Oamaru. The burning process drives off carbon dioxide gas (CaCO₃ = CaO + CO₂). It is called quick lime because of its vigorous reaction with water.</td>
</tr>
<tr>
<td></td>
<td>CaO</td>
<td></td>
</tr>
<tr>
<td>Hydrated Lime</td>
<td>Calcium Hydroxide</td>
<td>Formed by adding water to burnt lime. This is a vigorous reaction, giving off sufficient heat to boil water, hence clouds of steam as water is applied to the quick lime on the road surface. This material does the stabilising. Hydrated lime can be spread and mixed without further processing. The addition of water can help moisture content and workability but has nothing to do with a chemical reaction in this instance.</td>
</tr>
<tr>
<td>Dressed Lime</td>
<td>Ca(OH)₂</td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:**
- It is not burnt lime but hydrated lime which does the stabilising. Burnt lime must be "hydrated" (or slaked) before mixing.

**Figure 2.1:** Lime Columns used to reduce differential settlements.
highly exothermic hydration process.

Pozzolanic reactions involve the formation of strong cementious reaction products which produce long term strength increases. These reactions are now generally accepted as being the main process of long term soil strength improvement in lime stabilised soils. The compounds were first identified by Eades and Grim (1960), and Fergusson (1982) lists them in three broad groups:

- Hydrated calcium silicates (C-S-H)
- Hydrated calcium aluminates (C-A-H)
- Hydrated calcium alumino-silicates (C-A-S-H)

The bracketed notations are commonly used to refer to these reaction products to refer to the base compound of each material, according to the following key:

\[ C = \text{CaO}, \ S = \text{SiO}_2, \ A = \text{Al}_2\text{O}_3, \ \text{and} \ H = \text{H}_2\text{O}. \]

The exact mechanisms and chemistry of the lime-clay reactions are subject to continuing speculation and debate. Fergusson (1982) and Bell (1989) provide well presented, detailed discussions on the various mechanisms and concepts essential to lime stabilisation.

2.3 Lime Column Stabilisation

2.3.1 Introduction

The lime column technique is based on the pile concept where a load is carried via a column of stabilised soil to a level with adequate bearing capacity. Lime columns were developed in Scandinavia where large areas of thick glacial clays commonly cause bearing capacity problems in foundation engineering.
### Table 2.1: Types of Lime For Stabilisation

<table>
<thead>
<tr>
<th>COMMON NAMES</th>
<th>CHEMICAL NAMES</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone Ground Lime Carbonate of Lime Agricultural Lime</td>
<td>Calcium Carbonate ( \text{CaCO}_3 )</td>
<td>Found throughout New Zealand. Plays a part in stabilisation. It is important as starting point in the manufacture of other materials.</td>
</tr>
<tr>
<td>Burnt Lime Quick Lime</td>
<td>Calcium Oxide ( \text{CaO} )</td>
<td>Formed by burning or roasting limestone at 3 locations in New Zealand - Otohanga, Havelock North and Oamaru. The burning process drives off carbon dioxide gas (( \text{CaCO}_3 = \text{CaO} + \text{CO}_2 )). It is called quick lime because of its rigorous reaction with water. <strong>NOTE:</strong> It is not burnt lime but hydrated lime which does the stabilising. Burnt lime must be “hydrated” (or slaked) before mixing.</td>
</tr>
<tr>
<td>Hydrated Lime Slaked Lime</td>
<td>Calcium Hydroxide ( \text{Ca(OH)}_2 )</td>
<td>Formed by adding water to burnt lime. This is a vigorous reaction, giving off sufficient heat to boil water, hence clouds of steam as water is applied to the quick lime on the road surface. This material does the stabilising. Hydrated lime can be spread and mixed without further processing. The addition of water can help moisture content and workability but has nothing to do with a chemical reaction in this instance.</td>
</tr>
</tbody>
</table>

---

**Figure 2.1:** Lime Columns used to reduce differential settlements.
The applications of the lime column method in soft clay soils are (Broms, 1984):

a] the reduction of total and differential settlements for light structures (Figure 2.1).
b] the increase in settlement rate and reduction in settlement for heavy structures.
c] the reduction of settlement and increased bearing capacity for industrial floors (Figure 2.2).
d] the improvement in slope and excavation stability (Figure 2.3 and 2.4).
e] the decrease in differential settlements around structures supported on piles, and the reduction of negative skin friction on structural piles (e.g. by change in groundwater level) (Figure 2.5).
f] the reduction of traffic or construction vibration.

Mr Kjeld Faus, construction engineer and company director of the Swedish company BPA developed the method in the late 1960's (Broms, 1984). The first column was manufactured in 1971, with field trials in 1972 at a Swedish Geotechnical Institute (SGI) test site, and the first practical application of the technique took place in 1974. This was the stabilisation of a road embankment and deep trench near Stockholm in Sweden. Subsequently the research and application of the technique has advanced and been relatively well documented in commercial and scientific publications (Broms, 1984). Most information is based on laboratory and field experience in Scandinavian ground conditions. By 1983 more than 1000 km of lime columns had been manufactured, and been cost competitive with other ground improvement methods (Broms, 1984).

The publication:
Figure 2.2: Lime columns used to increase the bearing capacity and shear strength of a foundation. (after Broms, 1984)

Figure 2.3: Possible lime column configuration for slope stabilisation. (after Broms, 1984)
Figure 2.4: Lime columns used to stabilise a trench excavation in soft ground conditions. (after Broms, 1984)

Figure 2.5: Foundation design using lime columns for a single level structure. (after Broms, 1984)
source of most detailed technical material regarding Lime Columns in this thesis. Diagrams and technical information herein are primarily sourced from that publication, and acknowledgement is made as such.

The method has potential applications in a wide range of construction and remedial engineering situations requiring ground improvement. This study has an emphasis on application to roading corridor construction and stability, but the method has been used extensively in foundation engineering and ground support fields in Scandinavia. Variations on the lime column method are known from mainly Asian countries. The "Deep Mixing Method" (DMM) in Japan is similar in technique but has been used in the development of harbours and other marine installations (Saitoh et al. 1985). Lime column methods are used in India and Taiwan but these are generally not in-situ mixing methods, with lime and compacted soils (not physically mixed with the lime) being placed in predrilled holes. Rather than lime stabilising soils, the lime reacts with water and increases volumetrically, consolidating the soils in the immediate vicinity of the hole (Ruenkrairergsa et al. 1987).

2.3.2 Lime Column Construction

a) Equipment

In Scandinavia, special equipment has been developed by contracting firms (see Figure 2.6 and 2.8) to install lime columns quickly and efficiently. Such dedicated equipment does not exist in New Zealand. The main equipment requirement is a drilling mast, a rotary table and a kelly, with a lime handling system (Figure 2.8). Preliminary discussions with a local drilling contractor in the Hawkes Bay area indicate that existing tractor mounted rotary rigs could be adapted to the purpose without major problems. Some suitable lime transporting equipment would also be required to be linked to such a rig.
Figure 2.6: The Linden-Alimak LPS 4 lime column machine. (after Broms, 1984)

Figure 2.7: The Volvo BM LM 641 lime column machine. (after Broms, 1984)
b) Method

A 0.5 m diameter rotating mixing head or 'kelly' (see Figure 2.8) is screwed into the ground to the maximum depth to be stabilised. The head has pitched sub-horizontal blades (pitched to 10 cm per revolution) which initially break up the soil as the head is screwed into the ground, and then mix and rec ompact the lime-soil mix as the rotation is reversed and the head withdrawn from the ground at a constant 2.5 cm per revolution. Pulverised quicklime (CaO) is blown into the soil with compressed air via a pressurised delivery system, with an outlet in the mixing head above the blades, as the head is withdrawn.

The maximum grain size for the quicklime should be less than 0.2 mm. Scandinavian column installation machines are theoretically rated at 1 m/minute. This rate is reduced if predrilling or coring is required. The design of the lime columns is determined by the results of laboratory UCS testing, and in-situ strength testing using a purpose built lime column penetrometer (Figure 2.9), Menard pressuremeters, and screw-plate pull out testing (Figure 2.10). Laboratory methods are presented and discussed in Chapter Four.

Holm et al. (1981) report that conventional dutch cone penetrometer soundings tended to follow the relatively weak centre of the column left by the installation machine, and tended to underestimate the strength development. The bladed lime column penetrometer is pushed at a constant 20 mm/second, and the shear strength taken as 10% of the measured penetration resistance. Using the Menard pressuremeter in a predrilled hole in the lime column, the shear strength is taken as 0.18 times the measured limit pressure. The screw plate test utilises a 150-160 mm helical auger screwed into the column, and then pulled out at a constant rate.
Figure 2.8: Lime column installation equipment, and mixing head. Note the pitched blades. (after Broms, 1984)

Figure 2.9: The lime column penetrometer. (after Broms, 1984)
Figure 2.10: The screw-plate load device used for in-situ testing of lime columns. (after Broms, 1984)

Figure 2.11: The assumed stress-strain relationship in lime stabilised soil. (after Broms, 1984)
c) Stabilisation Mechanisms

Quicklime reacts with the water in the soil and a strongly exothermic hydration reaction occurs. Broms (1984) states that "Enough heat is released during the slaking of the lime so that the ground temperature increases to 30°C to 50°C at a spacing of the columns of 2 m or less." Strength development and bearing capacity of the lime column is influenced by soil temperature, increased soil pH, and curing time, with greater strength development after longer time periods, and at higher pH and temperature.

Kujala et. al.(1985) report substantial temperature increases (240°C maximum) in the immediate vicinity of the column after installation. Ground temperatures returned to the normal after 12 days. They also found that an increase in temperature from 7°C to 20°C improved strength gain 4 times, and the modulus of elasticity about 5 times.

In discussing their results, they surmise that laboratory mixing in open bowls may dissipate heat more rapidly than in-situ mixing, which may account for some of the observed differences between laboratory and in-situ shear strengths. In-situ testing gives consistently higher results compared to laboratory testing. This is thought to be related in part to the effects of confining pressure in-situ (Broms and Boman, 1979) which are difficult to reproduce in a laboratory situation.

Research by the Swedish Geotechnical Institute, presented in Holm et.al.(1983), shows that the performance of the lime column stabilisation method can be improved by the addition of between 25% to 50% gypsum (CaSO₄) with the quicklime. The conclusions Holm et. al. present based on laboratory and field investigations in Swedish conditions are summarised below.

1] The shear strength in a lime stabilised clay
increases much faster than the shear strength in a lime stabilised clay, and is 2 to 4 times higher from 10 to 100 days after mixing.

2] The shear strength in a lime-gypsum stabilised clay with a 50:50 lime:gypsum ratio reaches maximum strength after about 3 months, and then has a slower increase over a long period (years).

3] The shear strength in a 75:25 lime-gypsum stabilised clay increases more slowly than when the ratio is 50:50. From 1 to 3 months after mixing, the 75:25 shear strength is about 50% lower. However, one year after mixing the 75:25 lime:gypsum ratio gives about 50% greater shear strength.

4] Using a lime-gypsum mixture clays with a natural water content of up to 140% can be stabilised.

5] The increased rate and magnitude of strength gain associated with lime-gypsum clay stabilisation is beneficial in reducing time between column installation and construction. The 50:50 lime:gypsum ratio gives the best short term strength gain, but the 75:25 ratio gives a greater long term strength gain.

The mechanism involved in lime-gypsum stabilisation is discussed by Kujala and Nieminen (1983). The shear strength development is related to the formation of crystalline ettringite in the stabilised soil. Ettringite is only stable in high pH conditions, so the lime added must be sufficient to maintain the pH conditions in the soil, ie the 75:25 ratio for longer term installations. Sulphate ions by be freed by incomplete reaction of the quicklime, gypsum and water, and may be lost from the column in-situ. This may be significant if there are concrete structures close to the stabilised column (Holm et.al.(1983). A lime-gypsum ratio of 75:25 would be safer in that situation. The angle of internal friction for soils stabilised with lime-gypsum is elevated about 10° beyond that with lime only.(Kujala and Nieminen, 1983).
d) Application of the Lime Column Method

The lime column method is commonly applied in construction and remedial work to strengthen clay foundations for light (maximum two levels) buildings, stabilising excavations in clay soils, to enhance the rate and decrease the magnitude of settlement in a foundation area by increasing the soil permeability in the stabilised columns, and in slope stabilisation. Holm et al (1983) state that lime as a stabilising agent is effective in soft clays with a natural water content in the range 40% to 100%. Improvement of the clay properties decreases with increasing water content, but the upper workable water content range can be extended to 140% using gypsum as an additive.

Benefits arise from relatively low cost, simple construction, and the improvement of shear strength and bearing capacity with time following construction. The magnitude of settlement can be reduced if the loading is spread so that individual column loads do not exceed the column creep limit (Figure 2.11) determined from UCS laboratory testing and in situ trials (Holm et al. 1981). The number and distribution of columns is dependent on the creep limit of the column, the function they are required to perform, and allowable settlement. If differential settlement is critical to a particular structure, then the number of columns needed will be greater (Holm et al. 1981). Trench stabilisation may require overlapping columns to support the sides of the excavation along the entire length (Figure 2.4). The general design method is to limit column loads to 80% of the average column creep limit. Ideally the creep limit of 95% of all columns should exceed the design load (Broms, 1984). All aspects of lime column design, construction and evaluation are presented in great detail in "Stabilisation of Soil with Lime Columns", (Design Handbook) by Broms, B.B. (1984).
Figure 2.12: Lime column configuration for an embankment indicating the potential drainage of the foundation area. (after Broms, 1984)

Figure 2.13: The effects of preloading a site with lime columns. (after Broms, 1984)
The method of construction does not generate the excess pore pressures or soil displacements associated with conventional pile driving. This is particularly important in areas where pore pressures are naturally high. In addition, the stabilised soil within the columns is more permeable than the surrounding natural soil, enabling the columns to function as vertical drains as reported in Broms and Boman (1977a). In that paper the reported increase in permeability was between 100 and 1000 times greater than the original soil (depending on the clay soil). This is the main mechanism by which settlement rates are increased with the installation of lime columns (Figure 2.11). With 0.5 m diameter columns at 1.5 m centres, the surface area in contact with the natural soil is greatly increased for the depth of the column. Preloading an area relatively soon after construction of lime columns will load the unstabilised soil around the columns, initially causing higher pore pressures which are then dissipated by the columns (Figure 2.13). With time the columns strengthen, take up more of the loading and compress axially. The load carried by the unstabilised soil decreases in proportion to the increase in column strength (Broms, 1984).

The strength of the lime column method is derived from the presence of calcium ions in a high pH environment with clays. In areas where water flow persists there is potential for leaching to occur after the quicklime has slaked completely, removing the source of lime for cementation. Leaching or acidic ground conditions can limit the long term strength of a lime column installation by degrading the high pH conditions within columns. The presence of humus or organic material within the stabilisation area is also detrimental to lime column strength development (Broms, 1984).
Figure 2.14: Site conditions for the multi-lane highway embankment. (after Broms, 1984)

Figure 2.15: The Creep Limit curve derived from laboratory and field testing at the embankment site. (after Broms, 1984)
**e) Example of the Application of the Lime Column Method**

Highway Embankment

Broms (1984) describes a proposed multi-lane highway with a 50 m wide corridor, which was required to traverse a 120 m zone of very soft clay from 7 m to 18 m thick (Figure 2.14). Soil parameters of the clay were:

- Water content: 60%
- Undrained Shear Strength ($C_u$): 10 kPa (0.5 m)
- Unit weight of soil: 16.5 kN/m³
- Compression Index ($C_o$): 1.05
- Coefficient of Consolidation: $10^{-5}$ m²/sec

The embankment height was to be 1 m above existing ground level with a unit weight of 20 kN/m³, and constructed in two stages with a 5 year period between. A two lane road would be built in stage 1 and embankment material placed over the entire area. A 1 m drop in groundwater level was expected after 5 years when the remaining lanes were to be completed. To accommodate the embankment and traffic loading with a factor of safety of 1.5, an improvement of 16 kPa (60%) was required above the measured clay shear strength. A 2 week delay between installation of the columns and fill placement was needed to ensure the columns would safely bear the fill weight.

Column design was based on creep limit values determined from screw plate tests performed on six trial lime columns at the site, and laboratory testing. The creep limit was estimated to be 30 kN after a 6 month period (Figure 2.15). The column spacing was then calculated as 1.22 m at a 20 kPa loading (Figure 2.16). The second stage was designed assuming a 40 kN creep limit, which gave a 1.41 m spacing without traffic loading. The 5 year estimate for the creep limit of the columns was 60 kN which was able to accommodate
Figure 2.16: Load distribution between lime columns and the surrounding unstabilised soil. (after Broms, 1984)

Figure 2.17: Longitudinal section of the embankment site showing calculated settlement and lime column depth. (after Broms, 1984)
increasing load from the predicted drop in waterlevel. Column length was varied according to the predicted settlement in the variable thickness of clay to minimise differential settlement on the road (Figure 2.17). For a similar clay thickness, the column lengths along the edge of the embankment were about 1 m shorter than those at the centre. Estimated length of lime columns required was 30 000 m.

2.4 Lime Slurry Pressure Injection Stabilisation (LSPI)

2.4.1 Introduction

The concept of injecting lime slurry developed from the need to treat low strength subgrade materials at depths greater than conventional construction methods allowed. The use of lime as a surface stabilising agent is very common, and the extension of its use to greater depths began in the early 1960’s in Louisiana as a remedial technique to establish a moisture barrier beneath existing foundations (Wright, 1973). This involved individual injections around the perimeter of the foundation to be stabilised. The method has been developed mostly in the United States as a method of embankment and general foundation improvement to limit settlement, increase shear strength and bearing capacity.

Initial trials on a small scale were successful, reducing the down time on trouble spots on railway embankments and giving adequate foundation performance. Tanker trucks with mounted hydraulic probes and the ability to travel on rails were built by contractors and proved successful in increasing the speed of application. Tractor and crawler mounted injection rigs with a maximum 3 m injection depth were used for more conventional and off-road sites (Figure 2.18). The maximum depth of application for LSPI reported is 12.2 m on railway embankments, with most applications to only 3 m (greater than 90%, Boynton and Blacklock 1985).
Figure 2.18: "Typical" Lime Slurry Pressure Injection (LSPI) application using a tractor unit (which would be connected to a slurry tank). (after Boynton and Blacklock, 1985)

Figure 2.19: Seam stabilisation effects of LSPI. (after Boynton and Blacklock, 1985)
The method is now commonly promoted as a proprietary method by contracting firms, especially the Woodbine Corporation in Houston and Fort Worth, Texas. Initial development has been promoted by several Federal and State roading organisations in the southern United States and the Woodbine Corporation, with some research by the University of Arkansas, Little Rock, Arkansas, and the University of Calgary, Canada. Published material (e.g., Blacklock and Wright, 1986) on test sites has been written up by individuals generally pro-LSPI, and is not as objective or detailed as it could be. LSPI has been experimented with in other countries, but not on a commercial scale. The method has been used on a large scale for the Dallas/Fort Worth Regional Airport (460 000 m² at 1.5 m centres to a depth of 3 m), and several commercial sites on old landfills and areas requiring subgrade improvement for construction in the southern USA.

2.4.2 Method and Mechanisms

The method involves the mixing of lime (quick lime or hydrated lime), fly ash (FA) as required, wetting agents, and sufficient water to give a 30% solids slurry with a pH of 11.9-12.4 (Blacklock and Wright, 1985), and then pumping it under pressure into the soil to be stabilised. The slurry is pumped until either 1) the pressure (350 - 1400 kPa) is sufficient to return slurry around the probe or through other injection holes or ground fissures (refusal), or 2) a 5 minute period has elapsed (Cuthren, 1984), or 3) a specified amount of slurry has been pumped, whereupon the probes are moved upward 0.3 m to 0.45 m to the next injection level (upstage injection). Injection is performed by hydraulically pushed hollow hardened steel injection probes (38 mm to 41 mm outside diameter) with nozzles designed to distribute the slurry 360° into the adjacent ground (Figure 2.18).
Figure 2.20: Percent Lime/Lime Modification Optimum (LMO) verses Unconfined Compression Strength (UCS) for quicklime and hydrated lime. (after Petry and Lee, 1989)

Figure 2.21: Change in Optimum Moisture Content of a Texas Soil with different lime type. (after Petry and Lee, 1989)
Initially the probe is inserted at 0.9 m to 1.5 m centres over the area to be stabilised, though a second or even third injection pattern offset from the first is sometimes performed if required by the site conditions (Figure 2.18). Multiple injections have been necessary in several case histories in the USA to achieve an adequate underground distribution of slurry depending on the soil conditions at the site. Once the site has been injected, any excess slurry spilt on the ground surface should be mixed into surface soils and compacted. Establishment of good surface drainage in the area injected is considered to be an important part of the LSPI operation.

Estimates from contractors infer that rarely is the total amount (by weight) of lime injected greater than 1% to 1.5% of the soil (Wright, 1978). The result is a displacement of excess water from the soil mass surrounding the injection point and the injection of a network of thin seams and sheets of lime slurry which tend to "encapsulate" (Welch, 1975) blocks of soil. This isolates the encapsulated soil from further changes in water content, and thus the volume changes associated with seasonal variation in water content.

The clay soil immediately adjacent to the network of injected lime seams undergoes rapid modification by the diffusion/migration (Davidson et al., 1965) of a Ca²⁺ rich solution, causing cation exchange, agglomeration and flocculation (Joshi et al., 1981). The extent of this diffusion/migration modification away from the lime seam is small (1 cm-2 cm), with a very slow increase over time (Figure 2.19). Fohs and Kinter (1972) report migration of 3.8 cm-5 cm in 180 days. Volumetrically only a relatively small amount of the soil injected is modified by direct contact with the lime slurry. Welch (1975) describes the changes to the soil mass following injection as increased volumetric stability related to the limiting of seasonal changes in water content, and increased shear strength in the long term.
as pozzolanic cementious reactions progress.

The true nature of the LSPI stabilisation mechanism is still to be verified, but much of the published literature now refers to rapid physicochemical reactions (cation exchange, agglomeration and flocculation) providing immediate improvement by direct lime modification, with subsequent improvement mainly attributable to pozzolanic (time and temperature dependant) reactions, and to diffuse reaction (Stocker, 1972). Stocker (1972) proposed that soil could be affected by the diffusion of Ca²⁺ ions in pore water beyond areas in intimate contact with lime. Joshi and Wright (1978) describe the 2-3% increase in average water content of a soil mass following LSPI. This increase is noted not just in the vicinity of the injected slurry seams, but away from them as well. This may be related to the positive displacement of the soil’s original excess pore water. They surmise that this could allow subsequent cation exchange to occur from the Ca(OH)₂ saturated slurry influencing a larger volume of soil. Some doubt exists regarding the permeability of clay soils being adequate to allow this to occur, and it has yet to be proven in field testing.

Fly ash is a pozzolanic material defined in ASTM C219-82a as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementious value, but which will, in finely divided form and in the presence of moisture, economically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementious properties". It is a by-product of the combustion of coal in thermal power stations, and is collected by mechanical or electrostatic precipitation from boiler flue gases. Boynton and Blacklock (1985) indicate that the quality of fly ash is extremely variable, depending on the source of the coal and the impurities (e.g. unburnt coal) present. The fly-ash evaluation and classification methods are apparently common in the U.S.A. where coal-fired thermal power stations are numerous and fly-ash is freely available at low cost.
Assessment methods are not documented in New Zealand although ASTM standards could be adopted if required. Sources of fly-ash in New Zealand are limited, as natural gas is burnt in preference to coal at present in the Huntly Thermal Power Station. Transportation cost would also be significant for areas like East Cape. The durability of fly ash in the presence of water is significant, as well as its ability to function as a pozzolan.

2.4.3 Comparison of Hydrated Lime and Quicklime Slurries

Petry and Lee (1989) investigated the effects of quicklime and hydrated lime slurries in subgrade stabilisation practice in North Central Texas. The expected soil-lime reactions were cation exchange, ion crowding, dissolution of clay, flocculation and agglomeration, carbonation and pozzolanic reactions. These mechanisms would normally produce soils to have reduced plasticity indexes and shrink swell potentials, increased shear strength and reduced compressibility, increased workability and water repellency, reduced compactibility, and increased abrasion and erosion resistance.

The pH of lime slurries is less at higher temperatures because the lime solubility decreases, and free Ca\(^{2+}\) can exist only within the 11.9 to 12.4 pH range (Petry and Lee, 1988). Quicklime slurries develop heat on mixing with water (hydration), and field mixed slurries have finer particle graduation (95% of lime particles <5 microns in some cases) than similar hydrated lime slurries because the slaking process increases at higher temperatures. This increases the reactivity of the lime by virtue of the increased surface area available. Sedimentation rates are less in quicklime slurries as well.

Testing involved slurries with 31% solids (40% is the maximum considered pumpable). Slurry units commonly used are % solids, specific gravity, and weight of lime per unit
volume of water. Results showed that about 1.5 times more hydrated lime than quicklime was required to achieve the same specific gravity in slurries with almost identical percent solids. Supernatants from these slurries analysed by atomic absorption spectrophotometer indicated an 8.4% larger calcium concentration in the quicklime supernatant solution. Eades and Grim Lime Modification Optimum (LMO) testing on six Texas soils gave an average of 1% (by dry weight of stabiliser) less quicklime than hydrated lime required to achieve a sustained pH value beyond the LMO value.

The nature of stabilised soils with the addition of the different types of lime was also investigated by Petry and Lee (1989). They found that unconfined compression strength (UCS) values after 28 days were greatest with quicklime, and for 3% lime from the LMO to twice the LMO, after which the UCS values began to decline (Figure 2.20). The maximum UCS values for both types of lime occur at twice the LMO, and this was identified as the Lime Stabilisation Optimum (LSO). Water content and dry weight after curing decreased with increasing lime content. Mean failure strain decreased from 7.6% in natural soil to 1.6% for stabilised soil. Petry and Lee observed an 26-fold mean elevation of pore water calcium concentration in stabilised soils at the LMO for both lime types. Ca\(^{2+}\) concentration in exchange complexes within the stabilised soils at LMO went from 12.47 meq/100g to 189.9 meq/100g with hydrated lime, and 215.8 meq/100g with quicklime. Peak values of Ca\(^{2+}\) concentration in exchange complexes and UCS at 28 days occurred at the LSO (twice LMO), which lead them to infer that there may be a relationship between Ca\(^{2+}\) concentration in exchange complexes and UCS.

Petry and Lee’s (1989) paper documents the effects on compaction following the use of different types of lime slurry. Figure 2.21 (reproduced from Petry and Lee (1989)) shows the increase in optimum moisture content beyond the natural compaction curve for both lime types as expected, but by a greater degree for hydrated lime. Swell tests showed
significant reduction in volume change on the addition of lime, with mean water content after swelling about 7% higher with hydrated lime slurry.

Blacklock et al. (1982) indicate that the curing of lime/fly ash grout is dependent on time, temperature and moisture factors. A guide to strength gain with time and temperature is given as follows. The 60 day compressive strength for grout cured at 10°C will be obtained in 30 days at 21°C, and in 10 days at 30°C. This value is thought to be about 50% of the 365 day usable strength at 25°C. The strength gain will continue for several years under favourable conditions (Blacklock et al., 1982). These findings would require confirmation in New Zealand conditions, but could successfully speed up testing programmes.

2.4.4 Application of LSPI

The decision to use LSPI is essentially an economic one once the soils have been proven to be lime responsive and suitable equipment is available. The LSPI soil testing procedures in the literature are set out in Blacklock (1982), Boynton and Blacklock (1985), and Blacklock and Wright (1986). Limited interpretation of results and reasons for the type of injection used from sites where LSPI has been applied are presented. This may be because the method is generally being marketed and promoted in the literature rather than being analysed scientifically and objectively. The greatest lack is the technical justification for of the use of LSPI or L/FASPI (Lime/Fly Ash Slurry Pressure Injection) in preference to other methods, and information regarding selection of the most appropriate type of fly ash for a particular site. This is a consistent weakness in almost all LSPI literature, although the knowledge exists in industry. Information in general lime stabilisation reports from the 1960’s (Highway Research Board Bull.335, 1962) dealing with conventional surface stabilisation are technically superior
to most modern literature, particularly in regard to lime/fly ash mix design and testing.

The essential criterion for the application of LSPI is an improvement in the strength of the soil by the application of lime, and UCS (Unconfined Compressive Strength) testing is the most common method. Samples with different percentages of lime are tested after a series of curing periods from 1-28 days or more. This could include testing with percentages of fly ash if it was available. Soils showing marked strength gains (200%-500% ,Boynton and Blacklock (1985)) are potential candidates for LSPI.

Welch (1975) presented a more conventional test evaluation programme for roadbed stabilisation evaluation. The testing uses conventional laboratory test methods, and requires no extra specialised equipment. His programme is:

a) Determine natural moisture content and density.

b) Examine structure of soil for seams, fissures, slickensides .etc.

c) Perform grain size analysis to determine the percentages of sand, silt and clay.

d) Use Eades-Grim quick test to find optimum lime content.

e) Find the liquid limit and plastic limit for natural soil with optimum lime content.

f) Test unconfined compression specimens of natural soil and soil with optimum lime content after curing.

Welch also provides some practical guidelines by which the testing can be evaluated to assess the suitability of LSPI for a particular site. Blacklock and Wright (1986) prescribe a preliminary surface investigation, then a detailed subsurface and laboratory investigation. The testing programmes will be discussed in detail in Chapter 4. The injection project should not proceed until testing of all
site materials is complete. The testing should provide a
basis for slurry design, and the level of improvement that
can be expected from injection.

2.4.5 Examples of the Application of LSPI

1] Railway Embankment, Chicago and Northwestern
Railway (C&NW).
Location: Des Mione, Iowa, USA.
(from Boynton and Blacklock, 1985)

An embankment consisting of 8.4 m of clay fill over a 3
m clay subgrade overlying loose to medium, dense sands was
experiencing settlement because of increased traffic and axle
loadings on the track. The site had become a constant
maintenance problem which defied conventional remedial
techniques. Remedial methods attempted included 1] widening
of the fill 4 m either side of the embankment, 2] cement
grouting, 3] shallow LSPI, 4] installation of a 250 mm
subsoil drain, and 5] adding rip-rap material to the base of
the slope. Some methods slowed settlement, but ultimately it
returned and additional work was needed. The area had
received about 200 wagon loads of ballast aggregate over a
seven year period so that some parts of the track were
underlain by up to 3 m of aggregate. The increased weight of
the fill caused bulging at the toe of the embankment and
disruption to the track on top. Train speeds were greatly
reduced, and maintenance costs very high.

In 1976 C&NW decided to use a three-stage LSPI programme
on the basis of "testing indicating a good lime reaction".
The first two stages involved LSPI to 3.5 m below track level
using a three probe unit with 2-3 days curing between
injections. The second injection was halfway between the
first injection points. The third injection was made from a
single probe to a depth of 12 m at 1.5 m spacings along a 300
m track length. In excess of 300 tons of lime (type
unspecified) were used, with the 12 m injections using close
to a ton per hole. Displacement of water from the fill was noted during the first two stages of the injection.

In three weeks the train speed had returned to 70 kph over the section. Since 1976 the maintenance for that section has been minimal.

Location: Route 77, Cape Girardeau County, Missouri, USA. (from Blacklock and Wright, 1986)

The site investigated by the Missouri Highways Department had a history of failures, and unsuccessful remedial works. The site evaluation laboratory testing used both standard soil test methods and lime glaze and L/FA-seam tests. The test section had 3:1 (Horz:Vert) slopes, but slides had occurred on slopes as shallow as 6:1.

Soil Testing: Two soil types were present, 1] Sharkey Clay; a highly plastic, grey, waxy alluvial clay; 2] Memphis Soil; an imported loess material used in maintenance of the area. Both soils were found in the slide zone of failures and are both lime responsive, the Sharkey clay more so than the loess unit. The Sharkey clay was the original fill construction material, and was inferred to be the origin of most slide failures in the area.

Injection Procedure: The injection contractor decided on a L/FA slurry injection programme using 1.82kg solids to 3.8 litres of water with a 1:3 lime to fly ash (by weight) ratio. Part of the embankment was to be single point L/FASPI injected (210 m length) at 1.5 m centres, and the remaining part (30 m length) double injected [the first injection was LSPI, and the second L/FASPI]. No discussion of the decision making process was presented, but the effectiveness and economic benefits of single against double injection was of interest to the roading authority. A total of 139 tons of lime and 301 tons of fly ash were injected to a maximum depth of 3 m over an area of 2600 m2, at pressures from 350 kPa to 1400 kPa.
Performance: In the 14 months following the injection no movement was observed in the slope, while 3 control sections in the area all failed with 6 months. Monitoring was to continue for five years.

2.5 Synthesis

LSPI and Lime Column methods of deep lime stabilisation are attractive as potential construction and remedial options in the areas of the world where the technology required to implement them has been developed, i.e. the Southern States of the United States of America and Scandinavia respectively. The development of testing methods that accurately assess the degree of improvement possible with a particular technique is a very important area of on-going development with both techniques.

The Lime Column method has had a larger amount of technical information published and is impressive in the number of successful stabilisation programmes implemented. The ability the be able to design the columns in a similar way to a conventional pile foundation will make its presentation to engineering staff as a concept relatively easy.

LSPI is well presented as a concept, but is lacking in the detailed technical appraisals that the lime column method has. This stems from the promotion of LSPI by the National Lime Association in the United States of America who reflect the interests of member contractors and lime producers. The literature is missing the details of decisions made to establish the degree and type of LSPI or L/FASPI most appropriate to a particular site. Despite this omission, the technique is attractive by virtue of its simple application method and reported relatively low cost against more conventional stabilisation programmes.
Chapter 3: Site Investigation

3.1 Introduction

3.1.1 Timing and Aims

The investigation of the Panikau Hill site took place over a period of seven days, including three days drilling supervision, one day excavating and logging trenches, and numerous subsequent visits to monitor piezometer levels and survey movement.

The principal aims of the field investigation were to:

1) Map the engineering geology of the site, and determine the depth and mode of slope failure.
2) Sample soil materials for a laboratory strength testing and lime stabilisation evaluation programme.
3) Establish and monitor a survey network at the site to determine the rate of deformation.
4) Assess the suitability of the site as a candidate for either LSPI or the Lime Column stabilisation.

3.1.2 Location

Panikau Summit is located 35km north of Gisborne on State Highway 35 [Grid Ref: NZMS 270 Y17 Te Karaka: 650835] (Figure 3.1), south of the Panikau Road intersection. The site is about 4 km from the Pacific Ocean coast at Whangara. The section of the State Highway is known locally as the Whangara Hill, and is adversely effected by numerous earthflows crossing the road corridor. Panikau Summit is at the northern end of Whangara Hill and represents the highest topographic point on the hill. State Highway 35 curves round the western aspect of the ridge from the south, crosses the summit, and drops into the next valley to the north (PW Gisborne 7034/2, Appendix 6).
Figure 3.1: The Australia-Pacific Plate Boundary, East Coast, North Island.

Figure 3.3: Average Rainfall Year (1968-1991)
3.1.3 East Coast Geological Setting

The geology of the East Coast North Island is dominated by the Pacific and Australian convergent plate boundary located off shore in the Hikurangi Trough (Figure 3.1). Oblique plate convergence (50 mm/year westward in Walcott (1978b)) has created a highly deformed margin (up to 200 km wide) in on shore areas of the East Coast, North Island. The eastern part of the Raukumara Peninsula is characterised by complex deformation of Mid-Cretaceous to Early Tertiary age marine "flysch" sediments, overturned sequences and melange zones or broken formations (Moore, 1988). Deformation has occurred along low angle normal and reverse faults, with the development of Late Oligocene and Early Miocene decollement zones within the Cretaceous-Early Tertiary stratigraphic sequence (Kenny, 1984). In the Whangara area the decollement zone is associated with highly deformed "bentonitic" Lower Tertiary lithologies surrounding local Cretaceous basement highs (Ridd, 1964,1967).

3.2 Engineering Geological Field Investigation

3.2.1. Site History

a) Site Development

State Highway 35 in the Panikau Summit area was located on the Eastern side of the summit prior to a realignment in 1968-69 (Figure 3.2). The eastern alignment had been severely disrupted by a series of earthflows which were judged to be too extensive to realistically reinstate. In 1966 a scheme was developed called "PANIKAU RECONSTRUCTION: R.M. 189.7-190.4" involving a major change in the horizontal alignment of State Highway 35 to the western side of the hill, away from the earthflows (PW Gisborne 7034/1, Appendix 6). A penetrometer investigation was performed in June 1967, and the final design was a scheme called "PANIKAU HILL RECONSTRUCTION: R.M. 189.41-190.48", dated June 1968. It was
Figure 3.4: Location of Trenches and Survey Pegs in the Headscarp Area.
built in the 1968-69 construction season, and involved fill embankments up to 2.8m above existing ground level between CL 5900 and CL 6100 (Centreline units in feet), and up to 3.1 m high beneath the new Panikau Road formation (PW Gisborne 7034/1, Appendix 6).

b) Failure History

Weather Conditions

The southern side of the new Panikau Road intersection embankment failed in the winter of 1969, and continued to move in some unseasonal heavy rain in November-December of that year (Figure 3.3). The rainfall record for this period is presented in Appendix 1. The records show that the 1968-70 period was below the 1971-91 average. No extreme rainfalls were recorded, though the November total was high relative to the rest of the year.

Construction Conditions

The 1968 plans (PW Gisborne. 7034/1, Sheet 3) show a subsoil drain ending at about CL 5910 on the upslope side of the highway with no discharge point to the downslope side of the embankment. The natural drainage from this point would be beneath the southern end of the new intersection exiting close to the area which failed in 1969. A 380 mm (15") subsoil drain was detailed following the 1969 failure (PW Gisborne 7256, Appendix 6) north of the intersection from the upslope watertable crossing to the western side of the new embankment, which tends to indicate that the original construction did not have such a drain. The site of the subsoil pipe draining this upslope watertable is opposite (by visual estimation using old plans at the site) where embankment failures subsequently developed. No evidence of such an installation could be found at the site during this investigation, which means that it has either been buried by top-ups of the subsiding area, or it was never installed.
Figure 3.5: Excavator covering Trench 1, with 1972 piezometers in the foreground.

Figure 3.6: Panikau Summit looking north. Note the non-vertical fence and pole, and remedial tree planting in headscarp area.
Figure 3.7: Geological Map and Generalised Stratigraphy (after Ridd, 1967).
The 1972 penetrometer investigation location sketch maps (not included) indicate that watertable drainage flowed to about the detailed intake position of the subsoil drain. No clear construction information exists about the site other than the plans mentioned, so no clear inference can be drawn about the presence or absence of such a subsoil drain. It is safe to say that poorly controlled drainage in that location would be significant in terms of slope stability, and could easily account for the observed embankment failures.

Post Construction

A Ministry of Works and Development flood damage file dated 21 January 1970 reported a batter failure below the new Panikau Road intersection, and the need to move the Panikau Road horizontal alignment north, away from the failure. Drawings for the remedial work (PW Gisborne.7256) show a proposed retreat away from the southern batter of the Panikau Road intersection and basic surface reshaping of a 20m long by 1-2m failed block which had dropped 1-1.3m (PW Gisborne 7256, Appendix 6). Pavement detailing indicates that the pavement section between CL 5800 and CL 6100 was cement (150 mm 5% cement) stabilised. No construction record of the work exists.

In February 1971, a further failure caused by "the wet 1970 winter" was reported and urgent recommendations were made to relocate Panikau Road away from the receding headscarp in the vicinity of the intersection before the 1971 winter. The Panikau Road intersection was then relocated 300m north on SH 35 to its position today. No file reference or plans could be located for this work.

The Panikau Summit area was subject to an investigation by MOWD Soil and Water staff in the 1971-73 period, which included the deep soundings (dutch cone penetrometer) presented in Appendix 2, and discussed in Section 3.4.1. Several shallow (less than 3 m) standpipes were installed in the area immediately below the failed embankment (Figure
Figure 3.8: Photograph from Peg A2 looking north. Note filled backscarp, and vehicle parked on old Panikau intersection. The borehole is recessed in a toby box behind the vehicle.

Figure 3.9: Looking upslope and east toward SH 35. Note the low relief, abundant marsh grass, and the vehicles on the highway below the pine plantation.
3.4), and artesian water pressures were recorded. This data was unobtainable, but verified verbally by DSIR Water and Soil personnel who did the work (Don Miller, Gisborne pers.comm.).

These standpipes are still in existence (Figure 3.5), though on-going slope movement has bent them downslope and snapped one off. One of the standpipes flowed a little for the winter period, indicating high water pressures within the slope. This area is downslope from the area that may have been affected by poor drainage, and directly below where the 380 mm subsoil drain may have exited the slope (assuming it was installed). These water pressures could be sustained by uncontrolled drainage from the upslope watertable seeping through the subbase and subgrade lithologies.

Recent History

No further mention of the Panikau Summit site is made until 1975, when a general flood damage file reported that most information about the area had "gone missing" about the time Napier District Laboratory had become involved. No specific records of maintenance of the highway in the area exist for the period to 1989. Since then a more rigorous road maintenance records system (RAMMS) has been initiated, and expenditure able to be related to specific sites. The Panikau Hill area was mentioned in general flood damage reports in 1978 and 1982 (WORKS Gisborne SH35 Flood Damage Files) regarding continued subsidence problems. At least two erosion stabilisation proposals were put forward for the Whangara-Panikau area, but only a limited amount of reshaping and tree planting in 1981 was ever completed. This work was responsible for the planting of the rotated block at the southern end of the site (Figure 3.6).

A December 1986 Water and Soil report (Hall, unpublished report) identified RP 289/6.1-8.8 Whangara Hill (Panikau Summit is RP 289/6.8) as an area with erosion problems. The report stated that problem areas were related to the outcrop
of Weber Marl (poorly draining and high bentonite content) with Haumurian argillite, and the occurrence of gas seeps and springs at argillite contacts. The presence of artesian water pressures was again noted.

Following Cyclone Bola in March 1988 there was a broad assessment of flood damage on SH 35 by Works Consultancy staff, and the Whangara-Panikau area was mentioned as an area disrupted by numerous dropouts (underslips) and earthflows. Repairs were not recorded in detail, but some reshaping and resealing was required at Panikau Summit (T. Boyle, Works Gisborne Pers.comm.). Cyclone Bola produced a large amount of roading work in the Gisborne-East Coast area, and a consultant engineer (Mr Guy Evans) was employed to cope with some of the immediate problems. He produced a WORKS internal report (Evans, 1989) on many problem sites on SH 2 and SH 35 and recommended the application of Lime Slurry Pressure Injection (LSPI) to stabilise several of the sites, including Panikau Summit, as well as some estimates of costs for a trial LSPI treatment programme.

That report then lead to this Transit New Zealand funded study beginning in March 1990. Survey work began in May and continued to May 1991. The highway had arcuate depressions across a 30 m length of the north lane to the centre line when this work began (Figure 3.4). The north lane shoulder was in a poor state of repair. The area was resealed in October 1990 as part of regular maintenance, with no major repair to the existing depression.

3.2.3 Geology

The basement geology of the Whangara-Panikau area is poorly exposed, largely due to the veneer of highly deformed Lower Tertiary fine grained marine units which drape the area (Figure 3.7). An abandoned quarry 200 m west of the site, and a working quarry 1 km north on SH 35 expose the local basement lithology, which is called Whangai Argillite (Ridd,
1967) or Whangai Formation (Moore, 1988). This is a dark grey to red brown, well indurated, weathered to highly weathered, highly fractured and sheared mudstone. The unit has been structurally disturbed and is locally associated with gas seeps and springs where it crops out. Microfossil ages from oil company work and limited macrofossil evidence (Ridd, 1967) indicate a Haumurian age (Late Cretaceous).

The poorly exposed early Tertiary and younger lithologies together constitute the covering veneer of deformed silts, clays and cemented silty sandstone overlying the Whangai Formation. Typically hillsides have a 1 m-8 m thickness of moist to saturated, very soft to firm, plastic silts and clays with a variable thin soil cover. The soils have little material integrity and are subject to endemic landslides and earthflows.

Lithologies include hard, light bluish green, slightly glauconitic silty fine sandstone in angular boulders and cobbles; very soft to firm, moist to saturated, medium to dark bluish green and light brown, sheared silts and clays; areas of dark reddish brown and green silty clay. The highly deformed nature of these materials makes stratigraphic distinction meaningless within the study area. The latest geological interpretation includes these lithologies in Undifferentiated Miocene (Moore et al., 1989) with some Weber Formation. Weber Formation is notorious as a 'bentonitic' unit throughout the East Coast. Clays within the study area are more correctly designated illitic-montmorillonites (Chapter 4, Section 4.1.2). Atterberg Limit comparison with commercial bentonite and a bentonite from Tarndale in the Mangatu State Forest are shown in Table 4.1.

Organic soil horizons are thin (<0.3m) and commonly buried by failures from the slopes above. The clay rich soils are dense and often saturated. Areas of road fill material are also present in road embankments (Figure 3.8).
Figure 3.10: Photo and line drawing of the left lateral scarp of an earthflow below the highway embankment.
Figure 3.11: Hillside southeast of Peg C2. Note the hummocky ground, and deformed fenceline and power pole.

Figure 3.12: Looking south from the fenceline above Peg C2. Note the standing water and marsh grass. Whangara coast of Pacific Ocean in background.
3.2.4 Geomorphology

The Panikau Summit site faces east to east-southeast, from the ridge at 150m (above sea level) to the valley floor at about 60m elevation (Figure 3.2). The Panikau area is typified by subdued hummocky relief (Figure 3.9) on hillsides prone to creeping slope movement (10°-20°). The failed area is in the upper 50m (vertical) of the slope in largely pastoral grassland with scattered trees and low scrub. Exotic pine trees cover the ridge top above the highway which crosses the slope at about 140m. The area below the fence line is grazed by beef cattle which cause severe disruption of the ground surface (Figure 3.6) in very wet and locally steep grassed areas.

State Highway 35 at Panikau Summit is a fill/sidling fill construction keyed into the natural ground surface in the upslope watertable and onto a fill embankment downslope. The fill embankment slope is 45°, and 40° to vertical in areas where slope failures have produced backscars. End dumping of material (without compaction) from local watertable cleaning appears to be common at the site (Figure 3.8).

Total headscarp length is 95 m (Figure 1.3 and 3.2), in an arrowhead geometry which follows the line of the 1968 Panikau Road-SH 35 intersection and the present highway to the south. A back-tilted block (rotated 15° upslope) fills 20m of the southern end of the scarp parallel to the highway (Figure 3.4). The ground below the backscarsps surface is uneven, sloping at 12° for 25m in a series of scarplets and larger grassed blocks. The backscarp area along the old Panikau Road intersection has more relief than the area below SH 35, as if the failed blocks have remained (more or less) where they originally failed.

The left lateral scarp of an earthflow trends at 025° at heights of up to 1.2m for 6m upslope of the fenceline at the
edge of the road reserve (Figure 3.4 and 3.10), and marks the start of the transport zone of the earthflow. The area south of this feature (Figure 3.11) shows evidence of soil creep and slope deformation. The ground surface is uneven, often wet and swampy (Figure 3.12) having standing surface water for long periods except in summer. No well defined right lateral margin occurs at the level of the fenceline, though a small poorly defined scarp 20m south along the fence marks a slight change in ground relief (Foreground of Figure 3.12). Aerial photograph interpretation indicates that a second earthflow is immediately adjacent to the first earthflow and flows down slope to coalesce with the first (Figure 3.2). The southern limit of movement is the small ridge on which a telephone pole is located (Figure 3.4 and 3.11). Most of the ground surface north of the ridge shows evidence of creep, poor drainage, and scarp development. Several "islands" on the slope appear to be stationary relative to well defined earthflow features which flow around them (Figure 1.3 and 3.2).

3.2.5 Rainfall

The 1968 to 1991 rainfall records for the nearest New Zealand Meteorological Service rain station, Mokairau/Pakarae (D88421) located 3.2 km east-northeast of Panikau Summit, are tabulated (NB: data is in 0.1 mm units) in Appendix 1. The location of the rain station was changed to Pakarae Station in October 1969, which is close to the time of the initial Panikau Road intersection failure. There is a marked change in the data between the two locations, but comparison with other rainfall stations in the Whangara area indicates that the 1969 winter had below average monthly rainfall (Figure 3.3). The November 1969 rainfall was slightly above average, and may have been significant to slope stability after a dry period. Seasonal clay soil shrinkage may have been greater than normal (Figure 3.13) prior to the November rainfall allowing greater moisture access within the slope.
Figure 3.13: Shrinkage cracks common in summer conditions adjacent to Peg B1.

Figure 3.14: Variation in monthly rainfall totals at NZ Met. Service Station at Mokairau, Whangara.
The rainfall record shows a large variation in monthly rainfall totals for the station, as indicated in Figure 3.14, which shows the High-Low and average monthly totals for the period. This is typical of the East Coast area. The average recorded annual rainfall for this station in the 1968-1991 period is 1276.1 mm.

3.3 Surveying

A network of 27 peg stations and traverse pegs were installed at Panikau (Figure 1.3) in June 1990, and last surveyed in May 1991. Details of the survey setup and methodology are presented in Appendix 4. Surface movement displacement from the original peg location from the 9 month period are presented in Figure 3.15. Recorded surface movement between surveys was usually greater than the ±2 cm maximum error associated with the survey. The change to using a WORKS professional survey crew after the initial setup survey has cast doubt on the validity of some apparently large station movements (See Appendix 4) recorded between the author's initial survey and the first WORKS survey. The initial movement at stations C1, D4, and E4 (Magnitude and direction data in Appendix 4) is large compared to subsequent movement even allowing for winter rainfall in that period, and it may be in part a surveying artifact. Continued surveying over a longer period is needed confirm the true annual and seasonal trends in peg movement. Station movement close to the highway was minimal, with most movement detected in stations more than 30 m from the highway. Movement direction (beyond the survey error) is to the southwest and generally downslope. The maximum rate of movement (1cm/month) is very slow, and most peg movement rates are very slow according to the classification in Varnes (1978).

Some problems with the surface clay soils shrinking away (Figure 3.13) and loosening the pegs have been noted as surveys have been performed, and indicate that longer pegs (eg. steel waratah stakes) would be more stable. The survey
Figure 3.15: Total peg displacement relative the original survey location for each peg. The lower part is an enlarged version of the top part.
data indicates that in some cases the movement observed was probably the peg moving within a loosened hole.

3.4 Subsurface Investigation

The subsurface investigation consisted of summarising previous penetrometer information, the excavation of three trenches close to the site of the embankment failure (Figure 3.4), some Scala penetrometer soundings, and the drilling of a borehole for core recovery and piezometer installation.

3.4.1 Penetrometer Soundings

Three series of Dutch Cone penetrometer soundings were performed at the Panikau Summit site in previous investigations, the first prior to the realignment, and the rest in relation to post construction failure.

a1 1967 Pre-Realignment Dutch Cone Penetrometer Investigation

The soundings performed prior to construction by Napier Works Laboratory were not able to be located in the laboratory or general files, despite the fact that office records confirm the work was completed. These would have been very useful in interpreting subsequent soundings.

b1 1972 Dutch Cone Penetrometer Investigation

In June 1972 a series of five penetrometer soundings were performed in a long section perpendicular to the slope following the SH35-Panikau Road intersection failure. Sounding depth varies from 3 m to 25 m. The locations and elevations of the soundings are shown in PW Gisborne 7502 (Appendix 6), designated as Deep Sounding 1 to 5 (DS1-DS5). The point resistance only was recorded. Detail and interpretation of the plots is presented in the Appendix 2, and briefly summarised in Table 3.1.
Table 3.1: 1972 Penetrometer Investigation Summary

<table>
<thead>
<tr>
<th>Penetrometer Number</th>
<th>Depth to Inferred Low Strength Horizons (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS1</td>
<td>5m, 10.8-12m, 16.5-17.6m</td>
</tr>
<tr>
<td>DS2</td>
<td>1.4m, 5-5.2m, 10m</td>
</tr>
<tr>
<td>DS3</td>
<td>1m, 3m</td>
</tr>
<tr>
<td>DS4</td>
<td>4.2m, 6.8m</td>
</tr>
<tr>
<td>DS5</td>
<td>2m, 2.8m, 4m, 5.2m</td>
</tr>
</tbody>
</table>

[1] Post-Cyclone Bola Dutch Cone Penetrometer Investigation

Following Cyclone Bola in March 1988 a series of five 20-tonne truck mounted Dutch cone penetrometer soundings were performed by Gisborne WORKS Consultancy Engineering Laboratory in the immediate vicinity of the SH35 road surface to investigate the ongoing deformation occurring at the site. Point resistance and sleeve friction were recorded in MPa and Kpa respectively. The N and S designation refer to a northern section and a southern section within the site (Figure 3.4). No location map was made other than the positions of the soundings relative to the centreline at the time. Subsequent resealing and remarking do not enable exact location of this series other than PS#1, which had a surviving marker peg at the time of the site survey (Pene 1 on Figure 3.4). Inferred low strength horizons are associated with coincident low values of point resistance and sleeve friction in the penetrometer plots presented in Appendix 2. A brief summary of the results is presented in Table 3.2
Table 3.2: 1988 Penetrometer Investigation Summary

<table>
<thead>
<tr>
<th>Penetrometer Number</th>
<th>Depth to Inferred Low Strength Horizons (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PN#1</td>
<td>3m, 3.3m, 5-5.5m, 8.5m, 9.8m, 11.6m</td>
</tr>
<tr>
<td>PN#2</td>
<td>2m, 4.5m, 5.8m, 7.4m, 9.2m</td>
</tr>
<tr>
<td>PS#1</td>
<td>5.4m, 7m, 10.8-11.3m, 15.1m</td>
</tr>
<tr>
<td>PS#2</td>
<td>4.8m, 5.7m, 8.4m, 12.4m, 15m</td>
</tr>
<tr>
<td>PS#3</td>
<td>8m, 8.9m, 9.8m, 13.2m</td>
</tr>
</tbody>
</table>

Scala Penetrometers

A series of seven Scala Cone Penetrometer soundings were performed on the slope below the highway to give some continuity across the slope, and to give a penetration profile for the trench excavations. The Scala penetration plots are presented in Appendix 2. The Scala profiles exhibit extremely low point resistance values from ground level to at least 1.5 m and usually 2.5 m. Below this the value increase to refusal. Rod friction was significant in these materials, so the actual penetration values are probably of limited meaning. The depth of major penetration resistance changes compare well with depths to the zone of shearing observed in trenching and other cone penetrometer testing.

3.4.2 Trenching

a] Introduction

Three trenches were excavated to a depth of 3 m to 3.5 m below the existing ground surface using a hydraulic excavator. The location of the trenches is shown in Figure 3.4. The purpose of the excavations was to:

a] attempt to visually identify the depth of
shearing within the slope.
b) identify the lithologies present within the soil profile to correlate with penetrometer testing, and to obtain samples for laboratory testing.

b1 Trench 1: This trench, trending 315° was located (Figure 3.4) parallel to the fence at the edge of the road reserve and perpendicular to the left lateral margin of the earthflow which crosses the area. Logging of the lower materials in the trench was performed outside the trench for safety reasons.

The lithologies exposed (Figure 3.16) beneath a 5 cm to 10 cm black topsoil were mostly cohesive clayey silts with moderate to high plasticity and up to 10% mudstone and siltstone chips. Broad colour layering of heavy clayey silts occurs to a depth of 3 m where a 0.1 m to 0.3 m dark grey, soft to very soft, wet, plastic silty clay overlies highly sheared, elongate boulders of dark greenish grey, weak to moderately strong, fractured siltstone in a sheared matrix of moist grey silts, clays and siltstone chips. Small flows of water were observed at 3.1 m and 3.3 m in the centre of the 7m long trench (Figure 3.17). Flows were observed briefly at 1.3 m and 2.8 m, at low points in the undulating layering, but flow ceased after smearing of the sides by later bucket movement. No evidence of vertical fluid movement was observed in the exposure. The 3-3.3 m depth range is inferred to be the location of a failure plane on top of the blue green sandstone/silt unit. The graphic log of the trench is shown in Figure 3.18, with the full trench in Appendix 3.

c1 Trench 2: This trench, trending 220° is located immediately downslope of the borehole at the base of the downslope scarp (Figure 3.4). Depth varied from 3.4 m to 2.2 m because of the slope of the ground surface. The graphic log of the trench is shown in Figure 3.19, with the full trench in Appendix 3.
Figure 3.16: Trench 1. Note the dark colour change at the base of the trench. Sliding occurs at change.

Figure 3.17: Trench 1, 3 m. Clean water flow (at point of pencil) in the shear zone. Note contrast from smeared clay above the pencil to granular sandy material below.
Figure 3.18: Graphic Log of Trench 1.
Figure 3.19: Graphic Log of Trench 2.
Figure 3.20: Eastern (road) end of Trench 2. Note the blue colour change in the base of the trench.

Figure 3.21: Sheared zones within Trench 2. a] 1.5-1.7m "wedge shears" b] slickensided basal shear plane.
The upper 1 m to 1.5 m of the trench is material from the embankment upslope as evidenced by the granular roading material interspersed with the light brown sometimes speckled, moist to wet silts and a buried soil horizon (Figure 3.20). From 1.5 m to 1.7m on the southern side of the trench a series of polished clay surfaces dipping at 40° to 50° occur which form small wedge failures in dark grey, dry to moist, stiff to very stiff clayey silt (Figure 3.21a). The polished surfaces are sometimes cut by later surfaces, but no pattern is evident. This sort of unit would be an ideal target for LSPI, having obvious fissuring and high clay content. No water flow or staining and alteration from previous flow was observed through this material despite the apparent porosity.

A well defined undulating shear zone is present at a depth of 1.9 m to 2.3 m in very highly sheared, blue grey to dark reddish brown to creamy brown, wet, highly plastic, silty clay with pockets of crushed fragmented siltstone and mudstone. The relief on the base of the shear zone is caused by material riding over boulders of hard slightly calcareous siltstone in the greenish blue to grey, dry to moist, firm to hard, highly deformed and sheared, silt and clay which constitutes the local base lithology (Figure 3.20). Slickensides were present in the lower part of the sheared layer on many surfaces in the sheared zone (Figure 3.21b), sometimes completely surrounding pods of material. No flowing water was observed. Most moisture encountered was either in the upper 1.5 m associated with oxidised soil conditions, or in the sheared zone. This excavation compares well with Trench 1, although the soils above the shear zone are more diverse.

**d) Trench 3:** This trench was located immediately below what was the original Panikau Road-State Highway 35 intersection (Figure 3.4). The pavement seal coat and eroded basecourse were visible at the top of the backscarp above the excavation. Within the trench, a 5 cm to 10 cm black organic
soil overlies a 1 m to 1.5 m thick layer of road fill debris (loose to compact, moist to saturated, orangy brown, sandy gravel aggregate with clay matrix) (Figure 3.22). Water flow was present in the base of this horizon above a soft to stiff, dense, moist to wet, light creamy blue clay which underlies it. The clay layer is 0.3 m to 0.5 m thick and acts as a local aquiclude. Below this is a moist to saturated, black, organic horizon with large amounts of degraded woody material. Thickness is variable because of the compressibility of the material. This material indicates that the site of the fill embankment may not have been cleared completely before construction began in 1967-68, which may have had some influence on the stability when it failed. The organic material is underlain by a 1 m thick layer of very dense, stiff to hard, moist, creamy white, silty clay with brown and blue grey flecks (This is T3/2.1-3.1, the material used in the testing programme).

The base of the trench was the same highly deformed bluish grey silt with hard greenish grey siltstone boulders logged in Trenches 1 and 2. No well defined shear surface was observed in this trench. A graphic log of Trench 3 is shown in Figure 3.23, and the full trench in Appendix 3.

**Synthesis**

The trenches were all broadly similar, with a 2 to 3 m sequence of moist to saturated clay rich soils overlying a highly deformed bluish grey silt with hard greenish grey siltstone boulders. A shear zone was identified in Trench 1 and 2, above the bluish grey silt unit. Trench 1 had a relatively uniform series of clayey silts, but Trench 2 and 3 are more variable with buried organic material and soil horizons in the sequence. Water seepages tend to be controlled by layering within the sequence, notably above shear zones.
Figure 3.22: Trench 3. Note the thick organic horizon, and the blue colour change in the base of the trench.

Figure 3.23: Jacro Drill Rig
Figure 3.24: Graphic Log of Trench 3.
3.4.3 Drilling.

(a) Programme

A borehole was drilled in June 1990 by the Works Consultancy Services Napier Drilling Unit. Drilling was performed over 3 days on site, with logging completed subsequently at Works Napier Engineering Laboratory. The borehole is located between pegs A1 and A2 close to the backscarp west of the State Highway (Figure 3.4). Significant loss of core occurred as an artifact of the drilling process. Caving ground conditions were also encountered between 5 m and 6.15 m, and from 11 m to the end of the borehole (EOB). Difficult drilling conditions lead to the decision to end the borehole at 13 m.

(b) Borehole Lithologies

A log of the borehole is presented in Appendix 3. The borehole log indicates an initial thickness of "Panikau red metal" basecourse overlying a moist firm tan brown clayey silt subgrade with intermixed siltstone aggregate to 2 m. Below this level to 3.3 m the moist to wet, soft to firm, clayey silt materials are similar but with a lot of colour variation (pinkish brown, yellowish green, mottled creamy brown) and the same occasional aggregate of hard mudstone and siltstone. The material is contorted as might be expected in a compacted fill, but not sheared as sub 3.3 m lithologies are. This is inferred to be the fill material placed in the original realignment.

A thin layer of organic material at 3.25 m marks a change in lithology to a highly sheared light brown to greenish grey firm to soft clayey silt with chips of hard silty sandstone from 5 mm to >30 mm. Drilling fluid circulation improved below 3.5 m to 6 m. The sequence is not recognisable as a sedimentary because of the level of shearing that has occurred. The observed deformation is an
artifact of shearing in geological conditions not active at the site today. Siltstone and sandstone material has been very highly polished by shearing, and the softer clays and silts have been smeared around them. The intensity of shearing is variable, but commonly most intense around blocks of hard competent siltstone and sandstone. The less competent clays and silts are plastic, firm to very stiff and commonly moist. Moisture content variation on core recovered is shown in Figure 3.. These lithologies could best be described as a broken formation, or even melange if the competent sandstone and siltstone were not originally interbedded. The deformation is typical of lower Tertiary units (Ridd, 1967) adjacent to uplifted Whangai Formation in the Whangara area.

As discussed previously the rapid changes in competence caused significant core loss and jamming of the core catcher in these materials. A caving zone between 5 m and 6.15 m with a 20% circulation loss and numerous blackened shear zones, and lost core from 6.15 m to 6.45 with complete loss of circulation indicate a weakened zone. This depth range is represented by a low point resistance zone in penetrometer sounding DS#2. Based on a 3 m fill and about 2.5 m to the shear zone in Trench 2, this could be the failure zone. Below this area the circulation is 100% to the EOB, and moisture contents are markedly lower (Figure 3.25). The slope failure zone is inferred to be in the 4.9 m-5.3 m depth range. A 1 cm thick soft, highly plastic, blackish grey gouge material was logged at 5.2 m.

The rest of the borehole correlates well with the DS#2 penetrometer sounding, with hard fractured cemented sandstone and siltstone corresponding to major point resistance peaks and zones of sheared dark greenish grey clay and silts to troughs. The sequence alternates between the hard and firm to very stiff materials. The degree of shear deformation is high in all materials, but not as intense as in the 4.9 m-5.3 m range where black gouge-like sheared clays occur.
Figure 3.25: Borehole moisture content profile.
3.5 Slope Failure Classification and Mechanism

3.5.1 Classification

The Panikau Summit slope failure, classified according to Varnes (1978), is principally a complex slump-earth flow. The original slope had pre-existing earthflow features typical of these areas of highly deformed early Tertiary clay rich lithologies. In the vicinity of the highway the placement of fill on the existing earthflow probably reactivated the earthflow movement, and precipitated the embankment failure. Since the embankment failure caused disruption of the new road the slump-earth flow classification is used. Most material involved in the failure is fine grained clayey silt with broken siltstone and sandstone aggregate and boulders, but the debris classification is avoided as the variability of the materials is a natural feature of the in-situ soils. The rate of failure is presently very slow to extremely slow [max: 1cm/month](Varnes, 1978). Secondary solifluction movement is common on the surface of the slope, but not the major mode of failure at the site.

3.5.2 Failure Mechanism

The original failure in terms of this study was the slump failure of the newly constructed embankment beneath the Panikau Road intersection in 1969. This does not take into account the condition of the slope prior to 1969, which undoubtedly influenced the failure of the whole slope rather than simply the embankment. The pre-existing highly sheared nature of most lithologies at the site is significant to the shear strength of the slope. Aerial photographs of the site prior to construction indicate that earthflow features were already in existence on this slope and on others in the Whangara area prior to the highway realignment in 1969. The construction of fill areas as part of the new highway over pre-existing earthflows was part of the failure mechanism.
Since the natural 12° to 15° slopes are only marginally stable initially (section 4.3.7), the weight of a fill up to 3 m high (40-50 kPa) on the slope would be sufficient to cause an increase in the effective stress beneath the fill and induce some form of slope failure. The apparent lack of permeability of the clay rich soils would not allow ready dissipation of pore pressures.

Other factors in the failure mechanism include the method of fill construction, and the control of water on the slope. The subsurface investigation indicates that site preparation was minimal based on the 0.4 m thickness of decayed organic material found in trench 3. Further conclusions regarding the construction of the fill areas can not be made because of the lack of construction records. The drainage of the new highway is not well defined and, as previously discussed, is suspected to be a significant factor in the original 1969 embankment failure. The combination of these factors and the creep prone original slope is inferred to be a an adequate failure mechanism. Twenty years on it is difficult to place one factor as most significant.

3.6 Synthesis

The engineering geological site investigation of the Panikau Summit site indicates that the area has a 2 m-3.5 m thickness of moist to saturated clayey silts overlying a shear surface on top of a unit containing highly deformed bluish grey silt with hard greenish grey siltstone boulders. The ground surface is uneven and swampy, and visible earthflow features occupy much of the slope. The sequence observed in excavations is broadly layered, and water seepage observed tends to occur between layers. No evidence of possible vertical seepage paths was observed with the exception of the unit in Trench 2 with polished fracture surfaces at a depth of about 1.5 m. Lithologies are commonly clay rich and relatively impermeable. The slope movement recorded from surface peg stations was relatively small close
to the State Highway, with all displacement vectors in the
down slope direction. This evidence from a short survey
period is consistent with a slow creeping slope failure. The
slope morphology at the site is very common in the hill
country around Panikau.

The initial failure in 1970 of the Panikau Road
intersection was influenced by additional factors,
particularly the drainage from the subsoil drain on the
upslope side of the State Highway seeping through the
subgrade to the western (downslope) side of the road. The
magnitude of the seepage can not be defined, but the failure
of the newly constructed fill embankment suggests that
destabilising forces present at the time of failure were
significant. The failure continued in the same area in 1971
before Panikau Road was finally relocated in its present
position. The western (downslope) side of State Highway 35
has been subject to on-going subsidence, as shown in Figure
3.4, since 1971. Arcuate depressions have formed on a regular
basis in the north-bound lane, and a block on the edge of the
embankment has also failed. Regular maintenance has filled
and sealed these depressions, but the features reoccur within
12 to 18 months.

The application of a deep stabilisation method to this
site should limit the on-going deformation at the site so
that a stable road configuration could be produced,
significantly reducing the maintenance costs for the site.
Chapter 4: Laboratory Investigations

4.1 Introduction

4.1.1 Purpose

The purpose of the laboratory phase of the study is to:

a) Identify the basic soil parameters and index parameters of soils from the Panikau Summit Site.
b) Determine the lime responsiveness of the soils at the site.
c) Evaluate methods of testing used for Lime Slurry Pressure Injection (LSPI) and Lime Column application.
d) Evaluate the potential for Deep Lime Stabilisation at Panikau.

4.1.2 Method

All soils obtained from trenches excavated with a hydraulic excavator and were immediately sealed in plastic bags. In-situ shear vane testing was performed as was safely possible in the trenches. Soil testing using appropriate New Zealand Standard 4402 Test Methods was completed at the WORKS Consultancy Laboratory in Napier. X-ray analysis was undertaken at the University of Canterbury Geology Department.

4.2 Natural Soil Parameters of Soils at Panikau Summit

4.2.1 Introduction

The plasticity index (PI) ranges from 18 to 50, as indicated in Figure 4.1, and Table 4.1. Higher values occur in the units overlying the bluish green mudstone/sandstone unit at or about 3 m-3.5 m below the ground surface in the area downslope of the highway embankment. Most observed slope
Table 4.1: Soil Parameter Summary Table.

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>w</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>LS</th>
<th>ICL</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1/1.8</td>
<td>13.3</td>
<td>55</td>
<td>22</td>
<td>33</td>
<td>11.33</td>
<td>1</td>
</tr>
<tr>
<td>T1/2.6</td>
<td>28</td>
<td>67</td>
<td>24</td>
<td>43</td>
<td>16.5</td>
<td>2</td>
</tr>
<tr>
<td>T1/3-3.1</td>
<td>30</td>
<td>65</td>
<td>21</td>
<td>44</td>
<td>17.4</td>
<td>3</td>
</tr>
<tr>
<td>T2/-3.1</td>
<td>11.64</td>
<td>45</td>
<td>19</td>
<td>26</td>
<td>11.7</td>
<td></td>
</tr>
<tr>
<td>T2/2</td>
<td>17.4</td>
<td>54</td>
<td>20</td>
<td>34</td>
<td>13.6</td>
<td>2</td>
</tr>
<tr>
<td>T2/0.5</td>
<td>22.1</td>
<td>50</td>
<td>32</td>
<td>18</td>
<td>12.9</td>
<td></td>
</tr>
<tr>
<td>T2/1.5</td>
<td>23.6</td>
<td>57</td>
<td>21</td>
<td>36</td>
<td>15.6</td>
<td>2</td>
</tr>
<tr>
<td>T2/-3.5</td>
<td>16.9</td>
<td>51</td>
<td>20</td>
<td>31</td>
<td>14.53</td>
<td>3</td>
</tr>
<tr>
<td>T2/0.9-1.8</td>
<td>21.7</td>
<td>58</td>
<td>24</td>
<td>34</td>
<td>13.86</td>
<td>3</td>
</tr>
<tr>
<td>T3/2.1-3.1</td>
<td>35.6</td>
<td>74</td>
<td>24</td>
<td>50</td>
<td>15</td>
<td>4</td>
</tr>
<tr>
<td>T3/-3.1</td>
<td>24.2</td>
<td>59</td>
<td>23</td>
<td>36</td>
<td>13.47</td>
<td>3</td>
</tr>
<tr>
<td>T3/2</td>
<td>26.1</td>
<td>64</td>
<td>22</td>
<td>42</td>
<td>14.8</td>
<td></td>
</tr>
<tr>
<td>Commercial Bent.</td>
<td>365</td>
<td>43</td>
<td>322</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tarndale Bentonite</td>
<td>244</td>
<td>23</td>
<td>221</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

w = natural water content; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; ICL = Initial Consumption of lime

![Plasticity Chart](Figure 4.1: Plasticity Chart for Panikau Summit Soils.)
movement is thought to occur above this depth as slow, creeping failure. Table 1 also indicates the natural water content of the soils to the Atterberg Limits. The values shown indicate that most of the soils at this site are highly plastic. The soils are mostly wet silty clays and clayey silts. T3/2.1-3.1 (trench 3, depth 2.1 m to 3.1 m) was selected as the principle testing material since it was close to the part of the slope that failed following the construction of the highway embankment, and because it was able to be obtained in quantity relative to the other units. The soil also had a natural water content 11% greater than the plastic limit, which gave it a good probability of contributing to instability in that part of the slope.

4.2.2 Changes in Atterberg Limits with Increase in Percentage Hydrated Lime

Additional Atterberg limit testing using T3/2.1-3.1 soil with 2%, 4%, 6%, and 8% hydrated lime to determine the change in consistency is presented in Figure 4.2. and in Figure 4.1. Soil was mixed with hydrated lime and sealed in a plastic bag for 7 days, and then tested (NZS 4402:Tests 2.2, 2.3, and 2.4). The trends apparent show that the plastic limit is most affected by the hydrated lime treatment. The trend of points A through E on Figure 4.1 show the small change in liquid limit, but a drop from 50% to 24% in the plastic limit. Figure 4.2 shows the related drop in plasticity index (PI). The change in soil parameters is immediate, with the 1.5% hydrated lime sample indicating a definite improvement (decrease) in the plasticity index. The rate of change levels out at 6% hydrated lime, with little change between 6% and 8% hydrated lime.

These results indicate that the T3/2.1-3.1 soil is lime responsive, and will continue to improve (with respect to PI) with the application of up to 6% hydrated lime.
Figure 4.2: Trends in Liquid Limit, Plastic Limit and Plasticity Index with increasing percent hydrated lime.

Figure 4.3: Initial Lime Consumption Test Equipment
4.2.3 X-Ray Diffraction Clay Identification

a) Purpose

The purpose of the testing was to determine the true mineral composition of the "bentonitic" soils at the site. "Bentonite" is not itself a mineral name, but rather a synonym for the montmorillonite mineral group which includes smectite clays. "Bentonite" is commonly used on the East Coast of the North Island to describe swelling clay soils, sometimes called "Wanstead Soils" after the lower Tertiary Wanstead Formation, in the region.

b) Method

A series of X-ray diffraction scans using Ni-shielded CuKα radiation on clay samples taken from trenches at Panikau Summit were performed using the University of Canterbury Geology Department X-ray diffractometer (Appendix 5). Samples from Panikau Summit were scanned from 2°-50° 2θ for each oriented (air dried) and glycolated slide, and 2°-30° 2θ for slides heated to 550°C for 1 hour. Selection of samples was based on the stratigraphic occurrence of soils relative to the inferred sheared zones in trenches excavated. Two samples (CMB039 and CMB041) were taken from below the shear zones on the greenish blue sheared siltstone/mudstone, with the remainder from sheared zones above this unit. Reduced copies of the scans are contained in Appendix 5.

c) Identification

The method of clay identification was based on the occurrence of, and movement of signature diffraction peaks in the 3 scans (orientated, glycolated, and heated 550°C) for each sample. Diagnosis and final identification were performed using methods outlined in Eslinger and Pevear (1988), with reference to Srodon (1980), and Fergusson (1985). No further investigation of cation exchange capacity
or similar laboratory testing was included in the study. Attempts to determine the proportions of illite:smectite in samples were made using methods outlined in Srodon (1980).

The mineral assemblages identified by X-ray analysis all contained smectite, illite, kaolinite, quartz and some minor calcite and chlorite. The major smectite peak in the orientated plots occurs at 15Å which indicates that the smectite has a divalent interlayer cation, either Ca$^{2+}$ or Mg$^{2+}$, with two hydrated layers (Eslinger and Pevear, 1988). The 15Å peak swelled to 18Å on glycolation, and disappeared on heating to 550°C. That change identifies the smectite as a low-swelling Cheto-type montmorillonite (Eslinger and Pevear, 1988). No Na$^+$ 12.5Å peaks were observed (high-swelling montmorillonite). Chlorite was identified in several of the scans, notably CMB039 and CMB041, both from the greenish blue sheared siltstone/mudstone unit above which most deformation is thought to occur. Detail of all identifications is presented in Appendix 5 together with XRD Scans.

4.2.4 Initial Lime Consumption Testing

a) Introduction

Lime response testing is an initial indication of the percentage of lime required to stabilise a soil. The test measures the amount of lime required to saturate a standard soil suspension with respect to Ca(OH)$_2$, as measured by the pH of that solution. Eades and Grim (1966) developed the original lime response test, and this study uses a refinement of the original test developed by Clauss and Loudon (1971). The revised test uses a correction factor that accounts for the lime consumption of the water used in testing.

b) Method

An Orion Research Digital pH meter with a combination electrode was used in the test (Figure 4.3), and calibrated
Figure 4.4: Estimated Creep Limit chart for a single column (after Broms, 1984).

Figure 4.5: Estimated relative shear strength with curing period. (after Broms, 1984)
with a pH 6.5 buffer control solution between each reading. Individual readings (to ±0.5 pH units) were taken after a 3 minute period to ensure a stable reading. Hydrated lime from Websters Hydrated Lime, Havelock North, was used in the test, and percentages were by dry weight of soil.

c) Results

The values for the initial consumption of lime (ICL) are given in Table 4.1. Most samples achieved a relatively stable solution pH from 3-4% hydrated lime. The values obtained are lower than expected from Eades and Grim (1966) and Clauss and Loudon (1971). One possible reason for this is the aggregates that formed as material dried in several cases, notably T1/1.8. This sample had aggregate lumps remaining even after prolonged soaking in the lime solutions. The values obtained are thought to be conservative, though they do match the plasticity index decrease observed for the T3/2.1-3.1 material shown in Figure 4.2.

4.3 Strength Testing

4.3.1 Introduction

a) Lime Columns

The laboratory test programmes for LSPI and Lime Columns are quite different in method and development. The design of a lime column foundation is mainly based on unconfined compression testing of soil mixed with 5-10% quicklime (CaO) and cured for 1 day to 360 days after mixing. From this the development of bearing capacity and shear strength can be estimated (Figure 4.4 and 4.5), and the required number and spacing of lime columns calculated. The lime columns can then be incorporated in foundation design similar to using conventional piles (Holm et.al.(1981). Broms (1984) describes all aspects of the design, testing and implementation of the lime column method in considerable detail. The use of
Unconfined Compression Strength (UCS) samples 36mm diameter by 72mm long manufactured using a Harvard Miniature Compaction Device for laboratory testing is the method presented by Broms (1984). Samples need to be sealed from the atmospheric CO₂, and are generally wrapped in plastic bags and stored under water until they are due for testing. Laboratory testing in Holm et.al.(1981) is corroborated by field testing using a purpose built lime column penetrometer, and a Menard pressuremeter. Field examples presented in Holm et.al.(1979) show comparable strength values are able to be obtained.

b) LSPI

Literature on the LSPI method is not so specific regarding the testing programme. The main reference (Boynton and Blacklock, 1985; page 23), states:

"The use of standard ASTM soil tests to evaluate potential injection stabilisation sites is not recommended. During the past 10 years, LSPI contractors and participating engineers have worked to develop specialised methods for predicting success of potential LSPI sites. Some of these tests are modifications of ASTM tests to account for the stabilising effect of lime seams, and others have been specifically developed for this purpose. Compression and shear strength tests should be used to evaluate sites with low strength soils; swell tests to evaluate sites with potential settlement problems. Other standard classification tests which give indirect indication of soil properties such as Atterberg Limits should only be used as preliminary tests or not at all, and should never be used in place of strength or volumetric tests."

The specialised testing mentioned in the previous quotation is described later in the same publication as:
Figure 4.6: Types of lime glaze stabilised compression specimens. (after Blacklock and Wright 1986)

Figure 4.7: Lime glaze consolidation and swell specimen. (after Blacklock and Wright 1986)
a) the Glaze Stabilised Compression Test
b) the Seam Stabilised Compression Test;
   (1) straight split seam
   (2) angle seam
c) the Glaze Stabilised Consolidation Test
d) the Seam Stabilised Swell Test

Line drawings of these test specimens are shown in Figures 4.6 and 4.7. The tests all involve the application of a lime, or lime and fly ash slurry to natural (but more commonly remoulded) candidate soils. These samples are then cured with a matching untreated control, and the change due to the application of the slurry evaluated with the appropriate test method. The basic concept is to try and simulate lime seams in the natural soil. Further to these specialised tests are a series of UCS tests to establish the slurry mix design which are not discussed in detail in any LSPI literature reviewed. It appears that detailed evaluations of testing programmes are not presented for commercial reasons, as the matching of a particular site to a type of injection programme is critical to the successful application of the method.

The bulk of the literature on LSPI is produced by authors with connections to the Woodbine Corporation in Texas, a specialist contractor in LSPI and other forms of foundation improvement. Almost all LSPI equipment development and applications of LSPI published have involved this company in some way.

4.3.2 Sample Preparation Method

a) Introduction

Initially it was intended to use 36mm diameter remoulded samples obtained by pressing stainless steel sampling tubes into standard Proctor moulds of T3/2.1-3.1 soil with the
particular percentage of hydrated lime compacted according to New Zealand Standard 4402: Test 4.1.1. The samples would then be extruded and cured for 4, 7, 14, and 28 days in sealed plastic bags in a water bath. Problems were encountered with the stability of the samples to the extent that they were abandoned. Samples that survived extrusion often broke along compaction levels prior to testing, and to avoid this problem a whole Proctor sample was used to determine the UCS value for each curing period and lime percentage. The dimensions of the Proctor sample are not ideal for UCS tests because the 105mm diameter by 115mm samples are less than the optimal 2:1 length to width ratio, but the samples were robust enough to test, and survive curing. No correction factor to account for the dimensional ratio of the Proctor samples has been applied in this study. The main purpose of the testing programme was to prove that the UCS values increased in strength with time and percentage lime. Comparisons were relative to the untreated control specimen.

b) Discussion

A topic of significance at this point is the method of preparation of samples for this type of programme. The lime column method mixes quicklime with soil in-situ and imparts a limited degree of compaction by virtue of the reversing of the pitched blades on the mixing head, so test compaction is acceptable. The LSPM method imparts no compactive effort at all to the soil mass, so testing with compaction is not desirable. Blacklock (1982) utilises a static moulding method (ASTM D 1632-63) which involves no kneading of the soil as Proctor compaction does. The kneading method requires preliminary adjustment of the moisture content and dry density to obtain a specimen consistent in moisture content and dry density as well as surface texture, appearance, and constant dimensions.

Problems arise with clay rich soils in achieving a uniform moisture content, and uniform density distribution
within the samples. Blacklock (1982) reports that with experience and the appropriate equipment, consistent samples can be produced. Most authors including Blacklock (1982) acknowledge the difficulties associated with laboratory testing for LS PI. Static moulding equipment is also not commonly available in New Zealand laboratories. The use of the Proctor moulded specimens utilises readily available equipment, and a common compaction standard.

4.3.3 Strength Development with Increase in Percentage Hydrated Lime

a) Method

Based on the initial consumption of lime test in section 4.2.4, 4%, 6% and 8% percent (by weight) hydrated lime were selected to evaluate unconfined compressive strength (UCS) development with time. T3/2.1-3.1 soil at field moisture content was mixed in a large stainless steel tray, and physically cut into blocks small enough to pass a 19mm standard sieve. Hydrated lime was mixed into the soil as quickly as possible, and then mechanically compacted in standard Proctor moulds to New Zealand Standard 4402:Test 4.1.1 (3 layers at 27 blows per layer). The samples were extruded, sealed in 2 plastic bags and stored in a water bath for the curing period to attempt to approximate ground temperature. UCS testing was performed using a Shimadzu Universal Testing Machine (Type RH-10 T.V. No: 51586) at the Napier WORKS Consultancy Engineering Laboratory. Figures 4.7 4.8, and 4.9 show UCS plotted against strain for 4%, 6% and 8% hydrated lime with T3/2.1-3.1 soil.

b) Results

The 4%, 6%, and 8% UCS plots show an increasing trend of strength gain with increasing hydrated lime content and increasing curing time for each percentage hydrated lime. The change in the shape of the curves with increasing lime
Figure 4.8: Unconfined Compressive Strength against strain for 4%, 6%, and 8% hydrated lime.
content matches the change in the observed mode of failure from plastic, in the untreated sample (Figure 4.9) and the 4% curve (flattens out with no peak value), to increasingly brittle (Figure 4.10) in the 6% and 8% curves (a distinct peak value with a subsequent decline). The peak UCS values obtained are presented in Figure 4.11. The 4% curve is significantly lower than the 6% and 8% curves. Improvement is noted between the 6% and 8% curves, with the 6% 14 and 28 day values being obtained in 7 and 14 days with 8% hydrated lime. Most strength gain occurs in the first 14 days. From these results, the inference is that the T3/2.1-3.1 soil is lime responsive, and strength gains could be expected from the application of either LSPI or lime columns in this material.

c) Discussion

Results are based on single sample tests for each curing period and lime content, so no sample variation can be predicted. Material preparation and manufacture is simple, which ensures that all samples at a particular percentage lime are initially identical. Natural soil at field water content was used since this would be the case in any application of deep lime stabilisation. The selection of New Zealand Standard 4402:Test 4.1.1 was essentially to use a common standard that could be repeated with confidence. No attempt to simulate the lime column method is implied in using this compaction standard. Density and moisture content are effectively constant within sample batches, which makes the UCS values virtually independent of these sample properties. The observed results, using conventional methods and equipment, should accurately indicate the effects of lime on the soil over time.

Broms (1984) and other Scandinavian authors indicate that their results using the Harvard Miniature Compaction Device tend to underestimate the strength increase in lime columns in-situ by a factor of 3-5 for samples cured for 1-3 months. A similar relationship could be developed for New
Zealand conditions using Proctor moulded specimens or the Harvard Miniature Compaction Device. This would be an important goal for a future field trial for Lime Columns. Until this relationship is established, the principal purpose of conventional testing should be to determine if soil strength can be improved by the addition of lime, as this is the essential prerequisite for the application of any form of lime stabilisation.

With development and refinement, the degree of strength gain in the laboratory testing could be used in the design of a deep stabilisation project, but would have to be based on a much greater data base than this study. The overseas developments of both LSPI and the lime column method are based on extensive trials and comparative testing of laboratory and in-situ material.

4.3.4 Type of Lime Used

The testing in this study has mainly involved hydrated lime because it was commonly available in the laboratory where most testing was performed. The use of quicklime in laboratory testing would be preferable for a lime column project evaluation where it would be used in construction. The lime column method is more dependent on the development of bearing capacity and shear strength within the column of treated soil, and derives beneficial effects from the exothermic CaO hydration reaction which will improve the rate of strength development.

The Scandinavian examples of the lime column method in Broms (1985) involve soils with very high moisture content, frequently greater than the liquid limit. This was sufficient water to hydrate the CaO to make Ca(OH)₂ available to react with the clay fraction of the soil. Ekstrom and Trank (1979) in Broms (1985) found a 15% reduction of average water content on the addition of 10% CaO to a clay soil with an initial water content of 60%. This may be significant when
considering the relatively low natural water content of the range of soils shown in Table 1. There may be a lower water content limit on the method related to available moisture in the target soil. If insufficient water is present in the soil column then the hydration reaction would be limited, and the stabilisation effect decreased. This should be determinable from a laboratory testing programme.

Hydrated Lime reacts directly with the clay soil without the heat CaO generates, but the strength gains are slower and lower for a given curing period (Bell, 1989). Broms and Boman (1979) show that CaO (unslaked lime) gives greater shear strength increase than Ca(OH)₂ (slaked lime) with a silty varved clay and an organic silty clay over a 400 day test period. Most target soils for lime stabilisation have high water contents. The LSPI method has no similar limitation if CaO is used, as the hydration reaction (slaking) occurs when the CaO is mixed with water in the mixing tank prior to injection.

4.3.6 Lime Slurry Glaze Testing

a) Method

Following the method of Boynton and Blacklock (1986), a series of lime slurry glaze samples were prepared from T3/2.1-3.1 soil. The stability of remoulded soil in 36mm diameter tube samples was poor, so 36mm diameter samples were cut from blocks of the natural soil. The amount of suitable natural soil blocks limited the number of samples prepared.

Samples were rolled in a shallow tray containing a quicklime slurry (0.3kg/litre) for several minutes, with the heavy paste smeared over the samples using a spatula during that time. The slurry temperature was initially too hot to handle, but after a few minutes in the shallow stainless steel tray the heat of the slurry began to dissipate. Samples were then wrapped in 2 plastic bags and stored in a water
Figure 4.9: Failed T3/2.1-3.1 natural (recompacted) specimen.

Figure 4.10: Failed 6% hydrated lime 4-day specimen.
Panikau Summit
T3/2.1-3.1: UCS/Time for 4%, 6%, 8% Lime

Figure 4.11: Unconfined Strength development over time for increasing percent hydrated lime.

Panikau Summit
T3/2.1-3.1, Water Glaze, 4 Day

Panikau Summit
T3/2.1-3.1, Quicklime Glaze, 4 Day

Figure 4.12: 4-Day Water and lime glazed UCS results.
Figure 4.13: 7-Day lime glaze UCS results.

Figure 4.14: 14-Day water and lime glaze UCS results.
Figure 4.15: Seam-cut glaze specimen UCS results.

Figure 4.16: ELE Shear Testing Frame, Napier Laboratory.
bath. Control samples were rolled in a tray of distilled water and stored in plastic bags in the water bath. The results of UCS testing are presented in Figures 4.12, 4.13 and 4.14.

A set of six quicklime glazed samples were manufactured with artificial defects as detailed in Boynton and Blacklock (1985), and cured for 14 days under the same conditions as the glazed specimens. The six samples included four 45° angle-cut specimens, and two straight split specimens. After 14 days only three angle-cut and one straight seam sample remained intact. The UCS results are presented in Figure 4.15.

b) Results

The UCS results obtained have a large scatter at each curing period, and between curing periods. The results from this limited sample population do not show any strong trends in strength increase with time following glazing with the lime slurry. The control water glazed specimens showed little difference from the lime glazed specimens, and UCS values were consistent. The glazed specimens all had shallow surface cracking, and some were twisting asymmetrically. One specimen in the 4-day glazed group broke during the "squaring off" of the twisted cylinder ends prior to testing. The UCS of seam cut specimens was well below that of the glazed specimens with the exception of the surviving straight-seam sample which produced a UCS curve similar to the Proctor samples at higher lime contents. The failure mode in the glazed and angle-cut specimens was entirely plastic, with a more brittle failure in the straight-seam specimen.

c) Discussion

The size of the sample population is too small to infer much from these results, but further experimentation with the cut specimens may indicate more about their validity as an
evaluation test. These samples were difficult to make, handle, and test. The method is thought to be less useful than the previous UCS Proctor series in terms of an evaluation method for deep lime stabilisation because of the difficulty in producing uniform specimens that will survive the curing period. Blacklock (1982) acknowledges these problems in attempting to simulate the in-situ LSPI vein network, and points out (with the benefit of more testing than this study) that results obtained do not often represent actual strength values in-situ following LSPI.

The target soil(s) response to lime, and the appropriate in-situ ground conditions are a better gauge of the site suitability for LSPI. If the soils are lime responsive and have natural voids, then the LSPI seam network should be able to be injected through the soil mass. Stabilisation is effected by encapsulating blocks of natural soil in envelopes of lime modified soil adjacent to the seams. The resulting volume stability and virtually constant moisture content conditions increase the stability and overall shear strength of the soil mass.

4.3.7 Shear Box Testing

a) Method

A series of shear box tests were performed on natural T3/2.1-3.1 soil, and T3/2.1-3.1 with 1.5% hydrated lime to investigate the effects of relatively small amounts of lime on the cohesion and angle of internal friction ($\phi$). This was based on the estimate by LSPI contractors reported in Boynton and Blacklock (1985) that single injection LSPI represented between 1% and 1.5% lime by weight. Both materials were compacted into Proctor moulds, sealed in plastic bags and cured for 7 days. Samples were then cut from the moulded specimen and tested using a shear box machine at Napier Works Laboratory (Figure 4.16). The results are presented in Table 4.2. The normal load range reflected the insitu overburden
Table 4.2: Shear Box Results

<table>
<thead>
<tr>
<th>Material</th>
<th>Peak c'</th>
<th>0'</th>
<th>Residual c'</th>
<th>0'</th>
</tr>
</thead>
<tbody>
<tr>
<td>T3/2.1-3.1</td>
<td>12.5</td>
<td>13.5</td>
<td>0</td>
<td>18</td>
</tr>
<tr>
<td>T3/2.1-3.1 + 1.5% lime</td>
<td>8.5</td>
<td>33</td>
<td>11.8</td>
<td>14.5</td>
</tr>
</tbody>
</table>

Table 4.3: Infinite Slope Factor of Safety Using Shear Box Parameters.

<table>
<thead>
<tr>
<th>Material Parameters</th>
<th>F</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak c', 0'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Natural T3/2.1-3.1</td>
<td>0.36</td>
<td>Back Analysis to F=1 requires c=6.57 kPa</td>
</tr>
<tr>
<td>Residual c', 0'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5% Lime</td>
<td>1.62</td>
<td></td>
</tr>
<tr>
<td>Peak c', 0'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5% Lime</td>
<td>1.33</td>
<td>Back Analysis to F=1, c=7.76 kPa</td>
</tr>
<tr>
<td>Residual c', 0'</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.47: Shear Box Graphs

a) Natural T3/2.1-3.1

b) T3/2.2-3.1 (1.5% Lime)
loading at 3 m to 4 m depth, the inferred depth of failure in Trench 3. The samples were consolidated for 40 minutes prior to testing, and sheared at a rate of 0.1691 mm/minute. The best fit line was generated using the Grapher data plotting programme.

b) Results

The results indicate that substantial benefit develops in terms of the angle of internal friction of the soil stabilised with lime. This could indicate the type of soil modification possible after the application of LSPI. The residual cohesion and internal friction angle of the stabilised soil correspond to the peak values of the natural soil. The natural soil residual values compare well with ring shear testing results from the Mata Road Section of S.H. 35 in Franks (1989) [\(c' = 5\) kPa, \(\phi' = 20\), for normal loads between 50-150 kPa], and in Moody (1985) from the Makokomuka Valley on S.H. 35 [\(\phi' = 14\)]. Gibo et. al. (1987) quote ring shear values for a smectite clay in a Japanese landslide as \(\phi' = 18\), at very low normal stress, similar to this study.

c) Infinite Slope Analysis using Shear Box Results

A simple infinite slope stability calculation using slope dimensions similar to those observed in Trench 1 was performed to assess the results of shear testing with slope conditions that existed prior to road construction at Panikau Summit. Analysis of the constructed embankment was not attempted.

Using the infinite slope stability formula shown,

\[
F = \frac{c'}{\rho g \sin \beta \cos \beta} \left( \frac{\rho - \rho_{w} \tan \phi'}{\rho \tan \beta} \right)
\]

where

- \(F\) = Factor of safety
- \(c'\) = cohesion
- \(\phi'\) = angle of internal friction
\[ B = \text{slope angle} \]
\[ \rho = \text{bulk soil density} \]
\[ \rho_w = \text{density of water} \]

and assuming:

a] piezometric level at ground level (piezometers installed in 1972 indicate artesian conditions close to Trench 1),

b] slice thickness of 3.1 m
c] slope angle 15° and soil unit weight of 16 t/m³.

Factors of safety (F) were derived (Table 4.3). Back analysis for F=1 using the measured \( \phi' \), and \( c' \) as a variable are presented in Table 4.3.

On the basis of these results, the natural T3/2.1-3.1 soil is stable within the slope if peak shear parameters exist, but as residual shear parameters are mobilised the slope should theoretically fail. The slope failure that occurred immediately after the construction of the highway may have mobilised these shear conditions within the slope beneath the embankment. The loading of relatively impermeable saturated soils is more likely to have involved undrained shear failure, as discussed in the following section.

4.3.8 Shear Vane Testing

A Pilcon Direct Reading Hand Shear Vane was used to determine field undrained shear strength (\( C_u \)) values from in situ trench lithologies. All determinations used the 19 mm shear blade, with the blade at least 70 mm into the target soil. Rotation was as close to 360°/min as possible in field conditions, and as consistent as possible between individual tests. The residual value was determined 10 rotations after the peak shear determination. A water content sample was obtained from the material remaining on the blade following the test. Results are presented in Figure 4. No attempt to determine a relationship between water content and undrained shear strength was attempted as the sample populations for
<table>
<thead>
<tr>
<th>Peak (m)</th>
<th>Res</th>
<th>W%</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.0</td>
<td>5.0</td>
<td>29.7</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>17.0</td>
<td>5.0</td>
<td>22.3</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>16.0</td>
<td>4.0</td>
<td>25.2</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>20.0</td>
<td>6.0</td>
<td>26.4</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>26.0</td>
<td>5.0</td>
<td>26.8</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>17.0</td>
<td>5.0</td>
<td>26.8</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>32.0</td>
<td>6.0</td>
<td>30.8</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>30.0</td>
<td>14.5</td>
<td>31.4</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>36.5</td>
<td>17.5</td>
<td>32.3</td>
<td>T1/3-3.1</td>
</tr>
<tr>
<td>24.5</td>
<td>11.5</td>
<td>35.2</td>
<td>T1/3.1</td>
</tr>
<tr>
<td>30.0</td>
<td>14.5</td>
<td>31.4</td>
<td>T1/3</td>
</tr>
<tr>
<td>36.5</td>
<td>17.5</td>
<td>32.3</td>
<td>T1/3</td>
</tr>
<tr>
<td>38.5</td>
<td>21.0</td>
<td>35.3</td>
<td>T1/2.7</td>
</tr>
<tr>
<td>39.5</td>
<td>15.5</td>
<td>30.7</td>
<td>T1/2.4</td>
</tr>
<tr>
<td>33.5</td>
<td>11.0</td>
<td>26.7</td>
<td>T1/1.8n</td>
</tr>
<tr>
<td>41.0</td>
<td>15.0</td>
<td>32.1</td>
<td>T1/1.3n</td>
</tr>
<tr>
<td>41.5</td>
<td>10.0</td>
<td>19.8</td>
<td>T1/1.8s</td>
</tr>
<tr>
<td>66.0</td>
<td>9.5</td>
<td>20.4</td>
<td>T1/1n</td>
</tr>
<tr>
<td>44.0</td>
<td>7.0</td>
<td>16.4</td>
<td>T1/0.5n</td>
</tr>
<tr>
<td>41.5</td>
<td>14.0</td>
<td>25.3</td>
<td>T1/1s</td>
</tr>
<tr>
<td>40.0</td>
<td>18.0</td>
<td>37.6</td>
<td>T1/0.5s</td>
</tr>
<tr>
<td>74.0</td>
<td>4.0</td>
<td>25.3</td>
<td>T2/1.5</td>
</tr>
<tr>
<td>60.0</td>
<td>12.0</td>
<td>24.6</td>
<td>T2/1.5</td>
</tr>
<tr>
<td>50.0</td>
<td>7.0</td>
<td>24.0</td>
<td>T2/1.5</td>
</tr>
<tr>
<td>47.0</td>
<td>9.0</td>
<td>24.8</td>
<td>T2/1.5</td>
</tr>
<tr>
<td>47.0</td>
<td>12.0</td>
<td>25.8</td>
<td>T2/1.4</td>
</tr>
<tr>
<td>56.0</td>
<td>17.0</td>
<td>21.4</td>
<td>T2/1.4s</td>
</tr>
<tr>
<td>24.0</td>
<td>5.0</td>
<td>46.5</td>
<td>T2/1.4</td>
</tr>
<tr>
<td>17.0</td>
<td>4.5</td>
<td>37.0</td>
<td>T2/1.4</td>
</tr>
<tr>
<td>32.0</td>
<td>8.0</td>
<td>29.2</td>
<td>T2/1.6</td>
</tr>
<tr>
<td>35.0</td>
<td>12.0</td>
<td>35.4</td>
<td>T3/2.1-3.1</td>
</tr>
<tr>
<td>38.0</td>
<td>12.0</td>
<td>35.8</td>
<td>T3/2.1-3.1</td>
</tr>
<tr>
<td>68.0</td>
<td>10.0</td>
<td>22.4</td>
<td>T3/ Sub 3.1</td>
</tr>
<tr>
<td>50.0</td>
<td>7.0</td>
<td>22.6</td>
<td>T3/ Sub 3.1</td>
</tr>
<tr>
<td>53.0</td>
<td>9.0</td>
<td>23.1</td>
<td>T3/ Sub 3.1</td>
</tr>
<tr>
<td>26.5</td>
<td>5.0</td>
<td>24.4</td>
<td>T3/0.5</td>
</tr>
<tr>
<td>48.0</td>
<td>9.0</td>
<td>55.3</td>
<td>T3/1.5</td>
</tr>
<tr>
<td>17.0</td>
<td>8.0</td>
<td>52.9</td>
<td>T3/1.4</td>
</tr>
<tr>
<td>29.5</td>
<td>9.0</td>
<td>28.7</td>
<td>T3/1.2</td>
</tr>
</tbody>
</table>

Figure 4.18: Pilcon Shear Vane Undrained Shear Strengths.
individual lithologies was too small.

The undrained strengths obtained show a spread of peak values, but a small range of residual values. The T3/2.1-3.1 soil registered $C_u^{(peak)}=36$ kPa, and $C_u^{(residual)}=12$ kPa. When used in the same infinite slope calculation as the shear box parameters the $\phi'=0$ analysis shows that the $C_u^{(peak)}$ gives $F=2.9$, but the $C_u^{(residual)}$ value only $F=0.97$ (failure) for a totally saturated 3.1 m thick 15° slope. Back analysis by solving for $F=1$ with respect to cohesion produces $C_u=12.4$ kPa. This would be consistent with the observed creeping earthflow features on the slope, which indicate that marginal stability conditions exist on the natural slopes.

The analysis used is simple, but gives some perspective to the shear strength values obtained. More elaborate methods of slope analysis could be used for later design purposes prior to finalising the slope stabilisation programme.

4.4 Synthesis

The test programme completed as part of this study has shown:

1) The observed range of plasticity index for Panikau soils is from 18 to 50, with several soils having natural water contents greater than the liquid limit.

2) Hydrated lime mixed with the T3/2.1-3.1 soil produced a reduction in plasticity index. The plasticity was reduced from 50 to 24 with increasing weight percent lime. No reduction was observed between the 6% and 8% samples.

3) X-Ray diffraction scans of -425μm clays from the site indicate that smectite (montmorillonite), illite, lesser kaolinite, quartz, and minor calcite and chlorite are present in the sampled soils.

4) The Initial Lime Consumption of the T3/2.1-3.1 soil is 4%. Other soils tested gave values ranging from 1% to 3%.

5) Unconfined compressive strength (UCS) of Proctor moulded specimens manufactured at similar density and
moisture content, and increasing percentage hydrated lime for up to 28 days, showed increasing UCS with time and increased percent lime.

6] Attempts to manufacture lime slurry glazed specimens were unsuccessful, with difficulty experienced in producing stable 36 mm diameter specimens able to survive the curing period.

7] Shear box testing of natural T3/2.1-3.1 soil and the same material remoulded with 1.5% hydrated lime gave the following parameters. Natural soil: Peak $c'=12.5$ kPa, $\phi'=13.5^\circ$, Residual $c'=0$ kPa, $\phi'=18^\circ$. T3/2.1-3.1 + 1.5% hydrated lime: Peak $c'=8.5$ kPa, $\phi'=33^\circ$, Residual $c'=11.8$ kPa, $\phi'=14.5^\circ$.

8] Pilcon Hand Shear vane testing of soils exposed in trenching gave a range of peak undrained shear strength values (12-74 kPa), and low residual strengths (5-21 kPa).
Chapter Five: Summary and Conclusions.

5.1 Deep Lime Stabilisation
for Panikau Summit

5.1.1 LSPI Stabilisation

The Panikau Summit site has potential as a site for LSPI to be applied as a partial solution to existing stability problems. The failure observed at Panikau is inferred to be caused by the pre-existing creeping slope failure as evidenced by the earthflow features below the highway, and drainage and site preparation problems associated with the construction of the highway in its present location. The drainage from the eastern (upslope) side of the road would be the primary solution for this site. A cutoff subsoil drain to top of the blue grey highly sheared unit (or as deep as practically possible) would be required along upslope watertable of the highway for the 110 m length of the site to eliminate seepage beneath the existing pavement. The LSPI programme would be applied at 1.5 m centres from the upslope watertable to 5 m below the toe of the highway embankment (30 m width) on the western side (downslope), for a 110 m length, with an average depth of 3 m (9900 m³). The 3 m depth assumes that 3 m is the maximum application depth of a conventional mechanised LSPI rig. Conventional grouting equipment could reach greater depths, but would require predrilling in difficult ground conditions.

Such a treatment programme could not be said to be purely a LSPI solution, as a substantial degree of stabilisation would be derived from the subsoil drain installation. LSPI would establish relatively constant moisture content and greater shear strength in the treated clay subgrade materials.

The use of LSPI without a drainage installation in the
upslope watertable is seen as unwise. The LSPI would not allow for the dissipation of pore pressures on the upslope side of the highway, and consequently the pressures would accumulate. This could lead to the failure of the entire highway corridor, or the destabilisation of another area of the slope downhill of the LSPI stabilised area. Some method of drainage is necessary in addition to the LSPI application at Panikau Summit.

The ground conditions at Panikau Summit are generally favourable for LSPI (on the basis of the limited testing involved in this study), but areas with high organic content were observed in Trench 3, and "floaters" of hard silty sandstone do occur with soft silty clays which cold damage injection probes. Soils with high organic content are not stabilised to the degree non-organic soils are (Boynton and Blacklock, 1985). If LSPI was to be used, then some equipment would need to be developed, though for initial evaluation purposes a grouting contractor should be used.

5.1.2 Lime Column Stabilisation

The lime column method is the preferred method of deep stabilisation for Panikau Summit. The installation of lime columns would stabilise the subgrade to a greater depth than conventional LSPI, and establish deep drainage to the depth of installation. Lime columns would eliminate the need for a deep subsoil drain in the upslope (eastern) watertable, although a shallow (0.5 m) drain should still be installed to collect drainage from the columns installed and the road surface in the area. Lines of columns would be installed at 1 m-1.5 m centres along the upslope (eastern) shoulder of the highway for the 110 m length of the site to cut off seepage. The distribution of columns over the area to the east would be sufficient to allow uniform drainage settlement (if significant) and strengthening of the highway embankment. The estimated depth would be 8-10 m from existing penetrometer soundings and drilling, with depths confirmed by test column
installation at the site. The same field testing would also provide data to design the final layout of columns for the rest of the site. The total area of lime column treatment should be less than that envisaged for LSPI stabilisation.

Site stability would be enhanced by keying the columns into "solid ground" to provide shear resistance against downslope movement, but the limited degree of instability apparent at Panikau Summit make this less important. The definition of "solid ground" at Panikau is problematic, as almost all lithologies encountered have been sheared to some degree.

The lime column machinery envisaged in this study is an adaption of an existing rotary hydraulic tractor mounted drill rig operated by local drilling contractors in the Hawkes Bay (Bayliss Bros., Napier). The rotary hydraulic rigs have a good torque rating which would probably cope with hard silty sandstone "floaters" less than the diameter of the rotary head (0.5 m)(Bayliss Bros., pers comm). Once a mixing head was constructed, and equipment to pressurise the powdered quicklime in the existing drill water system adapted the method could be developed to evaluate the lime column concept in local conditions.

5.1.3 Synthesis

Based on a preliminary laboratory testing programme, the Panikau Summit site is a potential candidate for the application of deep lime stabilisation. The lime column method is favoured over LSPI because of the need to establish drainage within the subgrade at the site. Lime columns would require minimal associated drainage works in comparison with LSPI. LSPI does not allow for drainage in the same way that columns do, so a deep subsoil drain would be required in addition to the injection programme.

Both methods of stabilisation have merit, but by differ
in the mode of application and function once applied. The degree of refinement apparent in the lime column literature make it attractive from an engineering design perspective, where as the LSPI literature is less specific regarding design and application. This mainly reflects the differing attitudes in the development and promotion of the stabilisation methods in professional and academic literature. Despite this, the technical basis for lime columns does appear to be more advanced than for LSPI, based on the literature reviewed in this study.

In a similar embankment without the seepage problems present at Panikau Summit, the LSPI would be adequate to strengthen the subgrade lithologies and establish a stable foundation for the highway. The provided that the soils present in a foundation area can be proved to be lime responsive, then the particular field conditions and construction performance required can be weighed up, and the most suitable method of deep lime stabilisation selected.

5.2 Conclusions

5.2.1 Literature Review of Deep Lime Stabilisation

Lime Slurry Pressure Injection (LSPI) and the Lime Column method of deep lime stabilisation are attractive as potential construction and remedial options in the areas of the world where the technology required to implement them has been developed, i.e. the Southern States of the United States of America and Scandinavia respectively. The development of testing methods that accurately assess the degree of improvement possible with a particular technique is a very important area of on-going development with both techniques.

The Lime Column method has had a larger amount of technical information published and is impressive in the number of successful stabilisation programmes implemented. The ability the be able to design the columns in a similar
way to a conventional pile foundation will make its presentation to engineering staff as a concept relatively easy.

LSPI is well presented as a concept, but is lacking in the detailed technical appraisals that the lime column method has. This stems from the promotion of LSPI by the National Lime Association in the United States of America who reflect the interests of member contractors and lime producers. The literature is missing the details of decisions made to establish the degree and type of LSPI or L/FASPI most appropriate to a particular site. Despite this omission, the technique is attractive by virtue of its simple application method and reported relatively low cost against more conventional stabilisation programmes.

5.2.2 Site Investigation

The engineering geological site investigation of the Panikau Summit site indicates that the area has a 2 m–3.5 m thickness of moist to saturated clayey silts overlying a shear surface on top of a unit containing highly deformed bluish grey silt with hard greenish grey siltstone boulders. The ground surface is uneven and swampy, and visible earthflow features occupy much of the slope. The sequence observed in excavations is broadly layered, and water seepage observed tends to occur between layers. No evidence of possible vertical seepage paths was observed with the exception of the unit in Trench 2 with polished fracture surfaces at a depth of about 1 m. Lithologies are commonly clay rich and relatively impermeable. The slope movement recorded from surface peg stations was relatively small close to the State Highway, with all displacement vectors in the down slope direction. This evidence from a short survey period is consistent with a slow creeping slope failure. The slope morphology at the site is very common in the hill country around Panikau.
The initial failure in 1970 of the Panikau Road intersection was influenced by additional factors, particularly the drainage from the subsoil drain on the upslope side of the State Highway seeping through the subgrade to the western (downslope) side of the road. The magnitude of the seepage can not be defined, but the failure of the newly constructed fill embankment suggests that destabilising forces present at the time of failure were significant. The failure continued in the same area in 1971 before Panikau Road was finally relocated in its present position. The western (downslope) side of State Highway 35 has been subject to on-going subsidence since 1971. Arcuate depressions have formed on a regular basis in the north-bound lane, and a block on the edge of the embankment has also failed. Regular maintenance has filled and sealed these depressions, but the features reoccur within 12 to 18 months.

The application of a deep stabilisation method to this site should limit the on-going deformation at the site so that a stable road configuration could be produced, significantly reducing the maintenance costs for the site.

5.2.3. Laboratory Investigations

The test programme completed as part of this study has shown:

1] The observed range of plasticity index for Panikau soils is from 18 to 50, with several soils having natural water contents greater than the liquid limit.

2] Hydrated lime mixed with the T3/2.1-3.1 soil produced a reduction in plasticity index. The plasticity was reduced from 50 to 24 with increasing weight percent lime. No reduction was observed between the 6% and 8% samples.

3] X-Ray diffraction scans of -425μm clays from the site indicate that smectite (montmorillonite), illite, lesser kaolinite, quartz, and minor calcite and chlorite are present in the sampled soils.
4] The Initial Lime Consumption of the T3/2.1-3.1 soil is 4%. Other soils tested gave values ranging from 1% to 3%.

5] Unconfined compressive strength (UCS) of Proctor moulded specimens manufactured at similar density and moisture content, and increasing percentage hydrated lime for up to 28 days, showed increasing UCS with time and increased percent lime.

6] Attempts to manufacture lime slurry glazed specimens were unsuccessful, with difficulty experienced in producing stable 36 mm diameter specimens able to survive the curing period.

7] Shear box testing of natural T3/2.1-3.1 soil and the same material remoulded with 1.5% hydrated lime gave the following parameters. Natural soil: Peak $c'=12.5$ kPa, $\phi'=13.5^\circ$, Residual $c'=0$ kPa, $\phi'=18^\circ$. T3/2.1-3.1 + 1.5% hydrated lime: Peak $c'=8.5$ kPa, $\phi'=33^\circ$, Residual $c'=11.8$ kPa, $\phi'=14.5^\circ$.

8] Pilcon Hand Shear vane testing of soils exposed in trenching gave a range of peak undrained shear strength values (12-74 kPa), and low residual strengths (5-21 kPa).

5.3 Summary

The investigation of Lime Slurry Pressure Injection (LSPI) and the Lime Column method show that low strength clay rich soils can be successfully stabilised without the requirement for large scale earthworks on sites such as Panikau Summit. Each site needs to be engineering geologically investigated to determine the nature and causes of failure, and then a stabilisation option selected to best solve the problem. As many subsidence and drop-out failures are related in some way to the presence of water in the slope, it is very important to identify the source and seasonal nature of prevailing piezometric conditions prior to deciding on the method of stabilisation to be adopted. This site data gives geotechnical personnel the technical basis to select the most beneficial deep stabilisation method.
The Lime Column method is suited to any site, including those where drainage is required in addition to stabilisation. The method is very flexible in terms of design, with the number of columns able to be increased or decreased as required, and the depth of treatment only limited by the capability of the available machinery. Specific loadings and settlement requirements can be designed for using the methods detailed in Broms (1984). The method will be conceptually appealing to engineers in its similarity to conventional piles. The method has good potential for use in New Zealand conditions if equipment can be successfully developed. More research on the permeability developed in lime columns and confirmation of design relationships from Broms (1984) for laboratory testing in New Zealand conditions is required.

The Lime Slurry Pressure Injection (LSPI) method has been applied extensively in the southern United States to sites where foundation soils require strengthening. The normal depth of application is about 3 m, though depths of 12 m have been achieved with special equipment in favourable conditions. LSPI has generally been used as a foundation improvement or remedial technique with many reported successful applications in the United States. LSPI evaluation prior to application is not as well documented as the lime column method, and the ability to predict the magnitude of strength gain from laboratory testing still requires development.

For application in New Zealand conditions, the methods of identifying particular LSPI treatments for given site conditions needs more technical information than the present literature provides. This would involve either contacting roading authorities and/or specialist LSPI firms in the USA on a fact-finding basis prior to the use of LSPI in New Zealand, or starting from scratch under local conditions.

5.3 Recommendations for Further Work
1] The laboratory based confirmation of Lime Column design methods detailed in Broms (1984) in terms of New Zealand soils and ground conditions.

2] A laboratory based programme to determine the changes in permeability associated with the construction of lime columns.

3] Development and field trialing of the lime column method with particular emphasis on adapting existing rotary drilling machinery.

4] Further investigation of the LSPI method by direct formal contact with roading authorities and LSPI contractors in the United States.
Acknowledgements

I wish to thank the following people for their assistance and support through the preparation of this thesis:

Transit New Zealand, Napier Office for their financial support and positive attitude to the study in difficult economic times.

David Bell for his initiation of the study, supervision and editing skills.

Works Consultancy Services (Ltd), Napier and Gisborne, for their logistical and technical support.

Bob McKelvey for his support and interest in the study, the use of his vehicle, and for the experience gained during thesis, and Chris Clarke for his survey skills and good sense of humour.

Arthur, Stuart, Larry, Nick, Mark, and Mel at the Napier Laboratory for technical help, and a great atmosphere to work in.

Greame Hawken, Stu Whiterod and the survey section for managing to do the surveys between other jobs, professional expertise and advice on the survey set up, plan plotting after 4pm, and general good company.

Mike Curry in Gisborne for remembering to stop and read the piezometers, and for organising the excavator.

Stephen and Catherine Brown for XRD preparation, grain size analysis, and especially computer expertise at critical times.

The University computer centre staff for helping when it was critical, and especially for the DeskJet that extra night.

To the people in Orem, Utah that made such an excellent WP program, and HP for an excellent printer.

Mike Marden for his interest and discussions on LSPI and landslide problems generally, the memorable trip to Tarndale, and warm hospitality at Mangatu.

Rose Fitzgerald for the transport and the loan of her car at the death, and Viv Smith and Todd for reading Chapter 4.

Gisborne District Council for rainfall data.

Simon Nathan and NZ Geological Survey Regional Mapping
For the use of 1:50000 field sheets of the Whangara Area.
Trevor, Sally, Steve, Stu, Bombs and Black for being a
home away from home.
Andrew and Delyth Taylor for putting me up in Napier for
ages, their excellent company, humour and friendship.
Neil and Wendy Taylor, in Napier for their hospitality,
and good taste in wine.
The Summertons in Gisborne for hospitality at the drop
of a hat, and great steaks Dave!.
Finally my long suffering, loving wife Zandra, who has
moved twice, spent many weeks keeping the home fires burning
while I was away in the field, been a computer widow for
months, but who made the course possible by providing a
stable, financial and loving base for two and a half years
without complaint. Thank you, part of this is yours.
References


APPENDIX 1: Rainfall Data

All data was obtained from Gisborne District Council as it became available through the New Zealand Meteorological Service. The record from February 1991 was unavailable as of June 1991. All graphs and data tables were produced with Quattro!

1985–1991 Rainfall NZMetServStn D88421

1968–1991 Rainfall NZMetServStn D88421
### Monthly Rainfall Data

#### N.Z. Met. Service Rainfall Station 88421

**Mokairau (to 7/69) / Pakarar (from 8/69)**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>1</td>
<td>334</td>
<td>450</td>
<td>338</td>
<td>260</td>
<td>332</td>
<td>1578</td>
<td>1773</td>
<td>1317</td>
<td>1303</td>
<td>442</td>
<td></td>
</tr>
<tr>
<td>Feb</td>
<td>2</td>
<td>484</td>
<td>871</td>
<td>466</td>
<td>1334</td>
<td>562</td>
<td>477</td>
<td>1099</td>
<td>1665</td>
<td>251</td>
<td>899</td>
<td></td>
</tr>
<tr>
<td>Mar</td>
<td>3</td>
<td>72</td>
<td>606</td>
<td>485</td>
<td>452</td>
<td>2227</td>
<td>1466</td>
<td>519</td>
<td>991</td>
<td>247</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Apr</td>
<td>4</td>
<td>652</td>
<td>499</td>
<td>419</td>
<td>997</td>
<td>1364</td>
<td>2146</td>
<td>909</td>
<td>2333</td>
<td>1557</td>
<td>1503</td>
<td></td>
</tr>
<tr>
<td>May</td>
<td>5</td>
<td>439</td>
<td>497</td>
<td>874</td>
<td>1608</td>
<td>1686</td>
<td>363</td>
<td>630</td>
<td>1643</td>
<td>434</td>
<td>975</td>
<td>984</td>
</tr>
<tr>
<td>June</td>
<td>6</td>
<td>1099</td>
<td>236</td>
<td>555</td>
<td>203</td>
<td>1565</td>
<td>3589</td>
<td>1652</td>
<td>896</td>
<td>3890</td>
<td>2171</td>
<td></td>
</tr>
<tr>
<td>July</td>
<td>7</td>
<td>837</td>
<td>510</td>
<td>316</td>
<td>3066</td>
<td>1258</td>
<td>1503</td>
<td>1014</td>
<td>760</td>
<td>1871</td>
<td>2251</td>
<td>2129</td>
</tr>
<tr>
<td>Aug</td>
<td>8</td>
<td>566</td>
<td>322</td>
<td>927</td>
<td>3507</td>
<td>760</td>
<td>3021</td>
<td>2354</td>
<td>1056</td>
<td>1796</td>
<td>959</td>
<td></td>
</tr>
<tr>
<td>Sept</td>
<td>9</td>
<td>215</td>
<td>263</td>
<td>234</td>
<td>1074</td>
<td>256</td>
<td>1769</td>
<td>1789</td>
<td>1615</td>
<td>1744</td>
<td>1911</td>
<td>1218</td>
</tr>
<tr>
<td>Oct</td>
<td>10</td>
<td>372</td>
<td>381</td>
<td>94</td>
<td>1182</td>
<td>954</td>
<td>933</td>
<td>1406</td>
<td>820</td>
<td>1253</td>
<td>644</td>
<td>1158</td>
</tr>
<tr>
<td>Nov</td>
<td>11</td>
<td>161</td>
<td>737</td>
<td>779</td>
<td>850</td>
<td>477</td>
<td>152</td>
<td>877</td>
<td>1053</td>
<td>614</td>
<td>179</td>
<td></td>
</tr>
<tr>
<td>Dec</td>
<td>12</td>
<td>354</td>
<td>376</td>
<td>263</td>
<td>401</td>
<td>740</td>
<td>836</td>
<td>1850</td>
<td>1053</td>
<td>412</td>
<td>348</td>
<td></td>
</tr>
</tbody>
</table>

**Monthly Average**

522 393 510 1176 1029 1015 1517 1293 1328 1345 1020

**Recorded Total**

4695 4711 5609 14106 10287 10152 18198 15520 15934 14799 12237

---

### Monthly Rainfall Data

#### N.Z. Met. Service Rainfall Station 88421

**Mokairau (to 7/69) / Pakarar (from 8/69)**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>390</td>
<td>531</td>
<td>491</td>
<td>175</td>
<td>931</td>
<td>296</td>
<td>131</td>
<td>50</td>
<td>1610</td>
<td>2074</td>
<td>510</td>
<td>729</td>
<td>913</td>
<td>1288</td>
<td></td>
</tr>
<tr>
<td>1118</td>
<td>795</td>
<td>1492</td>
<td>1753</td>
<td>77</td>
<td>1268</td>
<td>631</td>
<td>1281</td>
<td>735</td>
<td>2249</td>
<td>323</td>
<td>246</td>
<td>1281</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3815</td>
<td>2274</td>
<td>995</td>
<td>666</td>
<td>182</td>
<td>1255</td>
<td>2266</td>
<td>1151</td>
<td>2502</td>
<td>3318</td>
<td>244</td>
<td>1313</td>
<td>1281</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1131</td>
<td>1664</td>
<td>3232</td>
<td>1828</td>
<td>1133</td>
<td>1571</td>
<td>244</td>
<td>1607</td>
<td>211</td>
<td>617</td>
<td>913</td>
<td>1288</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1351</td>
<td>613</td>
<td>1829</td>
<td>683</td>
<td>425</td>
<td>1384</td>
<td>2050</td>
<td>1209</td>
<td>433</td>
<td>1081</td>
<td>3509</td>
<td>151</td>
<td>1080</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1210</td>
<td>3396</td>
<td>1963</td>
<td>1428</td>
<td>2128</td>
<td>1903</td>
<td>2040</td>
<td>1251</td>
<td>1112</td>
<td>1777</td>
<td>1016</td>
<td>2047</td>
<td>1688</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1291</td>
<td>920</td>
<td>2307</td>
<td>1218</td>
<td>1479</td>
<td>1128</td>
<td>2116</td>
<td>1681</td>
<td>2640</td>
<td>2664</td>
<td>679</td>
<td>1854</td>
<td>1543</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1751</td>
<td>1900</td>
<td>2415</td>
<td>374</td>
<td>1092</td>
<td>1212</td>
<td>564</td>
<td>1299</td>
<td>486</td>
<td>838</td>
<td>2894</td>
<td>2312</td>
<td>1473</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1004</td>
<td>831</td>
<td>339</td>
<td>1253</td>
<td>272</td>
<td>2852</td>
<td>703</td>
<td>2284</td>
<td>566</td>
<td>944</td>
<td>2060</td>
<td>1713</td>
<td>1170</td>
<td></td>
<td></td>
</tr>
<tr>
<td>951</td>
<td>614</td>
<td>1385</td>
<td>962</td>
<td>1079</td>
<td>583</td>
<td>850</td>
<td>688</td>
<td>333</td>
<td>100</td>
<td>625</td>
<td>795</td>
<td>790</td>
<td></td>
<td></td>
</tr>
<tr>
<td>540</td>
<td>521</td>
<td>1069</td>
<td>476</td>
<td>545</td>
<td>757</td>
<td>466</td>
<td>459</td>
<td>2379</td>
<td>386</td>
<td>1650</td>
<td>661</td>
<td>718</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1778</td>
<td>2534</td>
<td>85</td>
<td>146</td>
<td>794</td>
<td>410</td>
<td>182</td>
<td>1135</td>
<td>686</td>
<td>831</td>
<td>431</td>
<td>745</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**1968-91 Monthly Average**: 729 913 1288 1281 1080 1688 1543 1473 1170 790 718 745

**15199 16060 15547 12682 10076 14816 13735 12813 11922 17405 15285 12140 12779**
Appendix 2: Penetrometer Data and Interpretation

A2.1: 1972 Dutch Cone Penetrometer Investigation

Locations are shown on PW Gisborne 7502, Appendix 6.

DS1; RL 132.95 m: Located 13 m upslope of the highway seal edge (PW Gisborne 7502, Appendix 6). Point resistance increases to 7.5-8 MPa at 5 m to 6 m, and declines to 1.8-2 MPa at 8 m. Three peaks and matching troughs occur between 8 m and 10 m, and a weak zone at 10.8 m to 12 m. From 12 m to 16 m is a uniformly variable zone of 3.5-6 MPa, with a significant drop to < 1 Mpa from 16 m to 18 m. Below this point resistance increases to 6-20 MPa. A recorded water level of 0.6 m is indicated on the plot.

This profile is interpreted as soft clay and silt material to 4.5-5 m, with chaotic sheared sandstone blocks and firm muddy siltstone as observed in excavated trenches from 5-12 m. From 12-16 m the marked increase in point resistance suggests a more coherent sandstone lithology is inferred above a low strength horizon between 16.5-17.6 m. This feature is a significant zone of weakness within the profile, but is not thought to be the main control on the surface deformation observed on the slope. This cannot be ruled out absolutely, but the deformation observed within the upper 5 m of the soil profile is sufficient to explain the embankment failures that have occurred. Below the 19m level high point resistances indicate strong soils (weak rock).
PENETRIMETER SOUNDING, NEW Zealand

LOCATION OF WORK: Panikai Hill; slip area
SOUNDING NO. 1

Foundation Investigation S.M. 35. Gilgamon
F.T. 277
JUN 72

Average
Friction
Shear Strength

0 50 100 150 200 250 300
Tons/sq.ft.

Water table at 20'

NOTE: - Unit Friction (Tons/sq.ft.) - G.R. (Cone + adhesion tube) - G.R. Cone

R.W. 570(7/66)
DS2; RL 130.36 m: Located 9 m downslope of the lower highway seal edge, and 5 m downslope of the headscarp of the 1970-71 failure (PW Gisborne 7502, Appendix 6) This sounding has many high point resistance values in a series of generally increasing values with depth. Peak values above the reported watertable occur at 0.8 m, 1.6 m, 4 m and 4.6 m, with low point resistance zones at 1-1.4 m, 3 m, and 4.2 m. A zone of low point resistance is coincident with the reported waterlevel at 4.8 m down to 8 m, and from 11.5-13 m.

This penetrometer is thought to be close to the site of Borehole 1 described in section 3.4.3, and the interpretation is based on core recovered and drilling conditions from the borehole. The top 2 m are probably the basecourse and subgrade materials, with firm clayey silts intermixed with cemented siltstone aggregate material between 2 and 5 m. This zone also contained highly polished zones on sheared siltstone at the 4-5 m level. The lower point resistance zone associated with the water table coincided with a zone of poor core recovery and caving in the borehole. Highly shattered siltstone and blue grey silts were recovered in this depth range. From approximately 6.5 m downward the core recovered indicated sections of relatively intact cemented sandstone to highly sheared shattered sandstone and blue grey silts.
DS3; RL 125.79 m: This sounding (PW Gisborne 7502, Appendix 6) is only 3 m to refusal with a peak 1.2 m, and no recorded water level. This penetrometer is coincident with Trench 1, and represents 3 m of heavy clayey silts above the blue grey sheared clay and sandstone unit. Most slope movement is inferred to occur above the blue grey clay/sandstone unit. The refusal peak was probably on one of the larger blocks of sandstone in that unit.
NOTE: Unit friction (Tons/sq.ft) = G.R. (Cone + adhesion tube) - G.R. Cone

P.W. 570(7/66)
DS4; RL 123.51 m: Point resistance to 4.2 m is low, followed by a marked increase to refusal at 7 m. The reported water level was at 2.1 m. This profile is interpreted as a slightly thicker sequence of heavy clayey silts (0-4 m) overlying the same blue grey silt/sandstone unit observed in Trench 1.
NOTE: - Unit friction (Tons/sq.ft) = G.R. (Cone + adhesion tube) - G.R. Cone

P.W. 570 (7/66)
DS5: RL 119.24 m: Low point resistance values from 0-2 m are followed by a steadily increasing series of high and low values to refusal at 7 m. Low resistance troughs occur at 2.2-2.6 m, 3.4 m, 4 m, 5.2 m, and 6 m. No water level was reported. Again this profile is thought to be a 2m thickness of clayey silt overlying the blue grey silt/sandstone unit.
NOTE: - Unit friction (Tons/sq.ft), G.R. (Cone + adhesion tube) - G.R. Cone

P.W. 570(7/66)
A2.2: 1972 Dutch Cone Penetrometer Investigation

The location of this set of penetrometer is uncertain, as location pegs were not surveyed before maintainence operations obliterated the marks. Peg Pene 1 is thought to be PS#1, and the PN series is thought to be 20 m-30m north of that point.

**PN#1:** The sounding is located in the upslope watertable. The plot shows that coincident low point resistance and friction values are associated with areas immediately above marked peak values of point resistance. This is probably related to the alteration of material associated with perched water above the harder horizon which tends to be hard siltstone or sandstone, or shearing along the change in material competence. The initial point resistance to 1 m is inferred to be road basecourse and subgrade, which is underlain by 2m of weak material. Weak zones plot at 3 m, 3.4 m, 5.4 m, 6 m, 8.6 m, 9.6 m, 11.4 m and 12.4m. Point resistance peaks occur at 3.2 m, 3.8 m, 5.6 m, 8.8 m, 11.8 m and at the 12.6 m refusal.

**PN#2:** This sounding is located 11 m west of PN#1 in the downslope watertable adjacent to the edge of seal. This penetrometer is thought to be close to the borehole at 709954N,329243E. Coincident low point bearing and friction values occur at 2 m, 3.8 m, 4.6 m, 5.8 m, and 9.2 m. Point resistance and friction peaks are less distinct than PN#1, although the general trends are still comparable. Refusal occurs at 9.4 m on a sandstone horizon (from the borehole log). Below this the plot is inferred to be similar to PN#1.
PS#1: This sounding is located in the upslope (eastern) watertable 7 m from the centreline where PS#3 was put down. Both plots have low point resistance low to 4 m where there is a small peak and drop again. At 5 m friction values increase, and the point resistance plot is jagged with an increasing trend. There is a slight drop off in both plots between 10.8 m and 11.5 m, and at 15.2 m. This sounding correlates well with the DS1 sounding.

PS#2: This sounding is located in the downslope (western) watertable adjacent to the edge of seal, 6 m west of the centreline. Relatively high point resistance and variable friction from 0 m to 2.5 m is the basecourse thickness on the west of the highway where the subsidence has been 'topped up' in the past. Coincident low values occur at 4.8 m, 5.8 m, 9 m, 12.6 m and 15 m. The 4.8 m to 5 m weak zone above a strong peak at 5 m to 5.2 m has very low friction values, and is inferred to be significant in terms of overall slope stability. Point resistance values are significantly lower in this sounding (cf PS#1) in the 5 m to 14.5 m range, with friction increasing from about 9 m.

PS#3: This sounding is located at the centreline of State Highway 35 between the two previous soundings. The initial peak values are caused by the basecourse layer. Below this point resistance is very low to 8 m to 9 m where two peaks occur, with relatively low values below that to 13.5 m. Refusal is at 14 m. Friction values are low to 6.2 m, with a general increase in values below this. Coincident low values occur at 3 m, 5 m, 6.2 m, 7.5m to 8.2 m, 9.8 m, 10.4 m and 13 m.
A2.3: Scala Penetrometer Soundings

Scala locations are generally close to survey pegs.

The penetration plots have two X-axes: Blows/50 mm and bearing capacity ($q_\text{s}$). The first is actual recorded data values, while the $q_\text{s}$ values are approximations of bearing capacity from Stockwell (1977). The tests were extremely difficult to perform in these conditions as the soft clays and silts created large amounts of 'suction' or rod friction, making rods almost impossible to remove manually for the depths shown. The Scala profiles exhibit extremely low point resistance values from ground level to at least 1.5 m and usually 2.5 m. Below this the value increase to refusal. Rod friction was significant in these materials, so the actual penetration values are probably of limited meaning. The depth of major penetration resistance changes are shown, and compares well with depths to the zone of shearing observed in trenching and other cone penetrometer testing.
Appendix 3: Drilling and Trenching

A3.1: Drilling Programme

A Works Consultancy Services (Napier) Jacro trailer mounted wireline rotary drill rig used NQ (75 mmØ) rods with a barium bit and split tube core barrel in HQ casing were used with water as the drilling fluid. Drilling was performed over 3 days on site in June 1990, with logging completed subsequently at Works Napier Engineering Laboratory. The borehole is located between pegs A1 and A2 close to the backslope west of the State Highway (Figure 3.6).

The brief to the drillers was to obtain as much core as possible to enable correlation with the available dutch cone penetrometer results. Undisturbed thin tube sampling and in-situ shear vane tests were specified in the original borehole brief at 4.5 m based on estimated soft clay zones from adjacent penetrometer plots, but practical problems were encountered with the presence of adjacent soft and hard lithologies. Thin hard layers in soft clay and silt required regular adjustment of the core catcher in the drill bit to stop the core catcher jamming in and/or destroying soft materials. The increased thrust required to drill through the harder layers resulted in some softer materials immediately below being destroyed when the bit broke through the hard material. Significant loss of core occurred in the drilling process. Caving ground conditions were also encountered between 5 m and 6.15 m, and from 11 m to the end of the borehole (EOD). Difficult drilling conditions lead to the decision to end the borehole at 13 m.

Two standpipe piezometers were installed in the borehole. Piezometer 1 covers the 5.5 m to 11.8 m depth range, and Piezometer 2 the 2.3 m to 5 m depth range. Recorded waterlevels indicate that the two piezometers may be within the same hydraulic regime, as levels are similar in level and variability. Both broadly mirror the trends in recorded rainfall, but readings are too sparse to estimate a response time to individual rainfall events.
Panikau Borehole

DESCRIPTION

Basecourse layer Tri-coned, No Core to 0.8m

Core lost seating Collar casing 0.8-125m

SILT: clayey, whitish brown, weathered, soft-firm, with mst chips, int. plast.

SILT: clayey, mottled creamy brown, hi. plast.

SILT: firm, clayey with pockets of granular mst. Organic Horizon (?), 3.25m

SILT: highly sheared v. contains broken mst and siltstone cobbles. Polished surfaces. 3.65m Redox Boundary

SHEAR ZONE? 3.95m Polished black mst

4.10m Polished black mst

SILT + SILTSTONE: shearing common

Caving Hole in alternating silt and siltstone

Grey SILTSTONE washings

Total Loss of Circulation

Clay: silty, firm-v stiff, mixed, plastic

Sandstone: hard, strong, grey, Polished

SILT: clayey, sandy, moist, fractures slightly plastic. No circulation loss.

Sandstone: fine, hard, strong, closely fractured (highly polished at 8.2, 8.35m)

Rapid Descent


No circulation loss.

Very disturbed; blocks of sandstone in silt matrix. Shearing extensive

Clay: moist, plastic, soft, red-brown, (6.69)

Washing pink water with silt chips.

SILT: v. contorted, red-green, stiff, moist.

MUDSTONE: sty, hard, highly fractured

CLAY: silty, red-brown and green, moist to wet. Jamming core cadles.

Alternating soft silt and hard sandstone.
Borehole Piezometer Installation

Piezometers are housed in a recessed 'Toby' box.
A3.2 Borehole Lithologies

The borehole log indicates an initial thickness of "Panikau red metal" basecourse overlying a moist firm tan brown clayey silt subgrade with intermixed siltstone aggregate to 2 m. Below this level to 3.3 m the moist to wet, soft to firm, clayey silt materials are similar but with a lot of colour variation (pinkish brown, yellowish green, mottled creamy brown) and the same occasional aggregate of hard mudstone and siltstone. The material is contorted as might be expected in a compacted fill, but not sheared as sub 3.3 m lithologies are. This is inferred to be the fill material placed in the original realignment.

A thin layer of organic material at 3.25 m marks a change in lithology to a highly sheared light brown to greenish grey firm to soft clayey silt with chips of hard silty sandstone from 5 mm to >30 mm. Drilling fluid circulation improved below 3.5 m to 6 m. The sequence is not recognisable as a sedimentary because of the level of shearing that has occurred. The observed deformation is an artifact of shearing in geological conditions not active at the site today. Siltstone and sandstone material has been very highly polished by shearing, and the softer clays and silts have been smeared around them. The intensity of shearing is variable, but commonly most intense around blocks of hard competent siltstone and sandstone. The less competent clays and silts are plastic, firm to very stiff and commonly moist. Moisture content variation on core recovered is shown in Figure 3.34. These lithologies could best be described as a broken formation, or even melange if the competent sandstone and siltstone were not originally interbedded. The deformation is typical of lower Tertiary units (Ridd, 1967) adjacent to uplifted Whangai Formation in the Whangara area.

As discussed previously the rapid changes in competence caused significant core loss and jamming of the core catcher in these materials. A caving zone between 5 m and 6.15 m with a 20% circulation loss and numerous blackened shear zones, and lost core from 6.15 m to 6.45 with complete loss of circulation indicate a weakened zone. This depth range is represented by a low point resistance zone in penetrometer
sounding DS#2. Based on a 3 m fill and about 2.5 m to the shear zone in Trench 2, this could be the failure zone. Below this area the circulation is 100% to the EOB, and moisture contents are markedly lower. The slope failure zone is inferred to be in the 4.9 m-5.3 m depth range. A 1 cm thick soft, highly plastic, blackish grey gouge material was logged at 5.2 m.

The rest of the borehole correlates well with the DS#2 penetrometer sounding, with hard fractured cemented sandstone and siltstone corresponding to major point resistance peaks and zones of sheared dark greenish grey clay and silts to troughs. The sequence alternates between the hard and firm to very stiff materials. The degree of shear deformation is high in all materials, but not as intense as in the 4.9 m-5.3 m range where black gouge-like sheared clays occur.

Undisturbed thin tube sampling and in-situ shear vane tests were specified in the original borehole brief at 4.5 m based on estimated soft clay zones from adjacent penetrometer plots, but practical problems were encountered with the presence of adjacent soft and hard lithologies. Thin hard layers in soft clay and silt required regular adjustment of the core catcher in the drill bit to stop the core catcher jamming in and/or destroying soft materials. The increased thrust required to drill through the harder layers resulted in some softer materials immediately below being destroyed when the bit broke through the hard material. Significant loss of core occurred in the drilling process. Caving ground conditions were also encountered between 5 m and 6.15 m, and from 11 m to the end of the borehole (EOB).

Two standpipe piezometers were installed in the borehole (Figure 3.32). Piezometer 1 covers the 5.5 m to 11.8 m depth range, and Piezometer 2 the 2.3 m to 5 m depth range. Recorded waterlevels indicate that the two piezometers may be within the same hydraulic regime, as levels are similar in level and variability. Both broadly mirror the trends in recorded rainfall, but readings are too sparse to estimate a response time to individual rainfall events.
A3.3 Trench excavations

The following diagrams show the features within each trench.
Trench 3

- Topsol and Buried Organic Rich Horizon
- Gravel with clay matrix
- Clay
- Silt
- Sandstone
- Highly Sheared Silt

Vert=Horz

m

0 1
Appendix 4: Survey

A4.1: Survey Network Setup

The object of setting out a survey network was to attempt to establish a database for surface movement on the slope. From this an indication of changes and seasonal trends in surface movement could be observed, and related to subsidence problems effecting State Highway 35.

The site faces west at 12°-15° and has very few topographic high points suitable for locating survey network control points. Further to this, it proved to be difficult to initially establish the difference between "stable" and mobile areas across the hillside. The stations shown on Figure 1.3 were located based on field mapping and aerial photograph interpretation, keeping in mind practicality and maximisation of sightable peg shots able to be made from each control point. The 3 traverse control points (A2, CP1, Peg Base) were then traverse surveyed relative to a temporary local datum using a Wild T1000 theodolite and EDM. Trig 9831 was unable to be sighted from the original setup for the lack of suitable scrub cutting equipment. This was rectified in the following survey by the Napier WORKS Consultancy surveyors who have performed all subsequent surveys of the network using a Sokkisia Theodolite and electronic notebook. Tanilised 350 mm painted and tagged wooden pegs were used for the 23 stations set out across the slope. Most pegs have a galvanised nail sticking out of the top to sight on. Some allowance for peg losses were made in order to ensure adequate future coverage. The area immediately below State Highway 35 has the greatest peg density (Figure 1.3) so as to detect and surface movement that could be related to deformation observed on the highway.

A4.2: Survey Method

Each survey involved a traverse survey between the 3 control points (A2, CP1, Peg Base), and the tying in of each one to Trig 9831. A bearing was then taken from each control
point to every sightable peg station, so that each station has a bearing from at least two control points. The SERVIS survey computer package was used to calculate the intersection of the two bearings and the position recorded for each peg. The original concept was to use EDM measurements with a bearing to each peg station, but discussion with professional surveyors indicated that much greater accuracy could be obtained using the two bearing method. EDM measurements have errors associated with sighting to a staff and prism. The two bearing method locates the nail on top of each peg directly, reducing the error to sighting on the nail and the reading of the bearing. The surveyors were confident of ±2 cm accuracy using their equipment, i.e. that recorded movement greater than 2 cm was real and not an artifact of the survey method. The same WORKS survey team have done all but the initial survey, so few problems with survey operator errors or pegs not able to be located have occurred.

A4.3 Vector Movement

The data below shows the magnitude (m) and direction of peg stations between each survey. Total vector movement over the entire survey period is shown in the second table.
Appendix 5: X-Ray Diffraction Data

A5.1 X-Ray Equipment

This study used the facilities available at the University of Canterbury Geology Department. Scans were generated using:

- Philips PW 1729 X-Ray Generator (Ni shielded CuKα)
- Philips PW 1710 Diffractometer Control
- Philips PM 8203A One Line Recorder

Original plots were scanned at 1 degree 2 Theta/cm, then traced and reduced to the size presented below.

A5.2 Scan Interpretation

Clay minerals were identified by comparing the natural, 24 hour glycolated and 1 hour 550° scans for the same sample. Reference material was sourced from Esvlinger and Pevear (1988) and Fergusson (1985) to confirm interpretations.

**CMB034 [T1/1.8]**: The plot has a broad smectite peak at 15Å which shifts to 18Å on glycolation, strong kaolin peaks at 7.13Å and 3.58Å which disappear after heating, and low but sharp illite peaks at 10Å and 5Å which appear in all 3 traces. The glycolated trace has a "shoulder" on the right side of the 10Å illite peak indicative of interstratified illite-smectite in addition to pure 10Å illite. The diffuse lump at 13.8Å on the 550°C trace is possibly chlorite. This material has poor smectite peaks but good kaolinite and illite peaks.

**CMB035 [T1/3-3.1]**: This sample is similar to CMB034 except that the smectite peaks are sharper and of greater intensity, and the kaolinite 7.13Å and 3.58Å peaks are smaller. The illite 10Å and 5Å peaks are the same, but the interstratified illite-smectite "shoulder" right of the glycolated 10Å peak is more distinct. Estimates of the degree of interstratification using Srodon (1980) give 40% smectite layering. This sample is an illite-smectite clay, with discrete illite and some kaolinite.

**CMB038 [T2/2]**: The plots show large, sharp smectite peaks, and well developed illite and kaolinite peaks. The
interstratified illite-smectite "shoulder" is present right of the glycolated 10Å illite peak, and the estimated percent smectite interlayering is about 40%. The 550°C trace has a small 13.8Å chlorite peak that was obscured in the by the large smectite peaks prior to heating. This is an interlayered illite-smectite clay, with minor discrete illite, kaolinite and chlorite.

**CMB041 [T3/-3.1]:** The smectite peaks are small and sometime diffuse, but the illite and kaolinite peaks are sharp and well defined. The 10Å illite peak interstratification "shoulder" is very small, and the 13.8Å chlorite peak is present. This material is illite dominated with lesser smectite and kaolinite, and minor chlorite.

**CMB040 [T3/2.1-3.1]:** This is the material used in much of the testing later in the chapter. The traces show the largest smectite peaks in the series, and diffuse "shoulders" on most of the illite peaks. The percent interlayered smectite is estimated at 40-50%, and the peak shift on glycolation is the normal 15Å to 18Å shift. Kaolinite and discrete illite peaks are sharp but not large, and no chlorite was observed. The sample is a smectite dominated clay with interstratified illite-smectite, and lesser illite and kaolinite. In the field this material is described as a very dense, stiff to hard, moist, creamy white, silty clay with brown and blue grey flecks.
Appendix 6: Design Drawings, Panikau Hill Reconstruction.

List of Drawings:

1] P.W. Gisborne 7034/2
"Panikau Hill Reconstruction", Longitudinal Section.
The Panikau Summit area is at Centreline footages 5800-6150. Details of the original road and drainage also shown.

2] P.W. Gisborne 7034/1
"Panikau Hill Reconstruction", Plan.
This plan shows the position of the pre-1969 Panikau Road, the new S.H. 35 alignment, and the subsoil drain ending at the new intersection.

3] P.W. Gisborne 7034/1
"Panikau Hill Reconstruction", Cross sections 5100'-6500'.

4] P.W. Gisborne 7034/1
"Panikau Hill Reconstruction", Panikau Road Intersection.

5] P.W. Gisborne 7256
"Panikau Road Intersection", Proposed Improvements.
Note the position of the slumped block, and the 15"culvert.

6] P.W. Gisborne 7256
"Panikau Road Intersection", Proposed Relocation of Ramp.

7] P.W. Gisborne 7502
"Panikau Road Intersection", Drop-out.
Plan of the Panikau site in 1972 showing the location of "DS" series penetrometers.
Figure 3.2: Site Geomorphology

Survey Control Peg Station
Survey Peg Station
Approximate Contours
Major Scarp (>1 m High)
Scarp (< 1 m High)
Highway Cut Scarp
Terraces
Earthflow Debris
Back-filled Failed Block
Localised Flow Failure
Swampy Ground
Inferred Earthflow
Road Depressions
Inferred Earthflow
Backscarp Areas Filled by Excavation

Figure 1.3: Air Photograph Overlay on Survey Data

(based on NZ Aerial Mapping Survey No: 5134, Photo No: G2/12, Flown 28/9/1977, Enlarged to 1:892)

- Traverse Line
- Control Peg Station
- Peg Station

(68% of MITCHELL 19911. M.Sc. Thesis)