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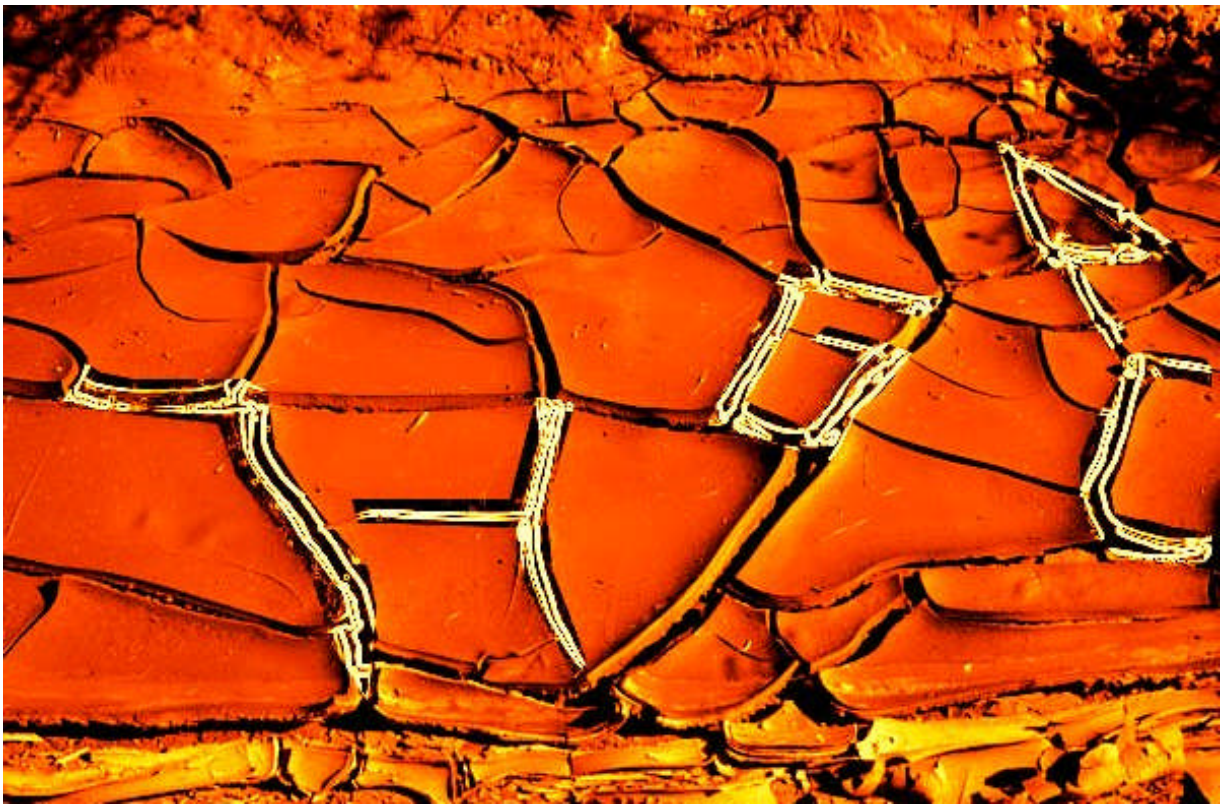
# 7YGPC

# 7th Young Geotechnical Professionals Conference

**ADELAIDE, 2006**

## Conference Proceedings



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7YGPC

# Proceedings of the 7th Australia – New Zealand Young Geotechnical Professionals Conference

**STAMFORD GRAND, ADELAIDE, OCTOBER 18TH – 21ST 2006**

**EDITORS**
**Brenton Harris, Wayne Potter, Brendan Scott**

Australian Geomechanics Society

New Zealand Geotechnical Society

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# Welcome

Welcome to the *Seventh Australia – New Zealand Young Geotechnical Professionals Conference*, Adelaide, Australia, 2006. Following on from previous successful conferences held in Sydney (1994), Auckland (1996), Melbourne (1998), Perth (2000), Rotorua (2002) and Gold Coast (2004), this biannual event is a joint initiative of the Australian Geomechanics Society and the New Zealand Geotechnical Society.

The aim of the conference is to:

- promote the **professional development** of delegates through sharing experience and ideas, and by presenting a paper to senior professionals and peers.
- expand and strengthen the lines of **communication between young professionals** within the field of geomechanics.
- promote an **enhanced perspective** of the varied roles, responsibilities and opportunities encompassed by the geotechnical profession.

The specific goal of the organizing committee has been to prepare an interesting, meaningful, informative, and enjoyable conference from which delegates can gain an enhanced understanding of their profession.

As with all previous YGP conferences, support of the geotechnical community is very important to ensure a successful end result. We would like to thank all companies and individuals that have provided assistance with the preparation of the conference. In particular, committee members: Brendan Scott, Wayne Potter, Jason Goldsworthy, Mark Jaksa and Matthew Duthy, who have contributed considerable time and effort over the past 2 years.

Thanks also to Mark Drechsler, Richard Herraman, Tony Meyers, Peter Mitchell and Don Richardson for their role in reviewing delegate papers. Special thanks also to Peter Robinson of the AGS for managing the finances for this event.

Keynote presenters and Senior Mentors: Mr Garry Mostyn, Dr Peter Mitchell, Mr Alan Moon and Mr Richard Herraman; without your valuable input, the conference would lack much of what makes it such a success.

Delegates should interact with their peers openly and maximise their exposure to the geotechnical community during the conference. A short question time has been allocated to each talk and I encourage delegates to ask questions during this period.

Most of all, have a rewarding time at the conference. The organizing committee members look forward to meeting you all, and joining you in an enjoyable and informative three days.

**Brenton Harris**
**Chair - Organizing Committee**

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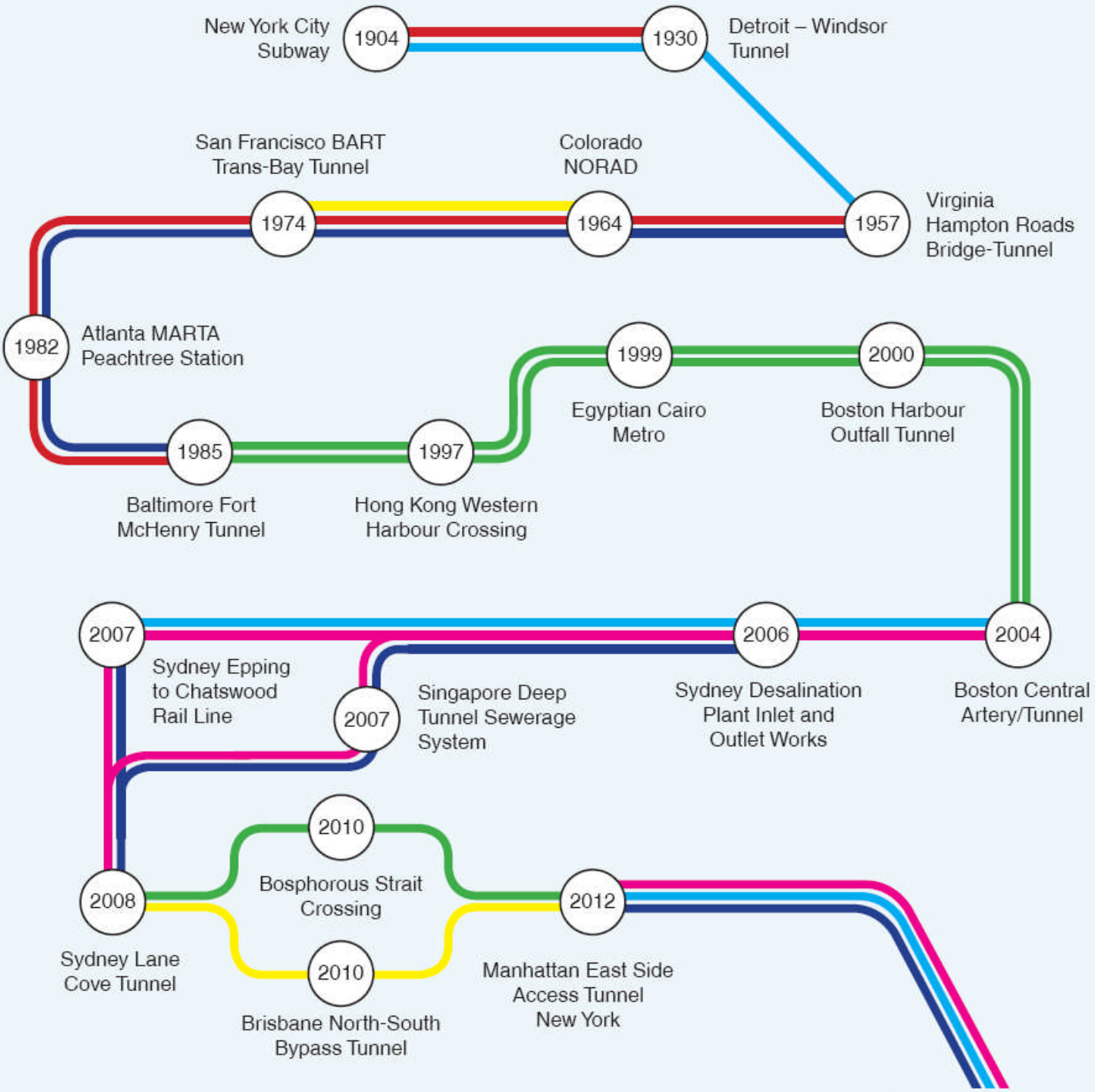


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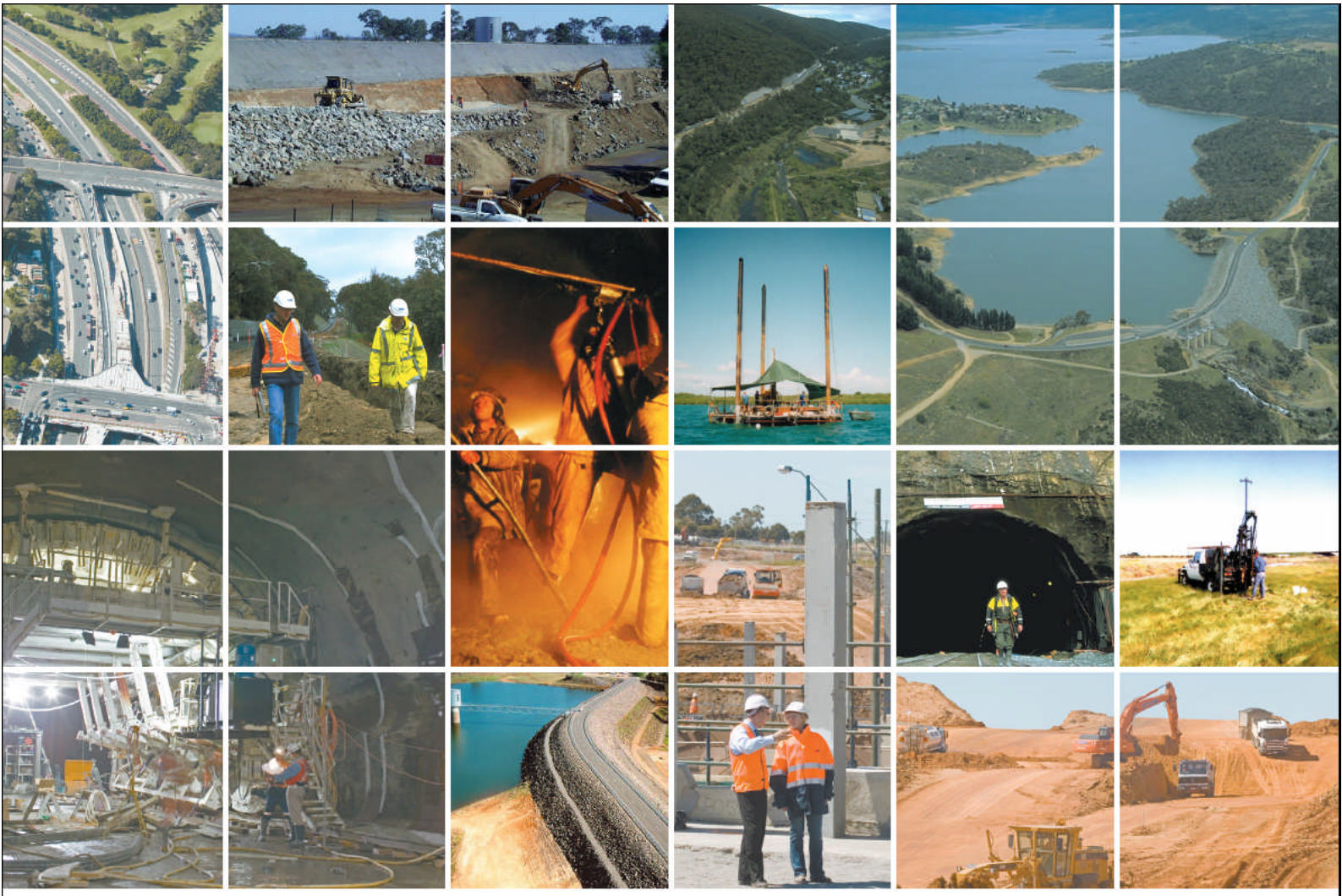

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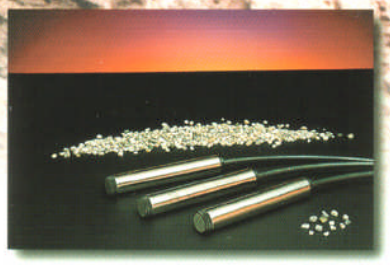

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# NEW STABILITY ASSESSMENT METHODS AT CONQUEROR MINE, ST IVES GOLD MINE

**Barsanti, B. J.**
*Gold Fields Australia, St Ives Gold Mine*
*Kambalda, WA*

**Andrews, P. G.**
*Gold Fields Australia, St Ives Gold Mine*
*Kambalda, WA*

## ABSTRACT

A reliable stability assessment for stoping is critical at the design stage of mining and the Conqueror Mine at Gold Fields' St Ives operations is no exception. The mining method at Conqueror is longhole open stoping with diminishing rib to island pillars.

The Mathews Stability Graph Method (SG) was initially used to assess the Conqueror primary stopes but proved unwieldily with the onset of the extraction of the secondary stopes from rib to island pillars. The St Ives Geotechnical Department therefore had to search for an alternative stability assessment method.

A Canadian stability method, Radius Factor (Milne, 1996), was investigated and subsequently trialled. Whilst Hydraulic Radius as considered in the Stability Graph method is based on boundary geometry (length x width), the Radius Factor method is based on surface geometry. Two new terms were introduced, Radius Factor (RF) is related to the overall stability and maximum deformation of a surface and Effective Radius Factor (ERF) is related to the local stability and deformation of a point on the stope surface.

The method was trialled as a tool for stope assessments and to back analyse old stoping areas. Back analysis showed the method to be effective for determining maximum stable spans. This result has enabled stope geometry and sequencing changes to incorporate best geotechnical practice into the mine extraction sequence.

## 1 INTRODUCTION

The St Ives Gold Mine is located on Lake Lefroy, approximately 60km south of Kalgoorlie. The site comprises 3 underground operations and 4 active open pits. It produces 550,000oz of gold annually.

The Conqueror Mine is one of three mines in the Leviathan Underground Complex which produces at a rate of 75,000 tons per month. The Conqueror series of stopes at Leviathan has 11 separate stopes from 20m to 135m long. Heights of the stopes vary from 15m to 45m.

## 2 SITE HISTORY

Open pit gold mining commenced at Victory Mine by WMC Resources in 1980 with subsequent underground development commencing in 1981. The Victory underground was mined in conjunction with open pit mining of the adjacent Defiance Pit from 1985, now called the Leviathan Complex. All underground mining ceased in 1998. Mining of several open pits in the immediate area, including Victory North, Britannia, Sirius and Victory North Extended, was completed by 1994.

The primary mining methods at Victory UG mine included mechanised mining by room and pillar, benching, flat backing, pillar stripping and longhole open stoping (LHOS). Most large (>40m high spans) LHOS voids are in the lower levels of Victory with most of the upper level void areas being extensive room and pillar. The Defiance pit mined through room and pillar workings which are now exposed in the east wall of the pit. Victory and North Victory pits broke through into vertical sediment lodes. By 1998 there was an estimated 90 km of lateral jumbo development during the mine life.

Total allocated ore production from open pit and underground mining was 14Mt for 1.76Moz of contained metal. Underground mining contributed 9Mt for 1.25Moz of this total.

In 2001, St Ives Gold Mine was brought by Gold Fields which started dewatering the Leviathan Underground Complex in 2002. The Leviathan Underground Complex currently consists of the non-active Victory and Defiance pits and 3 UG

mines; Sirius, East Repulse and Conqueror, of which 2 are currently active and the other is undergoing further exploration.

## 3 GEOLOGY

The Conqueror mineralised zone comprises several stacked lodes consisting of Felsic Porphyry, Defiance Dolerite and Kapai Slate, similar to those mined in the Defiance open pit. The lodes dip 20° to the southwest. They are up to 135m in strike length from west to east and are of variable vertical thickness. The location and orientation of mineralisation is controlled by the intersection of a low-angle shear zone (D5 shear zone) with the southwest striking Defiance Dolerite and Kapai Slate. The D5 shear zone is generally tightly healed although gypsum infill has been noted in the upper levels of the mine.

## 4 STOPE DESIGN

The Conqueror design consists of 11 stopes varying from 20m to 135m long and 15m to 45m in height. All stopes are 20m wide. The stopes strike east – west with accesses staggered down dip. The mining method will consist of primary longhole open stoping (LHOS) with diminishing rib to island pillars in the secondary stopes.

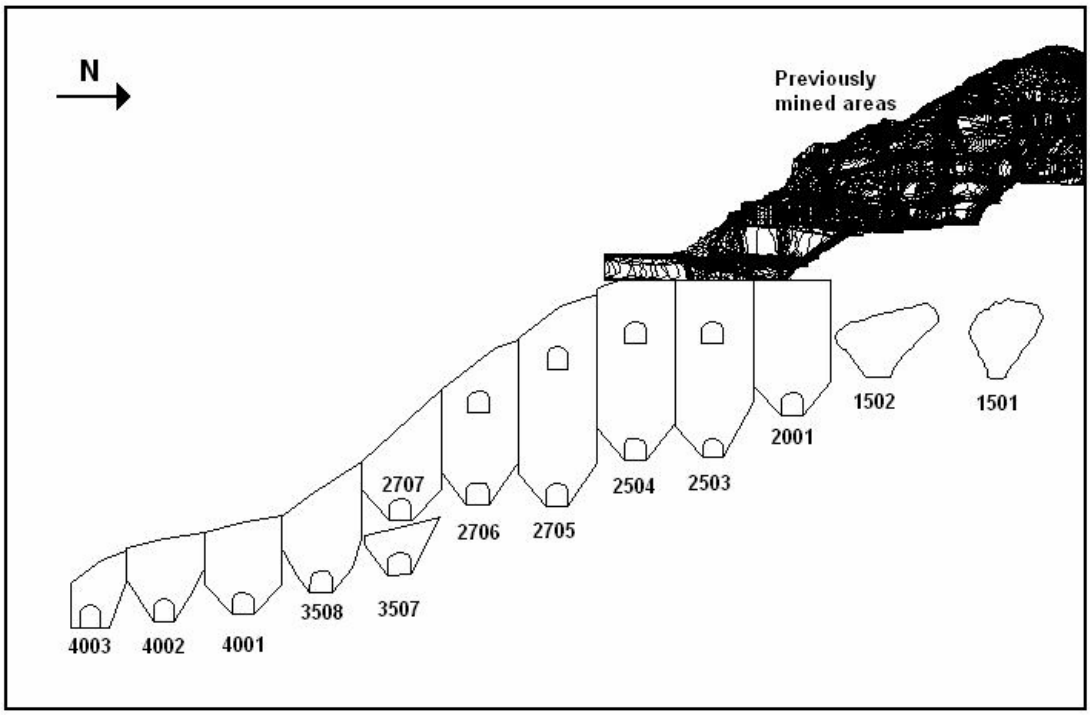



Figure 1: South – North cross section of the Conqueror stopes

Pillars have been designed into the secondary stopes with the exception of the 2001 stope which has a low grade zone which was left as an additional pillar. All pillars are 20m wide x 20m long and of varying height. The width to height ratios vary from 1:2.25 in the larger stopes to 1:0.75 in the smaller stopes to the south.

The four stopes with intermediate access will be stoped using a combination of short up holes from the lower levels along with up holes and long down holes from the intermediate level. During extraction of the stopes, the undercut retreats at a greater rate than the overcut, although a trial involving blasting the overcut and undercut simultaneously has occurred.

Once all the stopes have been extracted an open span down dip of 300m will be exposed. This void will extend from the old workings (280m below surface) to the end of the current stoping area (450m B.S.).

The stoping sequence implemented is not a conventional method of extraction, but the extraction of two or more primary stopes will commence and before they are completed the secondary stope extraction will commence.

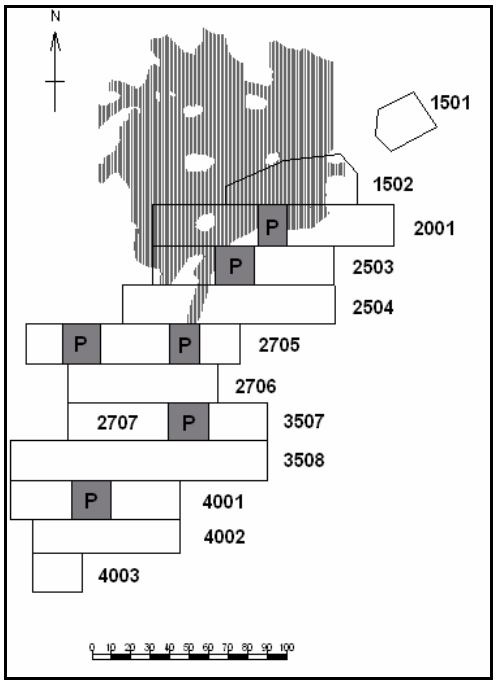



Figure 2: Plan view of the Conqueror stopes and pillars with the old workings overlayed

# 5 ROCKMASS AND STABILITY ASSESSMENTS

## 5.1 QUANTITATIVE DESCRIPTION OF THE ROCKMASS

The rockmass has been classified using the International Society for Rock Mechanics (ISRM, 1981) suggested system. The system allows various geometric and strength parameters to be characterised. These parameters have been tabulated below.

| Set | Dip | Dip Direction | Total Spacing | Persistence | Roughness | Wall Strength | Aperture | Filling | Seepage | Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 24 | 180 | V. Closely spaced | High | Smooth, planar | 40MPa | Tight | Talc & Gypsum | Dry | D5 shear |
| 2 | 90 | 358 | V. Widely spaced | Very High | Rough, planar | 280MPa | Tight | Quartz & Calcite | Dry | Dominant set |
| 3 | 74 | 303 | M. Widely spaced | Low | Rough, planar | 280MPa | Tight | Quartz & Calcite | Dry | Steep NW |
| 4 | 69 | 138 | M. Widely spaced | Low | Rough, planar | 280MPa | Tight | Quartz & Calcite | Dry | Steep SE |

Table 1: Joint classification data

## 5.2 ROCKMASS CLASSIFICATION

The Q system (Barton. et al, 1974) is used across all underground sites at St Ives Mine for rockmass classification purposes. The Q' value (Barton. et al, 1974) has two components; RQD/Jn is a measure of average block size for a jointed rock mass and Jr/Ja is a measure of joint surface strength and stiffness. The Q' value is calculated using the following equation:

$$Q' = \frac{RQD}{Jn} \times \frac{Jr}{Ja}$$

## 5.3 MATHEWS STABILTY GRAPH METHOD

The Stability Graph concept is a means of empirically predicting the stability of large unsupported openings and was conceived by Ken Matthews and co-workers (Mathews et al, 1981). It has proved to be a useful design tool for hanging walls of non-entry stopes. The original concept has been calibrated by Potvin (1988) and modified by Nixon (1992). The stability number N' is determined by multiplying the Q' number (Barton et al, (1974), by a stress factor (A), a structural orientation factor relative to the hanging wall (B), and a gravity factor (C).

The Hydraulic Radii is then calculated by dividing the area of the stope by its perimeter. Hydraulic radius is only suited to 2-Dimensional rectangular surfaces. Only the primary stopes in could be analysed using the method. Complex stope geometries when island pillars are created cannot easily be incorporated into the system.

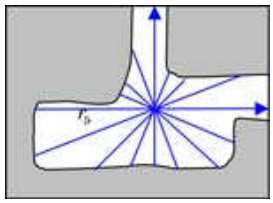
| | Q' | A | B | C | N' |
|---|---|---|---|---|---|
| **Mean** | 15.8 | 0.95 | 0.28 | 2.6 | 10.9 |
| **50th percentile** | 12.1 | 0.95 | 0.28 | 2.6 | 8.4 |
| **25th percentile** | 9.3 | 0.95 | 0.28 | 2.6 | 6.4 |
| **75th Percentile** | 23.2 | 0.95 | 0.28 | 2.6 | 16.0 |

Table 2: Conqueror rockmass classification data

### 5.4 EFFECTIVE RADIUS FACTOR

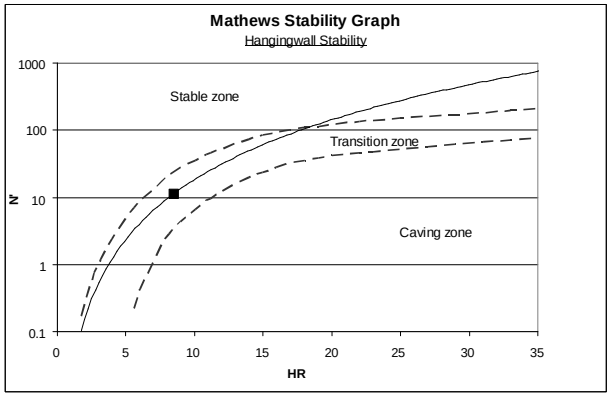
Due to the complexity of the design, stope design and stability analysis were carried out using the Radius Factor (RF) (Milne, 1996) method. The RF method is based on the deformation of complex geometries. The effective radius factor (ERF) measures the distance from a point located on the backs to the nearest abutment, pillar or brow. These measurements are taken at a specific radial angular interval, such as every 5°. ERF will have a maximum value towards the centre of the surface, where it is equivalent to HR, and decreasing values towards the edges. This method allows the effect of pillars to be taken into account in relation to hanging wall stability and can be used with irregular shaped backs, brows and intersections. The maximum RF numbers obtained can then be used on the Mathews Stability Graph in place of the hydraulic radius value to create the equivalent of the Mathews Stability Chart. The Radius Factor is calculated using the following equation:

$$RF = \frac{0.5}{\frac{1}{n}\sum_{\theta=1}^{n}\frac{1}{r_\theta}}$$



Where;

$n$ = number of measurements and $r_\theta$ = distance to abutment, pillar or brow



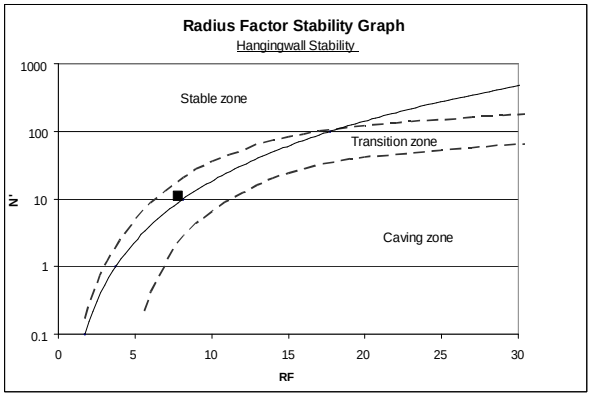




Figure 3: 2504 stope stability graphs - Mathews Vs ERF

ERF values can then be calculated on a grid pattern across the entire surface of the span to be analysed. Once multiple ERF values are obtained the values can be contoured. Contoured ERF values can be produced in 2 and 3 dimensions. For the stability assessment of Conqueror the ERF values were contoured using a Kriging method on the Surfer® software package.

The ability to contour the ERF values and overlay them onto a 2-D or 3-D representation of the span allows for much greater visual appreciation of areas of possible instability. The whole mining sequence can be viewed, step by step from start to finish. Using this sequential visual display it is easy to determine the mining sequence that allows for the greatest hangingwall stability for the life of mining.

# 5 STRESS ANALYSIS

Numerical modelling is also used to assess hangingwall stability with the help of the program, Map3D® (Wiles, 1988). This program has been used as a stability assessment tool and assisted with back analysing previously mined stopes.

As part of the Mathews Stability method the effects of mining induced stress are considered. The strength of the dolerite at Leviathan is at least 7 times greater than the induced stresses. Recent acoustic emission testing was carried out at Conqueror from a depth of 324m to estimate the in-situ stress and produced the following results:

| | Value (MPa) | Bearing (°) | Plunge (°) | MPa per metre |
|---|---|---|---|---|
| **Sigma1** | 25.9 | 288 | 03 | 0.079 |
| **Sigma2** | 21.9 | 197 | 28 | 0.068 |
| **Sigma3** | 8.0 | 023 | 62 | 0.025 |

Table 3: Sigma values for Conqueror Mine

The maximum induced stresses at the southern end of the mine (which is approximately 425m) are:

- **Sigma1** = 33.9MPa (horizontal east-west)
- **Sigma2** = 28.8MPa (slight dip to the south)
- **Sigma3** = 10.5MPa (steep dip to the north)

The direction of sigma 1 is oriented along strike of the D5 shear zone while sigma 2 is oriented along the plunge of the ore and sigma 3 is normal to the D5 shear.

# 6 RESULTS

## 6.1 BACKANALYSIS

As part of the geotechnical assessment of the stoping areas, a backanalysis of the old WMC workings in Conqueror was done using the ERF method to provide the previous span stability limitations. It was determined that the WMC design did not exceed an ERF value of greater than 13. The backanalysis of the stoping area showed after a pillar failure, an ERF value of 22 was created and subsequently the hangingwall broke to the D5 shear. The pillar failure was determined to be a tensional failure due to the stress directions and not a critical span failure.

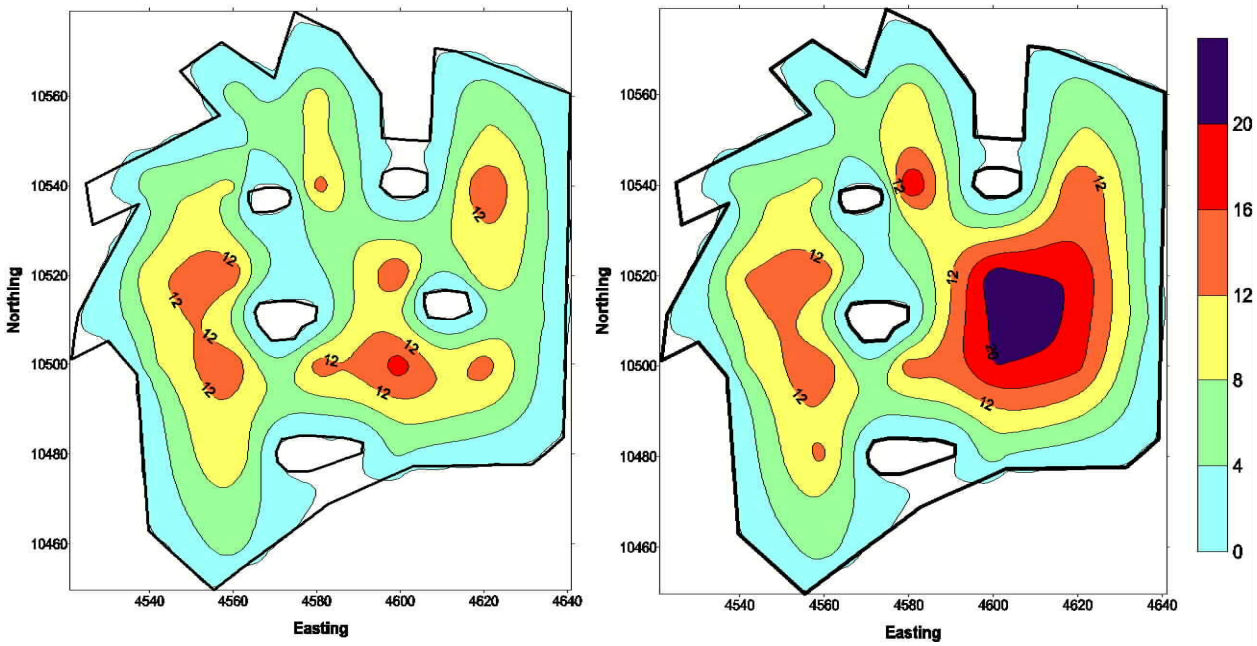
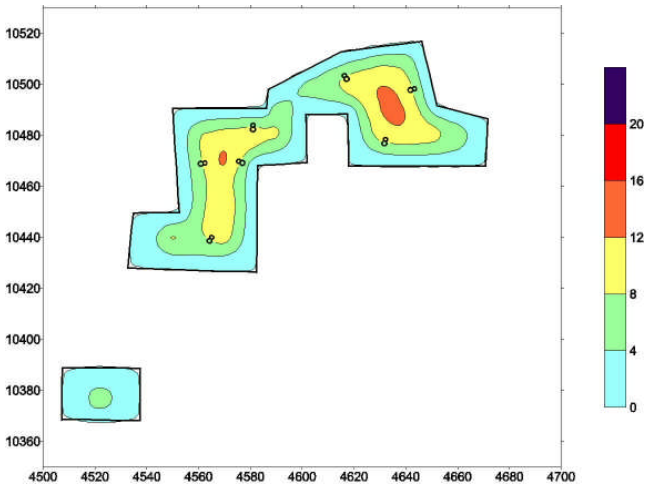



Figure 4: Plan view of the old workings ERF contour

## 6.2 PROGRESSIVE MINING RESULTS

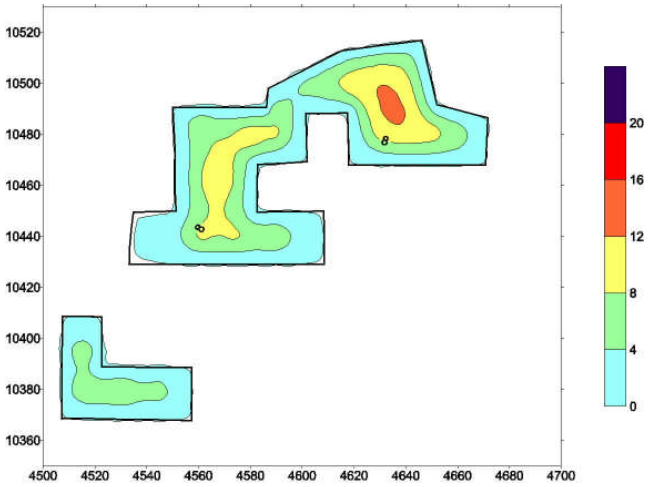
April

- The final extraction of the 1502 and 2001 stopes.
- The 2707 stope commences,
- Extraction of the 2503 and 2504 stopes is ongoing,
- The 2503 and 2504 stopes broke into the old WMC workings above,
- The largest ERF value of 13 is located between the 1502 and 2001 stopes.
- A north – south span of 60m is created from 2001 to 2504,
- Observations from underground; there is no visible overbreak or fretting of the hangingwall.



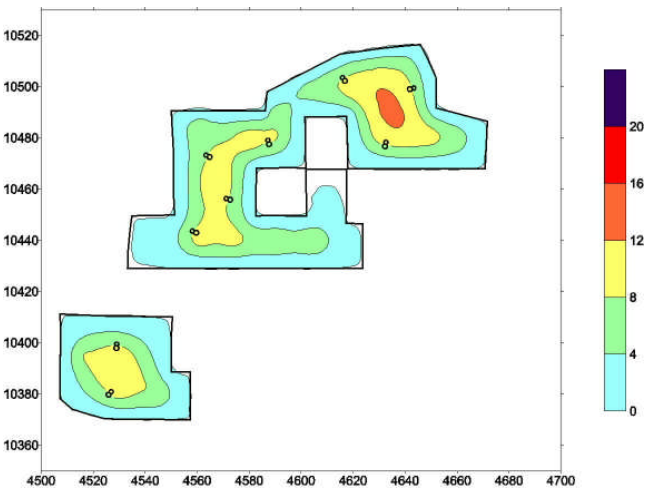

May

- The 2504 stope is extracted further,
- The 2706 stope commences,
- Final extraction of the 2707 is completed,
- ERF contour of 6 is reached in the 2706/ 2707 stoping area and 8 in the 2503/ 2504 stoping area.
- Observations from underground; there is no visible overbreak or fretting in the hangingwall.



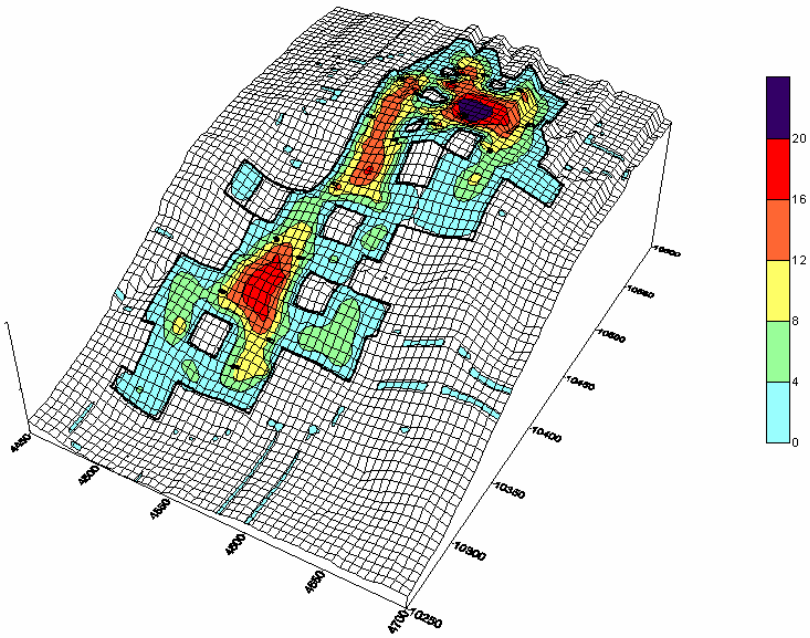

July

- The 2504 and 2706 stopes have been extracted further.
- Two island pillars are created in the 2001 and 2503 stopes,
- The 2503 stope 2 (east of the pillar) commences,
- ERF contour value of 8 is attained in the 2706/ 2707 stoping area.
- Observations from underground; there is no visible overbreak in the hangingwall in any areas.
- Fretting in the 2706 and 2707 area has occurred due to the hangingwall geometry and is not span related.



End of mine expectations

- The main area of concern is the stoping area between the 2705 and the 3508 as depicted by the red contours,
- This span reaches an ERF value of 19,
- From the back analysis conducted, a result with an ERF of 20 or greater increases the likelihood of hangingwall failure.




## 7 CONCLUSION

Since the inception of ERF as a geotechnical tool for stability assessments, it has provided St Ives an up to date visual display of stope stability. The ability to visualise stope stability in 3D allows for easier modifications to the stoping sequence and changes in geometry.

The major change ERF has contributed is in the Conqueror stoping sequence. Sequence reiterations using ERF as the controlling factor has allowed the 2705 stope to be brought forward in sequence. This change has enabled higher production rates whilst still maintaining hangingwall stability. It has also provided confidence that by extracting the 3508 stope last, that hangingwall stability will be maintained throughout the extraction sequence.

ERF values will be monitored on an ongoing basis as stoping progresses for the remainder of the mines life. The ERF data will be correlated with underground observations to provide up-to-date stability assessments. The added benefit of ERF is its ability to analyse hangingwall stability with an emphasis on future pillar recovery. A stope stability assessment can be analysed after modifications have been made to pillar dimensions or locations and the effect on hangingwall stability can be seen immediately.

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# STRATIGRAPHIC INTERPRETATION OF EAST COAST ESTUARINE SEDIMENTS USING THE CPTU

**Daniel Bishop and Stephen Fityus**
*University of Newcastle*
*Newcastle, NSW*

## 1. ABSTRACT

The cornerstone of any geotechnical site investigation and resulting geotechnical model is the geological framework developed and used for the investigation. Correct derivation of the spatial relationship between different materials requires an understanding of the original depositional environment. In this regard the structural complexity of the estuary-fill in coastal estuaries of NSW represents a major engineering challenge in the ongoing upgrade of the Pacific Highway and coastal developments in general.

This paper presents a method for interpreting piezocone (CPTU) profiles which utilizes recent advances in the understanding of the structure of coastal valley fills and the authors' work into the evolution of the Richmond River Estuary. These recent advances in estuary models have culminated in the development of the Incised Valley Sequence Stratigraphic Model (IVSSM). This model links the evolution of a sequence[1] of estuary fill, in terms of the spatial distribution of the depositional environments, to changes in the regional sea level curve over the period of infilling. The IVSSM defines how the sedimentary units (also called facies) that result from the filling process are spatial arranged and also the nature of the boundaries that separate them. The facies and the boundaries between facies have particular geomechanical characteristics defined by the grain size, bedding, mode of formation and the geological age which can be interpreted directly from the IVSSM. Since a CPTU profile detects both the mechanical behaviour of a single facies and changes in mechanical properties between facies the profile patterns can be interpreted directly within the IVSSM.

This paper demonstrates, through examples from the Richmond River estuary, the link between shape of the CPTU trace and the key boundaries and facies in the IVSSM model. Discussion is provided on the potential benefit that identification of these boundaries and facies has for defining the vertical and lateral distribution of geotechnically significant units, and how this can reduce the extent of investigation required to identify and correlate engineering properties across a large estuarine deposit.

## 2. SEQUENCE STRATIGRAPHY AND THE NSW COASTLINE

From a geotechnical perspective, an improved ability to infer the geometry and lateral extent of sediment units from isolated vertical profile data can greatly reduce the cost of investigation and enable improved estimation of sediment properties. Reliable models describing the arrangement and properties of natural estuary systems require a detailed understanding of the processes that have been responsible for the deposition of sediments throughout the system. In essence, a sedimentary deposit represents the integration over time of geological processes (erosive and depositional) that are specific to that deposit. In coastal deposits, such a those found along the NSW coast, the most significant parameter that varies over time is the elevation of mean sea level (MSL), as controlled by cycles in the earth's climate. Investigations of the structure of sediment deposits affected by MSL change has led to the development of 'sequence stratigraphy'.

The central idea in the sequence stratigraphy concept is that packages of sediment (facies) are deposited in particular depositional environments, depending upon the phase of the sea level curve in a particular climatic cycle. At any single instant in time, there exist a number of different depositional environments adjacent to each other. These are encountered successively, with increasing distance upstream of the river mouth. As the sea level changes, each of these depositional environments moves in the same direction as the advancing, or retreating, shoreline. The sequence stratigraphic framework for coastal systems within a valley structure are encapsulated in the Incised Valley Sequence Stratigraphic Model or IVSSM (Dalrymple et al. 1992; Dalrymple et al. 1994). The IVSSM is the basis for the ideas in this paper. An IVSSM interpretation of the Richmond River is presented, after the balance of past and present processes that drive sedimentation along the NSW coast has been considered.

The deposition and erosion of coastal sediments during the Quaternary along the NSW coast has been controlled by the interplay of the relative sea level curve, the surface morphology of coastal regions and the supply of sediment into the

[1] A 'sequence' is the total thickness of sediments deposited between two successive glacial events (sea level low stands).

coastal depositional system. Climatic factors directly affect the supply of sediment, controlling both the dominant wind and wave directions and also the regional rainfall patterns (Goodwin et al. 2006).

The relative sea level curve for the east coast of Australia over the past 340,000years is shown in Figure 1. The eustatic sea level variations shown in this figure are derived from oxygen isotope data from deep sea sediment cores (Roy, 1998). These changes are intimately linked to changes in global ice volumes. During periods of increasing global ice volume, the sea level falls (e.g. Stage 9, 7c, 5e) and shorelines retreat (interglacial regression). During glacial periods, sea levels are low, erosion within the valley system is widespread, and the thalweg may deepen. During postglacial periods, ice volume decreases (e.g. late stage 6 and stages 2, 1), sea level rises (postglacial transgression) towards an interglacial maximum and shoreline moves landward. Estuarine systems only exist fully during periods of highstand which means as a geological system they are a highly ephemeral feature of the Quaternary. The NSW coast is considered to be tectonically stable, wave dominated and not to have been subject to direct glacial activity (Roy 1998). A minor variation to the eustatic relative sea level curve (defined on a regional scale) is caused by hydrostatic loading (hydro-isostasy) of the continental shelf as the depth of sea water increases over the shelf and deforms the lithosphere (Chappell et al. 1982). This causes regions inland from the coast to lift upwards and over time, this can cause relative sea level fall, even if the eustatic sea level is stable.

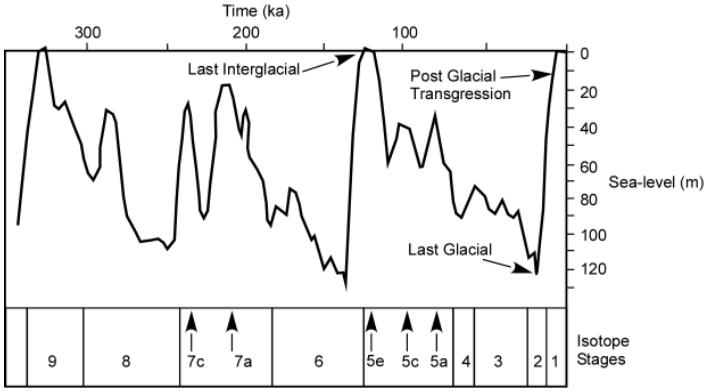


**Figure 1: Sea level curve for NSW taken from (Roy 1998) , Isotope stages based on interpretation of deep sea sediments.**

The cyclical pattern of sea level change greatly affects sediment deposition at some particular elevation, during the different periods of regression, glacial maximum, postglacial transgression or glacial minimum. In the case of the Richmond River estuary, the sediments were deposited over the range of +5m RL (last interglacial) to -45m RL (bedrock thalweg at the current estuary entrance).

Interglacial sediments (including those of the present day depositional environments) are characterised by 3 major lithosomes: the Barrier (incorporating the Tidal Delta), Central Basin and Fluvial Delta. These lithosomes make up the 'tripartite barrier estuary system' (Dalrymple et al. 1992) that characterises the current eastern Australian seaboard. A schematic representation of a typical barrier estuary system, based on the Richmond River work, is shown in Figure 2A. This figure shows the estuary at an immature stage of infill (open water conditions still present). Mature estuaries are completely infilled, with the last stage of infilling controlled by the hydro-isostatic rebound described earlier.

The key stratigraphic relationships of this system are summarised in section X-X in Figure 2B. The figure shows the three distinct sedimentation zones based on the surface patterns and the major stratigraphic boundaries. The existing models (Nichol et al. 1994) do not incorporate the preservation of significant volumes of stage 5e to stage 2 sediments (refer to Figure 1). This has been addressed in Figure 2B with the preserved material shown in grey. The sequence boundary, which is the most ubiquitous feature in the estuary fill, is the surface formed by erosion during a fall in the relative sea level. During glacial periods, the estuary region evolves into an incised valley, no longer adjacent to the coast. The Sequence Boundary separates transgressive and interglacial sediments deposited during one cycle of sea level rise and fall from sediments deposited in the subsequent cycle of sea level rise and fall.

Following highstand, rapid sea level fall results in deep incision into coastal plain sediments. The morphology of this erosion surface depends on the structure and consistency of the sediments/bedrock being eroded, the rate of sea level fall and the intensity of the hydrological processes. The remnant sediments (previous sequences) are stabilised predominantly by vegetation growth (Thorne 1994), however an important component of this preservation process on the Northern NSW coast is the formation of humate sand rock (Coffee rock). This 'rock' is formed by the precipitation of organic colloids at the seawater/freshwater interface within a coastal sand aquifer and is highly resistant to erosion. It is clear from the figure that the morphology of the preserved material has a significant effect on the distribution of

facies in the estuary. In summary the morphology of these preserved sediments is a system of 10-30m deep paleo-channels with river terraces and broad interfluves preserved at different levels (shown in grey).

During sea level rise there is a gradual fall in the hydraulic energy gradient across the incised valley. Fluvial sediment deposition increases and environments of deposition change gradually as the coastline moves landward, back across the continental shelf. The thalweg of the Richmond and many other NSW estuaries is around -60m RL and when flooded (Stage 1, Figure 1), marine sedimentation encroaches into the valley. Due to the preserved sediments at this point the barrier must aggrade vertically and the tidal deltas extend into the back-barrier regions. The next major transition in the estuary is when sea level breaches the level of the interfluves within the estuary basin. At this point the deposition environment changes from vertical aggradation within the paleo-channels to low energy tidal flat and central basin muds. This is the stage defined by the flooding surface shown in Figure 2B.

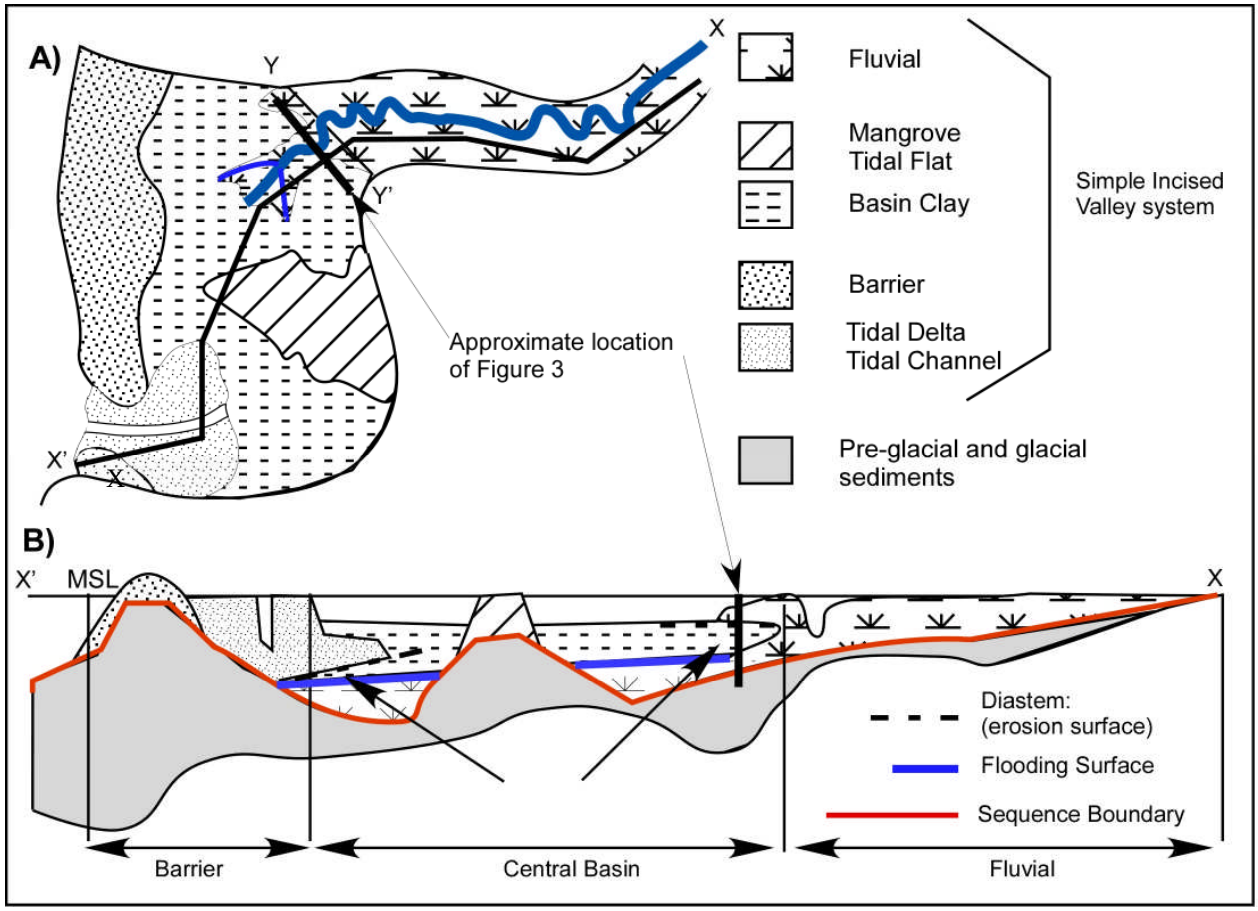


**Figure 2: A) Plan view of spatial distribution of estuary environments, B) Sequence Stratigraphic model derived along line X-X shown in A). The model is based on Richmond River data. Importantly, the profile of the last-interglacial and glacial stage sediments (Isotope stages 5e through to stage 2 in Figure 1; shown in grey) significantly control Stage 1 sediment patterns.**

## 3. APPLICATION OF THE CPTU TEST TO THE IVSSM

The CPTU test can be applied to interpret the key features of the IVSSM stratigraphic framework, based on the fact that the three major lithosomes and related stratigraphic surfaces have distinctly different mechanical behaviours that can be readily identified in the CPTU data. The lateral extrapolation of spatial information from a vertical profile is based on the knowledge that the depositional environments within each of the three segments of the barrier estuary system have particular spatial patterns. Once the stratigraphic position of lithosomes and surfaces is determined, it is possible to place constraints on geotechnically important structures such as paleo-channel dimensions, shapes and likely extents of sand bodies, locations of soft subsurface sediments etc.

Of primary importance is whether the stratigraphic relationships between different sediment types are characterised by a distinct sharp change in the mechanical behaviour of the sediment across a surface, or by a gradual change indicating a fining- or coarsening-upward grain size. The characteristics of the IVSSM surfaces that allow them to be identified in the CPTU data are summarised in Table 1. It is necessary to interpret the pore water pressure and tip resistance profiles

in tandem in order to distinguish between the different lithosomes, since fining- and coarsening-upward sediments are defined by both the number and ratio of course to fine beds in the unit as well as grain size changes within a single bed.

**Table 1: significant stratigraphic surfaces in the engineering context**

| IVSSM Boundary | Characteristic Features |
|---|---|
| Sequence boundary | erosion surface carved into interglacial sediments overlain by transgressive surface or transgressive fluvial sediments |
| Transgressive Surface | on-lap of sediments from depositional environments shoreward of the present environment arising gradually FINING UPWARD |
| Flooding Surface | rapidly FINING UPWARD sediments; shallow to deep water |
| Tidal Diastem | erosion surface into central basin clay, overlain by tidal channel or tidal delta sands COARSENING UPWARD |
| Fluvial Diastem | erosion surface into central basin clay overlain by fluvial sediments, organics COARSENING UPWARD |

The full interpretation of a Richmond River CPTU profile is shown in Figure 3 along with the related rotary wash borehole log. The presence of gravel/organics or shell fragments was used to confirm the transition between fluvial and central basin facies. A re-interpretation of the borehole using the CPTU data and the associated sequence stratigraphic interpretation is shown on the right of the figure.

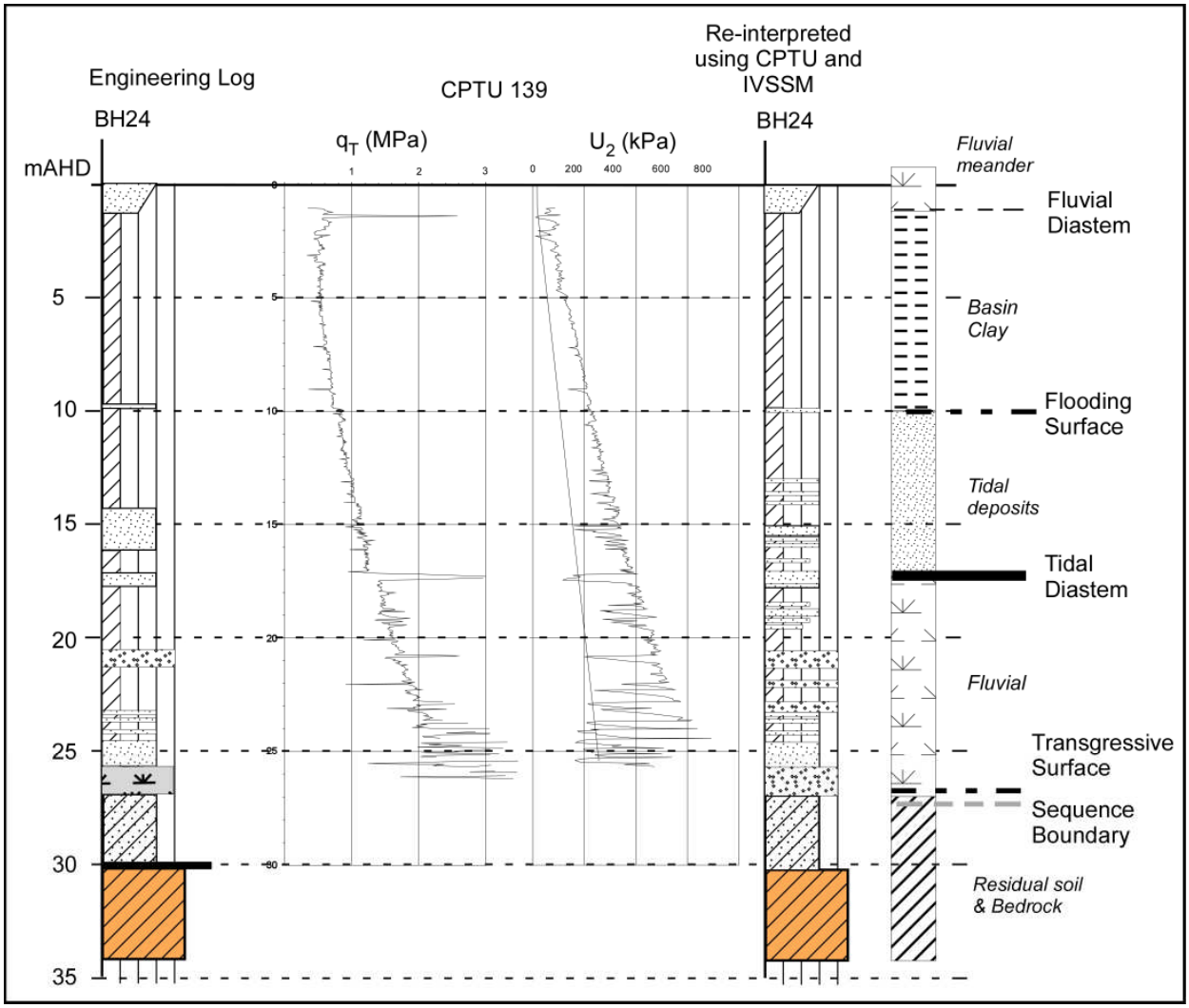


**Figure 3: Original engineering log re-interpreted using CPTU profile and IVSSM. Presence of shells or organic matter/gravel was used to distinguish marine and fluvial deposits respectively.**

The profile in Figure 3 is essentially composed of normally consolidated clay. Detailed correlation to core samples from the borehole indicated that the subtle variation in pore pressure towards the base of the profile were definitely thin (5-10cm) thick sand and gravel lenses. Using this data, the gradual fining up pattern in the fluvial system can be defined. This is characteristic of the early stages of transgression (Nichol et al. 1994). The top of the fluvial sediments is capped by an erosion surface which is defined by the sharp jump in tip resistance at -17.5m RL. This tip resistance change is likely to be the result of desiccation on the upper surface of the fluvial deposits. This surface is interpreted as a Tidal Channel Diastem (erosion surface) since it is overlain by interbedded shelly sands. The pattern of the sand beds fines up gradually and the small change in slope of the tip resistance data (-10m RL) is interpreted as the flooding surface. This change in slope can be interpreted as a difference in the rate at which the mechanical behaviour of the clay changes both above and below this depth.

Figure 4 presents the interpretation of section Y-Y, located as shown in Figure 2B, which includes CPTU 139 from Figure 3. The section is from within a tributary valley of the Richmond River. As it is not possible to cover all aspects of the interpretation in the space available, only the key aspects will be highlighted.

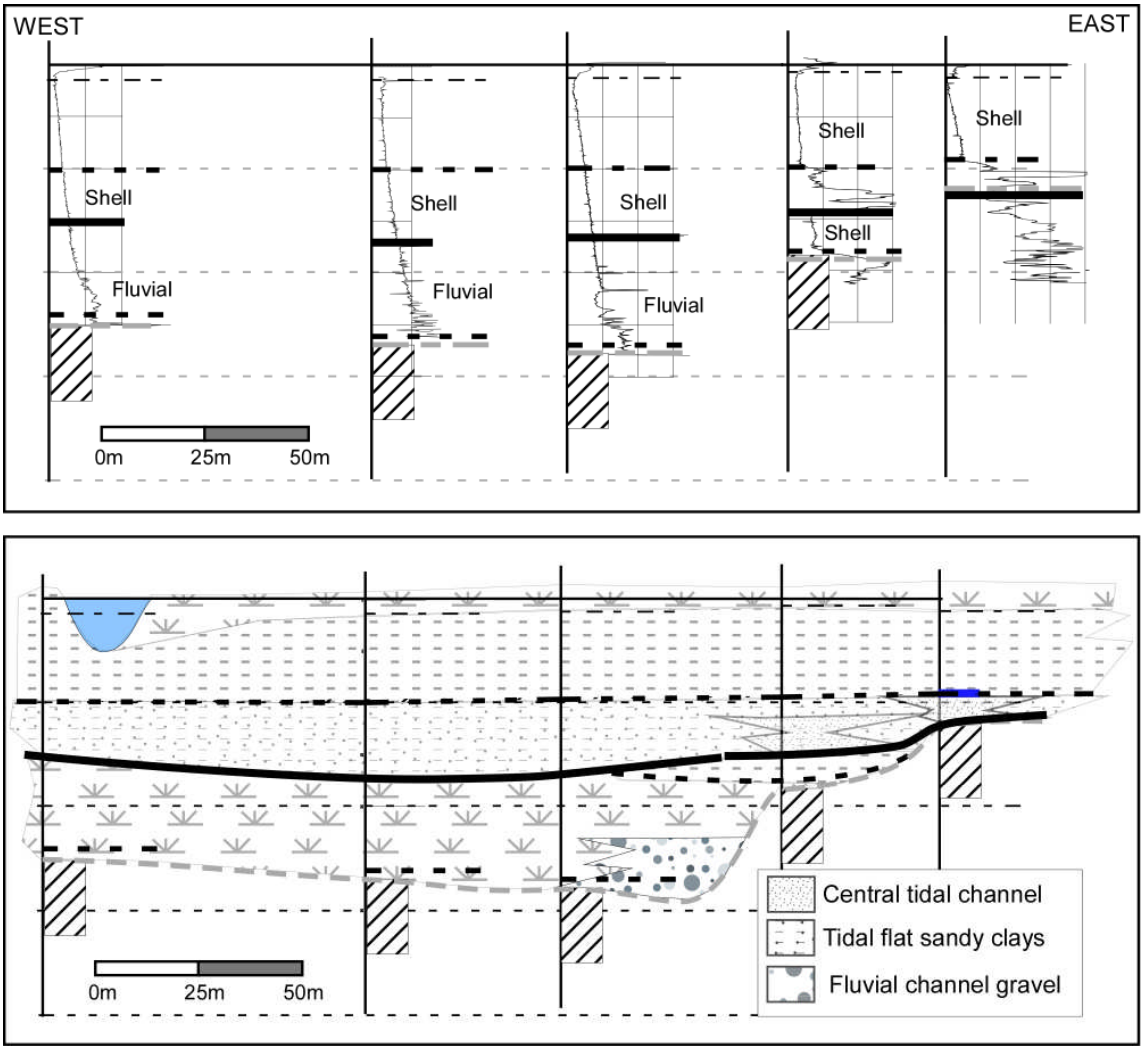


**Figure 4: A 250m cross-section Y-Y through a tributary to the main Richmond River Channel (refer to location shown in Figure 2A.) The valley is approximately 700m wide (measured at 0 m AHD) at this location. Only $q_t$ traces are shown in the diagram but interpretation also requires CPTU pore pressure data.**

Both the interpreted section and original data are presented in Figure 4. It is important to note that the stratigraphic surfaces can be discontinuous and in fact, this is commonly the case in systems that are highly active. An example of this is the lower flooding surface underlying the central tidal channel. It appears that an early phase of flooding was subsequently eroded by tidal processes. The width of the tidal channel has been constrained by CPTU profiles perpendicular to the section shown. Importantly the data suggests that the channel was of similar dimensions (25-30m) to the present day river (shown in light blue). It also indicates that the tidal channel was significantly straighter than the current meandering river system. During rising sea level and infilling of the estuary, the river system adjusts its profile

and cross section to adapt to the changing sediment load and hydraulic gradients (Miall 1985). The Sequence Boundary (SB, red) shows the characteristic paleo-valley and terrace pattern. To the east, the SB lies on desiccated fluvial clays and gravels, which overlie a sequence of marine clay. Both of these deposits are interpreted to have been deposited during the Stage 5e interglacial period. In the central and western parts of the section, the SB lies over bedrock. The lower fluvial deposits which overly the SB consist of a central gravel channel and interbedded clays and sand with minor organics. They are interpreted to have been deposited during the early stage of the Stage 2 transgression. The difference in $q_t$ values between the Stage 5e (2-2.5MPa) and Stage 1 (1-1.5MPa) clays is used to define the SB in other regions of the estuary.

## 4. CONCLUSION

This paper has demonstrated the basis of an interpretation method for CPTU tests in barrier estuary systems, as applied to the Richmond River in Northern NSW. The basis of the interpretation method is the fact that the pattern of sedimentation in these estuaries can be predicted to a greater extent using sequence stratigraphic concepts. The sequence stratigraphy, summarised in the Incised Valley Sequence Stratigraphic Model (IVSSM), relates variations in the relative sea level curve during the Quaternary to depositional environments found at a particular elevation in the incised valley. The boundaries and facies defined in the IVSSM also define surfaces and sediments which can be distinguished based on their mechanical behaviours defined by the CPTU. Recognition of the CPTU patterns relating to the different components of the IVSSM allows rapid determination of the stratigraphic structure of the estuary. Once the surfaces and facies have been identified a knowledge of the deposition environments and processes that operated in that facies can be used to extrapolate the CPTU data laterally away from the profile.

It has not been possible to present all the methodology used to interpret the original data due to paper length restrictions. However, the paper has demonstrated the level of interpretation that is possible using data from standard geotechnical methods, by understanding in detail the processes that have controlled the sedimentation. Ultimately this information can be utilised to improve the geotechnical models used in predicting the behaviour of structures constructed on these deposits.

## 5. ACKNOWLWDGEMENTS

The authors would like to acknowledge the cooperation of The Roads and Traffic Authority NSW, Coffey Geosciences and RCA for making data available for this study. Financial support from the Australian Research Council is also acknowledged.

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# URBAN CAPABILITY ASSESSMENTS IN NEWCASTLE, NSW, AUSTRALIA

**Matthew Blackert**
*Douglas Partners Pty Ltd*
*Newcastle, NSW, Australia*

## ABSTRACT

Urban Capability Assessments (UCA) are undertaken to provide developers and planning authorities with preliminary geotechnical and environmental information and recommendations at the initial concept stage of a development. UCAs ensure that geotechnical and environmental constraints and opportunities are considered at the concept design stage rather than at a later stage in the development process, thus minimising the potential for adverse environmental impact and/or costly geotechnical solutions. Two recent UCAs for proposed large scale residential developments were conducted at Cooranbong and Cliftleigh, NSW. These identified a number of different environmental and geotechnical issues, including shallow mine workings, saline and acid sulphate soils, slope instability, and contaminated soil/groundwater. The assessments resulted in the redesign and modification of concept designs to minimise development costs and maximise beneficial land use, and provided clear recommendations for additional studies required to allow detailed design of the proposed development to be undertaken.

## 1. INTRODUCTION

Urban Capability Assessments (UCA) require an integrated, multidisciplinary approach in the fields of geotechnical and environmental engineering to provide the necessary information for the initial critical concept stage of a development.

UCAs are an important first phase assessment, which have been successfully utilised to identify geotechnical and environmental challenges for new development, and form the basis for further stages in the development process. UCAs are increasingly being requested by Councils for rezoning submissions and also by developers prior to purchasing sites to establish potential land use constraints, opportunities and associated development costs (ie. first phase feasibility assessment).

UCA's generally include consideration of geotechnical and environmental issues such as topography, surface and subsurface geology, slope stability, presence and strength of rock, surface water and groundwater, possible clay and gravel resources, foundation/subgrade conditions, acid sulphate soil, salinity, mine subsidence and potential for site contamination. Assessments have generally included a desktop review of available data, a review of site history, site inspection, nominal subsurface investigation and testing. The results of the assessment are utilised to prepare site constraints maps, which can be integrated with other studies such as bushfire, flora & fauna etc, for use at the concept design stage of a proposed development.

This paper discusses the basic principles of undertaking UCAs, and subsequently focuses on two sites which have been recently assessed at Cooranbong and Cliftleigh. Both sites have been identified as having geotechnical and environmental challenges requiring further assessment or consideration prior to development.

## 2. BASICS OF URBAN CAPABILITY ASSESSMENTS

Urban Capability Assessments are designed to provide a preliminary overview of the geotechnical and environmental characteristics of a site and generally include the following:

- Desktop review including review of in-house reports on the site or neighbouring sites, topographic plans, soil landscape sheets, acid sulphate soil risk maps, regional geology sheets, aerial photographs and orthophotos, salinity risk maps and regional groundwater characteristics (registered groundwater bore search).
- Site reconnaissance survey (site inspection) by an experienced engineer;
- Discussions with site owners or personnel familiar with the site and/or local area;

- Preliminary subsurface investigation and sampling;
- Preliminary laboratory testing;
- Preliminary contamination assessment (NSW EPA, November 1997), which is likely to include site history review, discussions with personnel familiar with current and past activities on-site, walkover survey as above, preliminary subsurface investigation and laboratory testing (as required);
- Reporting.

The assessments are undertaken to consider the potential geotechnical and environmental constraints including erosion hazard, slope stability, uncontrolled filling, excavatability, soil reactivity, saturated soils/poor ground, rock outcrops/shallow bedrock (excavatability), acid sulphate soils (Stone Y, Ahern C R, and Blunden B, 1998), salinity (DIPNR, 2002), mine subsidence, and potential soil and water contamination.

Preliminary subsurface investigation and laboratory testing is generally undertaken to confirm typical subsurface conditions and physical and chemical characteristics. The results of the assessment are used to summarise geotechnical and environmental issues associated with the site, with recommendations provided regarding likely remedial and/or management measures required to address these issues, additional assessment requirements, and most importantly the suitability of the site for the proposed development.

An important component of the UCA is the site inspection, which should be conducted by personnel with experience in both the geotechnical and environmental fields due to the multidisciplinary nature of the assessment.

## 3. CASE STUDY - COORANBONG

The North Cooranbong site (Figure 1) comprises a total area of greater than 300 ha within approximately 16 allotments, and is located to the north of the existing Cooranbong town centre. The site comprises a mixture of rural, rural/residential and commercial properties, with current industries including an aerodrome and two operating poultry farms. The UCA (Douglas Partners Pty Ltd, 2001, 2002, 2003) was conducted for the developer, and was included with a rezoning submission to local Council. The UCA was supplemented where required with additional investigation to further assess particular issues identified in the UCA.

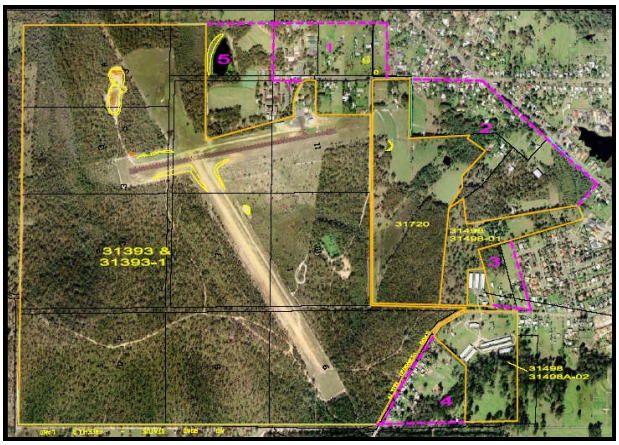


Figure 1 – North Cooranbong Investigation Area

The UCA included a desktop study, site history review, site inspection, subsurface investigation, soil, surface water and groundwater sampling and laboratory testing.

The desktop review indicated the following:

- the site is underlain by the Early Permian aged to Triassic aged Narrabeen Group, which typically comprises sandstone, conglomerate, siltstone, claystone, chert and shale;
- the site lies within a number of soil landscapes including the Doyalson erosional landscape and Yarramalong landscape, which comprise soils with the following potential hazards - erosion hazard, seasonal waterlogging, high run-on, strongly acidic, foundation hazard, high plasticity, moderate to high shrink-swell (clays) and low fertility;

- topography varies across the site with slopes typically less than 10 º, local relief between 3 AHD and 50 AHD, and a number of gullies and creeks traversing/bordering the site. Evidence of site filling and quarrying were present in the topographic maps;
- acid sulphate soils were not known to occur within the majority of the site, with the exception of the low-lying south-eastern corner of the site, which has a low probability of acid sulphate conditions at depths greater than 3m.

Geotechnical considerations identified in the UCA included erosion hazard, slope stability, uncontrolled filling, excavatability, soil reactivity, saturated low lying soils, acid sulphate soils and natural resources.

The general site conditions identified during the UCA can be summarised as follows:

***Erosion Hazard***
The results of laboratory testing indicated that the dominant soils at the site (i.e. clay) were non dispersive. A potential for adverse impacts resulting from erosion of upper silty sand topsoil materials was identified (ie. impact on water quality due to sediment laden run-off during and following development). However, owing to the limited quantity of this material encountered (ie 0.1 m to 0.3 m thickness), erosion and sedimentation were considered to be readily amenable to standard mitigation measures.

***Slope Stability***
There was generally no evidence of previous or incipient deep seated instability observed over the site. The site was generally considered to have a low risk of instability with respect to the natural topography. There was, however, evidence of erosion and surficial creep within batter slopes (cut and fill) identified within the site. Specific areas affected by cut and fill will need to be assessed at a later stage in the development.

***Uncontrolled Filling***
Localised areas of imported filling, placed in a non-engineered manner, as defined in AS 2870-1996, were observed across the site and would not be suitable for the support of structures in their current state. Additional subsurface investigation was recommended to assess the depth, extent and properties of filling if development was proposed in these areas.

***Excavatability***
Soils observed across the site generally comprised silty sands, clayey sands, silty / sandy clays, and weathered rock, which can be readily excavated by conventional earthmoving equipment. Site specific investigation during detailed design was recommended to assess the excavatability of the underlying bedrock.

***Soil Reactivity***
Laboratory testing of soil properties was not undertaken for the assessment. Based on previous laboratory testing in the Cooranbong area and the subsurface conditions encountered across the site, site classification is likely to range from Class M to Class H depending on the depth to bedrock. Soil reactivity, however, will be confirmed during future detailed investigations by classifying building sites in accordance with AS 2870-1996 (Standards Australia, June 1996).

***Saturated Soils / Poor Ground***
Localised saturated surface soils were observed in various locations across the site including gullies, low lying areas, creek/ponds, drains, effluent ponds and transpiration areas. Saturated areas may result in localised softening and swelling behaviour of surrounding soils, which should be considered for future detailed investigation and design. Some specific areas were considered to be unsuitable for the support of structures or pavements due to poor ground conditions and have subsequently been reserved as drainage areas in the concept design plan.

***Acid Sulphate Soils***
Based on the Acid Sulphate Soil Risk Map, specific investigation and procedures for management of acid sulphate soils will only be required if future development is proposed in the low lying southern portion of the site. Aggressive ground conditions (i.e. naturally acidic soils) may however be present within the site and should be considered for future detailed investigations.

***Natural Resources***
Subsurface conditions were anticipated to generally comprise silty / sandy clays to depths between 0.45 m and 3.0 m overlying bedrock (sandstone, siltstone and conglomerate at the former quarry). The suitability of this material as a resource for specific uses will depend on the proposed use and will require additional laboratory testing.

The geotechnical issues identified in the UCA for the site were generally localised and could be readily addressed prior to and/or during development through appropriate investigation and design.

The preliminary contamination assessment (PCA) conducted at the site as part of the UCA included a site history review and site inspection to assess potential contamination issues. The results of the PCA identified a number of potential contamination issues including:

- Effluent Sludge Dam – potential microbiological, nutrient and heavy metal impacts;
- Imported Filling –range of potential contaminants including metals, hydrocarbons, pesticides, asbestos etc;
- Aerodrome – potential contamination associated with fuel storage, workshop and maintenance;
- Effluent Irrigation Areas - potential microbiological, nutrient and heavy metal impacts;
- Poultry Farms – potential for microbiological, nutrient, heavy metals and pesticide impact to soil and groundwater from sheds, manure and waste management practices (including burial pits);
- Former Orchards – potential pesticide and heavy metal impact.

Based on the results of the PCA the potential contamination issues at the site were likely to be limited/localised. Additional detailed assessment was recommended to further assess these issues. However, as per the geotechnical issues they could be readily addressed prior to and/or during development through appropriate investigation, design and remedial measures (where required).

The UCA undertaken within the Cooranbong site identified a number of geotechnical and contamination issues that would require consideration prior to development. Recommendations for additional investigations were provided, including subsurface investigation for site classification, pavement design, detailed contamination assessment and specific management plans (ie. remedial action plan) to address objectives, methods and procedures for remediation.

Based on the results of the above UCA assessment, the site was considered suitable for future urban/residential development, subject to detailed investigation and localised remedial measures being undertaken. It is understood that the rezoning application was submitted by the developer and has subsequently been accepted by Lake Macquarie City Council.

## 4. CASE STUDY - CLIFTLEIGH

Cliftleigh is located 30 km north west of Newcastle between Maitland and Kurri Kurri. The Cliftleigh site (Figure 2) has been identified as one of the key future residential development areas of the lower Hunter Valley. The site is located within a primarily rural environment on the fringe of local urban areas. The site covers approximately 130 ha of former farming land and is situated about 10 mins from Maitland and Kurri Kurri.

The UCA (Douglas Partners Pty Ltd, 2005) was conducted for the developer to identify possible opportunities and constraints to future urban development, prior to the submission of a concept development plan to Council.

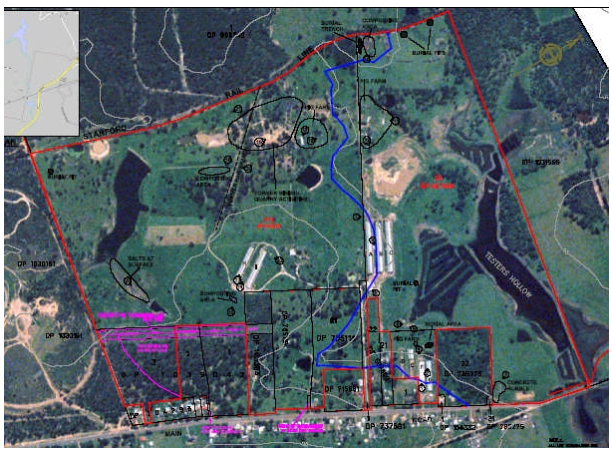

Figure 2: Cliftleigh – Site Features

The investigation comprised a desktop study including discussions with Department of Mineral Resources (DMR), Department of Natural Resources (DNR) and Department of Environment & Conservation (DEC), review of site history, site inspection, subsurface investigation (bores & pits), soil, surface water and groundwater sampling, laboratory analysis, preliminary microbiological assessment and reporting. A preliminary mine subsidence investigation was also carried out for the site, due to the presence of underground coal mining in the area.

The desktop review indicated the following:

- The site lies near the boundary of the Permian Aged Maitland and Dalwood Groups and the Great Coal Measures, which typically comprise sandstone, conglomerate, siltstone and coal, and Quaternary aged alluvium comprising sand, silt, clay and gravel. Subsurface investigation indicated that Quaternary alluvium is present in the northern part of the site around Testers Hollow, while the Greta Coal Measures underlie the majority of the site. Part of the site is underlain by underground coal mine workings;
- The Newcastle Soil Landscape Map indicates Testers Hollow is within the Wallis Creek Landscape, which comprises level to gently undulating floodplains up to 1000 m wide, deep (>2 m) Quaternary alluvial well to imperfectly drained soils, with limitations including flooding, high water tables, high run-on, high stream bank erosion hazard, groundwater pollution hazard due to high water tables and soils that are non-cohesive and of low fertility. The remainder of the site is within the Heddon Greta or Bolwarra Heights Landscapes which comprise low rolling hills or undulating aeolian sands over Permian Aged bedrock, slopes between 2% to 20% and limitations including high water erosion hazard, localised run-on, seasonal waterlogging, wind erosion hazard, localised non-cohesive and shallow soils, strongly acidic, low fertility soils, moderate foundation hazard and steep slopes with mass movement hazards;
- Topography is dominated by two knolls and a saddle in the south central area of the site acting as a watershed. Surface waters to the north of the saddle flow towards Testers Hollow, while those to the south flow to an off-site dam. There are a number of on-site farm dams and Testers Hollow has been partially dammed.
- The Beresfield acid sulphate soil risk maps indicate the southern area of the site is within an area of no known occurrence of acid sulphate soils, whereas the Testers Hollow area and the area to the west of Testers Hollow is within an area of high probability of acid sulphate soils within 1 m of the ground surface, and an area of low probability between 1 m to 3 m below the ground surface.
- The site history review indicated the site has generally been used for agricultural purposes with the exception of the Ayrfield colliery at the rear of the site. The poultry farming was undertaken at the site from the 1960's until recently. Poultry carcasses were originally buried in pits, but were later composted at the site. Two small piggeries were also present at the site and some minor cattle grazing and cropping was also undertaken. The mining of coal from the colliery on-site was undertaken between 1910 and 1938. From the late 1920's the mine was repeatedly flooded from the nearby Testers Hollow which slowed production significantly. The mine had two tunnel entrances and an airshaft. The mining operation was generally done by bored and pillar methods, however some small areas of complete extraction were also present. Following closure, the adits were backfilled with buses, trucks, engine blocks and the entrance was sealed with an explosive charge.

A site inspection identified the following predominant site features - former and existing poultry sheds, scattered equipment/building materials around former sheds, localised filling/dumping of demolition waste, former poultry burial areas, above ground diesel tanks, storage sheds, composting area, pig farm, farm dams, Testers Hollow, salt scalding and associated stressed vegetation, gravel quarry, 44 gallon drums & associated hydrocarbon impact, remnants of the former Ayrfield No 1 Colliery including some concrete/brick foundations, rail branch track and siding, and coal and soil waste stockpiles.

Based on the results of the desktop review, site history and site inspection, the principal sources of potential contamination were considered to be:

- Former chicken and pig farm areas (including former burial and composting areas) – potential hydrocarbon, metal, microbiological, nutrient and pesticide impact;
- Former Ayrfield No 1 Colliery area and quarry – potential hydrocarbon and metals impact;
- Imported/dumped materials and demolition waste – potential hydrocarbons, metals, asbestos, pesticides, PCB impact;
- General surface soils possibly impacted by former agricultural activities – potential hydrocarbon, metal, pesticide impact;
- Surface waters impacted by former land use, acid sulphate soils, salinity and adjacent land use (ie Testers Hollow) – potential nutrient, pH, EC, microbiological, hydrocarbon, metal impact;
- Former/current site structures – potential asbestos and metals.

Preliminary soil, surface water and groundwater sampling and laboratory analysis was undertaken to provide a preliminary assessment of both potential contamination and geotechnical issues identified at the site. An assessment of the site was also undertaken by a consulting microbiologist to assist in the assessment of the microbiological risk at the site.

The results of the assessment identified poultry burial areas and surficial composting of poultry litter at localised areas within the site. Some localised nutrient and microbiological impact was identified within these areas. Localised

remedial work would be required prior to development to address the malodorous material, microbiological and nutrients impacts.

The water quality assessment was undertaken in two stages. Initially a general assessment of surface and groundwater conditions was undertaken to see if there were any major contamination issues. Following the results of the initial assessment, further investigation was undertaken to assess the impacts of acid mine drainage from the Glen Ayr colliery located to the north of the site.

The initial water quality assessment comprised both insitu and laboratory testing for chemical, physical and microbiological contaminants from farm dams, Testers Hollow and groundwater. The visual amenity of the dams was also taken into consideration.

The initial results of testing generally indicated that surface water and groundwater were in good health with some minor impacts from site activities. It was recommended that appropriate management of surface waters be undertaken during the development.

Saline waters were identified within the large farm dam in the south of the site and within Testers Hollow. The saline conditions were consistent with the DNR identified saline zones. Testers Hollow was also identified as being acidic and saline which is likely to be attributed to the impacts of Acid Mine Drainage (AMD) migrating to the site from the adjacent Glen Ayr Colliery and also underlying saline soils. Further assessment of the long term impacts and management issues associated with Testers Hollow was recommended.

The results of the UCA identified a number of geotechnical and environmental issues which needed to be considered for future development. The main issues which would need to be considered for the proposed development include:

- Pothole Subsidence Risk Exclusion Zone (PSREZ) over the western part of the site due to former coal mining activities;
- Presence of saline soils and acid sulphate soils;
- Poor bulk fill quality within alluvial soils;
- Surface water quality;
- AMD impacts to Testers Hollow;
- Localised environmental and aesthetic remedial works.

Based on the results of the UCA, and assessments by others (ie flora and fauna and archaeological) a proposed structure plan was produced by the developer for the development. The plan includes over 800 residential allotments, community living areas, and recreational and historical parks, and provides for the re-shaping of Testers Hollow. It is noted that the UCA was critical to the design of the proposed structure plan.

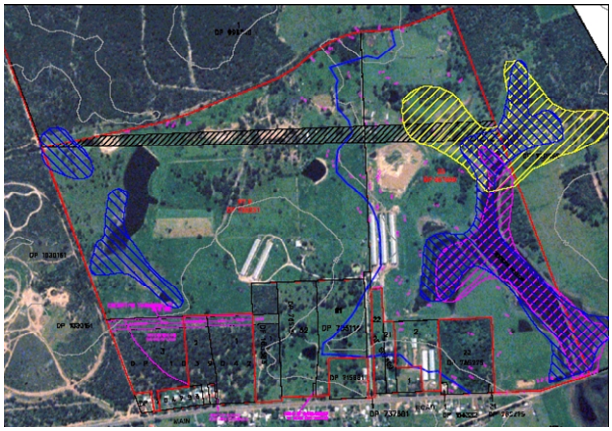

Figure 3 – Identified opportunities and constraints

It is understood that the concept plan has been submitted and approved by Council. The UCA has formed the basis for future detailed investigations, which will be required for design purposes.

## 5. CONCLUSIONS

UCAs provide a low-cost first phase assessment critical for both developers and Councils. They are an important tool in assessing opportunities and constraints to developments and are an imperative to establish well planned concept designs for new developments. They are used to maximize beneficial landuse and minimise development costs at the planning stage. The multidisciplinary approach utilised in UCAs can lead to significant savings to development costs, and result in an integrated development which considers the attributes of individual sites.

## 6. ACKNOWLEDGEMENTS

The author acknowledges the assistance and support of Chris Bozinovski (Douglas Partners Pty Ltd Senior Associate) and John Harvey (Douglas Partners Pty Ltd Principal) and the respective clients for providing challenging sites for Douglas Partners to assess.

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# DEVELOPMENT ON LOESS SOILS IN CANTERBURY, NEW ZEALAND

**Neil Charters**
*Golder Associates (New Zealand) Limited*
*Christchurch, New Zealand*

## ABSTRACT

Development on the Port Hills and Banks Peninsula, Canterbury, NZ provides varied challenges for engineering geologists and geotechnical engineers. The most common and most diverse of these is the loess soil which blankets the volcanic bedrock.

Loess is an aeolian deposit from large glacial river plains that existed during Pleistocene Ice Ages. It is highly dispersive and prone to both surface and subsurface erosion. Its strength is highly sensitive to moisture content changes and remoulding. The erodibility and strength of the material varies greatly between sites, and few correlations exist with any of the main geotechnical index properties.

Over the time since deposition, the loess has been subject to widespread slumping and mass movement. Many areas have subsurface erosion channels (tunnel gullies), and thick deposits of slopewash material are present on lower slopes. The tunnel gullies tend to be clustered.

These soil properties present the engineering geologist with some unique challenges in both investigation and design phases as well as posing many constraints to development. Remedial measures are discussed, and generally involve removing or reducing water ingress into the slope to provide adequate performance for both slope stability and foundations.

## 1. INTRODUCTION

This paper summarises problems associated with development on dispersive soils, particularly Banks Peninsula Loess, and provides the practitioner with some guidance on possible investigation methods as well as design or mitigation solutions. It summarises research carried out by the Author.

Loess soil is defined as “any fine textured deposit of Aeolian origin other than sand dunes of tephra” (Raeside, 1964). The engineering properties of Banks Peninsula loess are described in Section 4.

The paper describes the geological setting of the Banks Peninsula, Canterbury, South Island of NZ, then describes the material properties of loess. Geotechnical investigation methods suitable for loess sites are reviewed, and constraints to development and potential mitigation measures are summarised. The development on the Banks Peninsula is largely residential housing, typically one or two storey buildings.

## 2. TUNNEL GULLIES

Tunnel gullies are the main problem encountered in Banks Peninsula Loess as a result of its dispersive nature. They tend to lead to minor infrastructure damage and inconvenience to landowners rather than serious injury or major damage to infrastructure. Tunnel gullies are subsurface erosion channels that form as a result of water ingress and concentration on layers in the loess. Figure 1 shows the stages in development of shallow tunnel gullies in a typical layered loess soil. The soil layers referred to differ slightly from the terminology presented in this paper, but the layers are consistent between terminologies.

The tunnels can extend for up to a couple of hundred metres in a downhill direction, and are usually between 1 and 5 m across. They usually form in the middle of three layers described within the loess, usually between 0.5 and 2 m depth. There are often a number of tunnel gully features on a site, but commonly only one of the gullies is actively eroding at any one time.

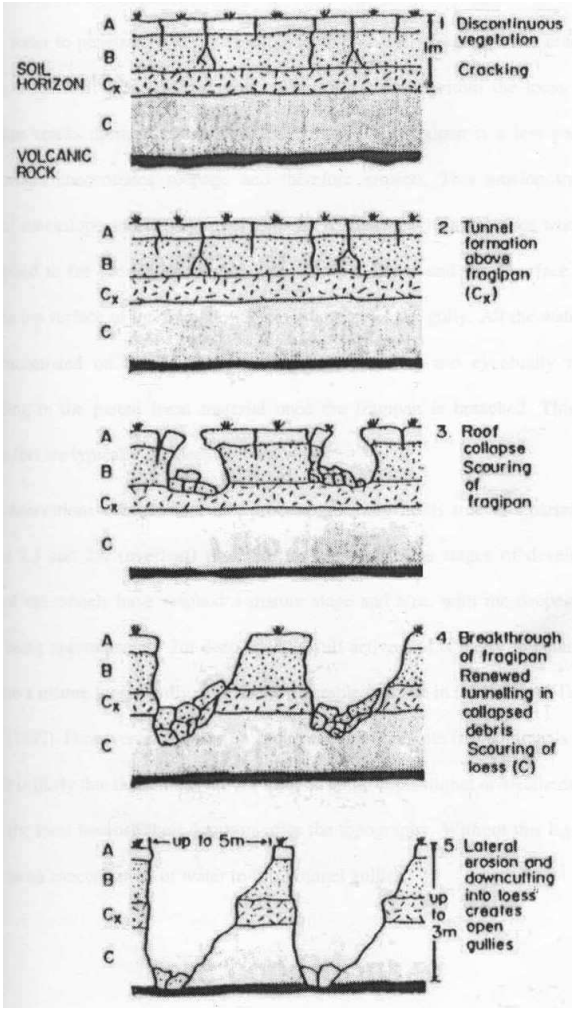


Figure 1. Stages in Tunnel Gully development (from Bell and Trangmar, 1987)

## 3. LOCATION AND GEOLOGY

Banks Peninsula is located on the east coast of the South Island of New Zealand (Figure 2). The Port Hills are those hills closest to the city of Christchurch and have experienced major housing and associated infrastructure development in the recent past. Banks Peninsula is an extinct volcanic complex consisting of two main vents and is surrounded by gravel of the Canterbury Plains. The Canterbury Plains extend approximately 100 km to the Southern Alps. The plains are formed of gravel from the rivers draining the Southern Alps. The plains and riverbeds downstream of large glaciers present during the Pleistocene ice ages had large expanses of dry silt and clay. It is this material that was blown across the plains and deposited on the Banks Peninsula and Port Hills to form an extensive cover of Loess.

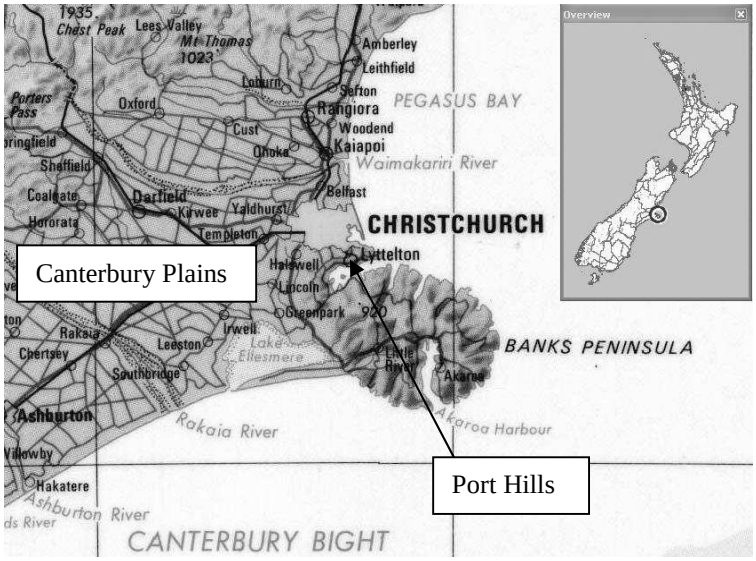


Figure 2. Location Plan (from TopoMap NZ)

## 4. MATERIAL PROPERTIES

Loess soil is variably distributed over the Port Hills and Banks Peninsula. In places the depth to bedrock can be as much as 15 m. The depth to bedrock generally reduces upslope, with only very thin pockets of loess encountered on hilltops (Bell and Trangmar, 1987).

Loess soils on the Banks Peninsula are generally layered with three distinct layers visible (Hughes, 1972 introduced the terminology). There is usually a surface (S) layer which consists of topsoil and sandy silt, grading into a compact (C) layer of moderate plasticity clayey silt (often with minor-some sand) and a parent (P) layer consisting of non plastic silt (often with some clay and minor-some sand). Figure 5 shows typical engineering properties of the three main layers within the loess. There are also commonly paleosols within the loess, which can further concentrate water flows, and which indicate the deposition was an episodic process. Evans (1977) and Bell and Trangmar (1987) worked on the development controls on tunnel gullies, and both concluded that the major controls were the shrinkage of the soils when dry, the layering, which concentrates water flows, and the climate. Heavy rain immediately after a dry period is the worst case, resulting in rapid infiltration into the soil through shrinkage cracks. Groundwater flows are concentrated in the C layer soil, immediately above the P layer interface (the fragipan in Figure 1). Figures 3 and 4 show two views of a tunnel gully: from the outside, and from the inside. Note the flat base caused by water flow shown in Figure 4.

Figure 3. Tunnel Gully

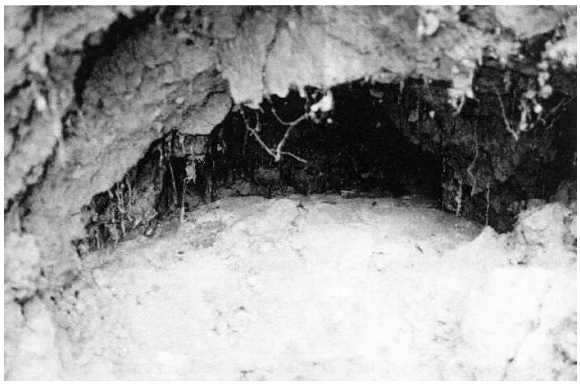

Figure 4. Inside a tunnel gully

Charters (2003) carried out a suite of laboratory tests on loess from an actively eroding site. This showed that the in situ loess is generally sandy SILT, but that samples taken from the base of actively eroding tunnels are generally silty SAND or sandy SILT with less sand. The mineralogy of the clay particles was also tested, and showed that the clay particles (generally less than 5 % of a loess sample) are typically 10-20 % clay minerals, with the majority being quartz or feldspar particles.

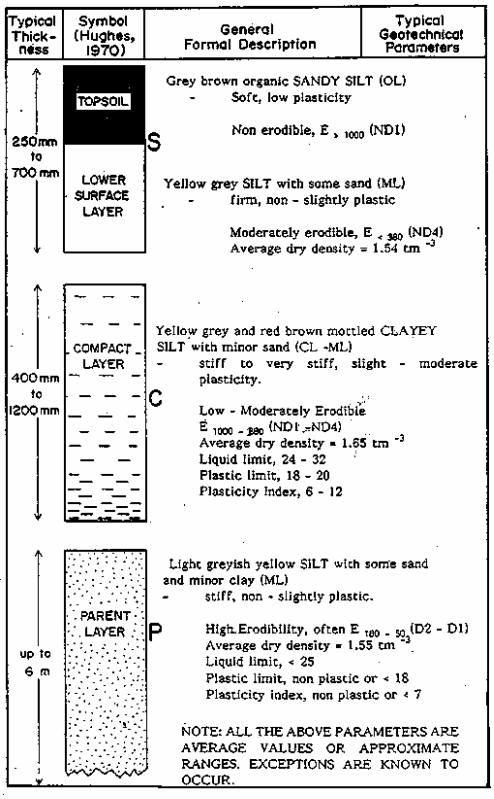


Figure 5. Typical Loess profile (from Yetton, 1990)

Loess exhibits high sensitivity to changes in moisture content. When dry, it can be very hard and crumbly, easily meeting the criteria for "good ground" in NZS 3604: 1999 for an allowable bearing capacity of 100 kPa. Typical engineering parameters for the soil have been determined by Goldwater et al. (1990), and show that the P and C layers have the same remoulded properties, with phi' ($\varphi$') of $28.5^0$ and no cohesion. This is reasonable on the basis that the mineralogy and grain size is the same for both layers. In situ properties vary slightly, with $\varphi$' of $29^0$ for the C layer and $31^0$ for the P layer, again with no cohesion. There is, however, a significant amount of strength from apparent cohesion (due to soil suction) or cementation, of up to 30 kPa, although this varied between sites, and cannot be relied upon in design. Figure 6 shows how steep loess cuts can be made and remain stable albeit with significant rilling. This was observed adjacent to a minor local road, and probably was not subject to a rigorous design process. When it becomes wet, it is highly prone to erosion as well as being significantly weaker due to loss of cementation and apparent cohesion. This contrast in strength does tend to make for relatively easy construction provided it is done during a dry period.

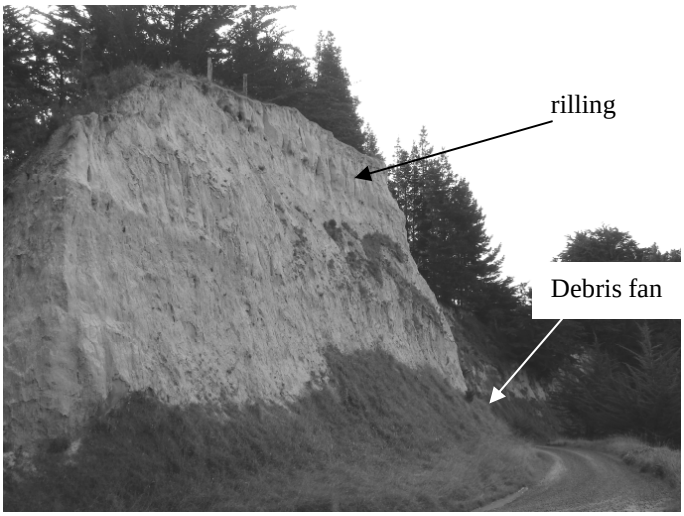


Figure 6. Steep road cut in loess

## 5. GEOTECHNICAL INVESTIGATIONS

Site specific geotechnical investigations are necessary to define the hazard posed to a site by tunnel gullies and subsurface erosion. Air photograph interpretation and detailed engineering geological mapping are the preferred options, although air photos must be very detailed to show individual tunnel gullies as they can be very small features. Ideally, inspections should also be carried out during wet conditions to allow assessment of tunnel activity to be undertaken. Realistically, this is limited by project timeframes and the project deadlines. The grain size of the material can be used as an indication of tunnel gully activity, where the silt is washed away leaving sandier material in the base of an active tunnel.

In addition to these non-invasive mapping methods, there are a number of subsurface methods. The usual options of test pits and hand augers are possible, although test pits can be difficult in a residential setting due to access difficulties for large excavators. Hand augers also have limitations due to the hard nature of dry loess. The material tends to shave rather than fill the auger head. It is also possible to use a scala penetrometer to probe for cavities, although this technique is more useful for tracing known gully features rather than locating new ones. Gullies can also be traced using smoke or dye techniques, which involve placing smoke or dye into the inlet of one tunnel and observing where it exits. The dye method has limitations because the water flowing from tunnels tends to be more of a slurry than clear water so can be difficult to dye.

## 6. GEOTECHNICAL CONSTRAINTS TO DEVELOPMENT

Tunnel gullies pose a range of geotechnical constraints to development, including slope instability, undermining of foundations, and damage to infrastructure including buried services and roads.

There are two types of hazard posed by tunnel gullies: slope instability and erosion. The concentration of water flows clearly has significant impact on slope stability. Shallow slope failures are relatively commonplace on undeveloped farmland during heavy rain periods. This has very little significant impact, but once the land becomes developed for housing, shallow slope failures are a more significant hazard. Erosion of the land poses a hazard to foundations, and can undermine shallow foundations and cause severe damage (Charters, 2003). Either slope instability or erosion of the land can cause major damage to houses, roads and associated infrastructure due to loss of foundation support.

Subdivision and building is regulated in New Zealand by the Resource Management Act (RMA, 1991). The relevant requirements under this legislation are that the land in question is not subject to slippage and erosion, and that development will not increase the risk of erosion or slippage on neighbouring properties.

## 7. MITIGATION MEASURES

Typical mitigation measures are centred on water management. This can involve preventing water from entering the site (both surface and subsurface) and controlling water on the site. Subsoil drains and surface interception drains can be placed on the uphill side of the site. Figure 7 shows a typical design detail for a combined subsoil and surface drain. Water management on the site includes piping stormwater and runoff from roofs and paved areas to council approved discharge sites or into the council stormwater network for treatment and disposal off site. The RMA requirements for not increasing risk to neighbouring sites make it particularly important to exercise care when disposing of stormwater.

In addition to the mitigation above, there are a number of construction-related methods that can be applied to the loess. All service trenches should be carefully backfilled to ensure they do not become conduits for water and cause erosion. This can be achieved by using loess mixed with approximately 3 % lime to reduce erodibility and permeability (Bell and Trangmar, 1987). Similarly, all fills should use lime stabilised loess. Any tunnel gullies exposed during site works should also be treated in a similar way. They may need subsoil drainage material and a pipe beneath the lime stabilised loess.

Any retaining walls should have drainage metal or fabrics behind them to ensure the material remains dry and so retains its strength. Similarly, foundation trenches should not be exposed for any longer than necessary to avoid ponding water reducing material strength. If necessary, they can be excavated to 200 mm above final level and then cut out immediately prior to concrete placement. Avoiding construction work during the wet months over winter also reduces the risk of water softening.

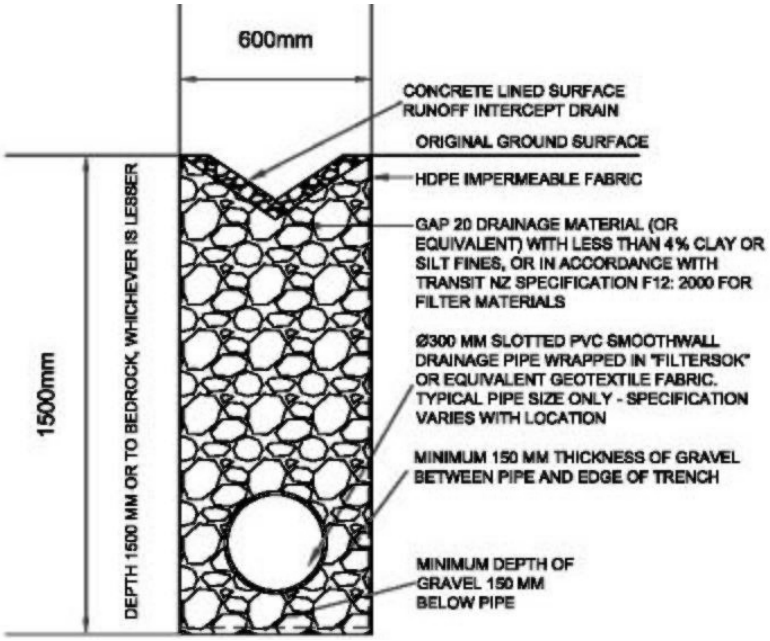


Figure 6. Typical subsoil drain detail

## 8. CONCLUSIONS

Whilst development on the dispersive loess soils on the Banks Peninsula provides challenges for engineering geologists, it is possible provided detailed investigation and mapping is carried out to identify all potential constraints, and these constraints are then remedied. Mitigation is generally focused around removing or reducing water ingress into the slope, with both surface and groundwater management. The hazards and mitigation measures discussed in this paper apply not only to new developments, but also to existing properties built on loess. In addition to designing suitable mitigation measures for new properties, the effects of new developments on neighbouring land must be considered under the RMA in New Zealand.

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# GEOTECHNICAL ASPECTS OF THE UPPER HARBOUR BRIDGE DUPLICATION AND CAUSEWAY WIDENING

**Lucy Coe**
*Beca Infrastructure Ltd*
*Auckland, New Zealand*

## ABSTRACT

The Upper Harbour Bridge Duplication and Causeway Widening Project has recently been constructed for Transit New Zealand to widen State Highway 18 between Greenhithe and Hobsonville in Auckland, New Zealand. The project involved construction of a second harbour crossing, adjacent to the existing Upper Harbour Bridge, and widening of the existing causeway. This project was completed under a Design and Build contract, with Fletcher Construction Ltd as the Contractor and Beca Hollings and Ferner Ltd as the Designer.

The existing causeway has been widened over the seabed by 20m and extends 850m west of the new bridge abutment. The new widening is comprised of a rock bund combined with a shear key and shoulder constructed using cement stabilised marine sediments (mudcrete) and sand fill to meet both design requirements and construction methodology. Key geotechnical issues include the presence of weak marine and alluvial deposits, slope stability and long-term settlements.

The second bridge structure spans approximately 460m across an incised harbour inlet formed within the Waitemata Group siltstone and sandstone. The bridge foundations consist of concrete bored piles constructed in groups of two to four at each pier. The piles are founded within the underlying rock encountered between 1.5 and 11m below seabed.

This paper outlines the geotechnical design aspects for the causeway widening. A discussion is presented on the materials and methods utilised during construction, in particular the causeway and use of excavated marine sediments to form mudcrete.

## 1. INTRODUCTION

The Upper Harbour Bridge (UHB) Duplication and Causeway Widening Project was constructed for Transit New Zealand between 2003 and 2006. The widening of State Highway 18 (SH18) between Greenhithe and Hobsonville forms part of the proposed motorway linking Albany Highway with SH16 in Auckland, New Zealand. The project works have been completed under a Design and Build contract with Fletcher Construction Ltd (FCC) as the Contractor and Beca Hollings and Ferner Ltd (Beca) as the Designer.

This project consists of two main elements: construction of a second Upper Waitemata Harbour crossing, adjacent to the existing Greenhithe Bridge, and widening of the existing causeway to form the western approach for the new crossing. The new bridge provides three east bound lanes, including a passing lane, and a designated cycle and pedestrian lane.

The existing 850m long causeway has been widened by up to 20m over weak marine sediments and alluvial deposits. The widening has been constructed using cement marine stabilised sediments (mudcrete) to form a shear key and outer facing, combined with rock and sand fill confined within the embankment. The construction below water level was an alternative design proposed by FCC and Beca at the time of tender.

## 2. SITE DESCRIPTION

### 2.1 SETTING

The site is located on SH18 between Greenhithe and Hobsonville, crossing the Upper Waitemata Harbour to the north of the existing bridge and causeway. The bridge crosses an incised inlet of the Waitemata Harbour formed following a post-glacial rise in sea level.

The approach causeway is situated in a sensitive coastal environment within the inter-tidal zone and has been extended over areas of mangroves and mudflats. The seabed slopes gently out from the existing causeway at approximately 5 degrees to the north and south. The new causeway is a maximum height of 5m above the seabed at the western abutment where the seabed dips steeply towards the channel. The bridge spans across the navigation channel with average water depths of 8 to 10m.

The Geological Map of Auckland 1:50,000 (Kermode, 1992) indicates the bedrock is Waitemata Group siltstone and sandstone of the Miocene aged East Coast Bays formation. The siltstone and sandstone is overlain with Pleistocene aged Tauranga Group alluvium and Holocene aged marine and estuarine sediments.

### 2.2 SOIL PROFILE

The site investigations indicated the soil profile typically comprises recent marine sediments and Tauranga Group alluvium, overlying a weathered Waitemata Group profile ranging from residual soils to slightly weathered soft Tertiary rock. Geotechnical field investigations were undertaken along the causeway and bridge alignment during a previous phase of the project, and included a series of machine boreholes, test pits, hand augers and laboratory testing performed on collected samples (Meritic, 2002a and Meritic, 2002b).

The existing causeway is comprised of silt and clay fill of variable strength ranging from a soft fill directly over the original seabed to very stiff close to the pavement. The recent marine and estuarine sediments are a combination of very soft silts and clays, and loose sands. These sediments are up to 2m thick and are located on either side of the causeway and under the existing footprint. The Tauranga Group alluvium layer is up to 3m thick and consists of firm to stiff clays and silts, and loose to medium dense sands. The underlying Waitemata Group weathered profile consists of up to 4.5m of residual soils over slightly weathered to unweathered rock. The residual soils are stiff to very stiff silts and clays with medium dense to dense silty sands. The very weak to weak siltstone and sandstone lies approximately 1.5 to 5.0m below the seabed along the causeway alignment.

## 3. GEOTECHNICAL DESIGN OF THE CAUSEWAY WIDENING

The design of the causeway widening adopted for construction is based on an alternative design proposed by FCC and Beca at the time of tender. The alternative design utilises mudcrete and locally available materials to replace large quantities of rock fill and large boulders required in the conforming design. The design was developed to suit construction methods and improve design standards.

The causeway widening design involved construction of a shear key into underlying stiff materials, a rock fill bund with a mudcrete outer facing and sand fill at the rear of the widening. Geotextile and armour rock were placed on the seaward face of the widening for erosion protection. A typical cross-section for the causeway widening is presented in Figure 1.

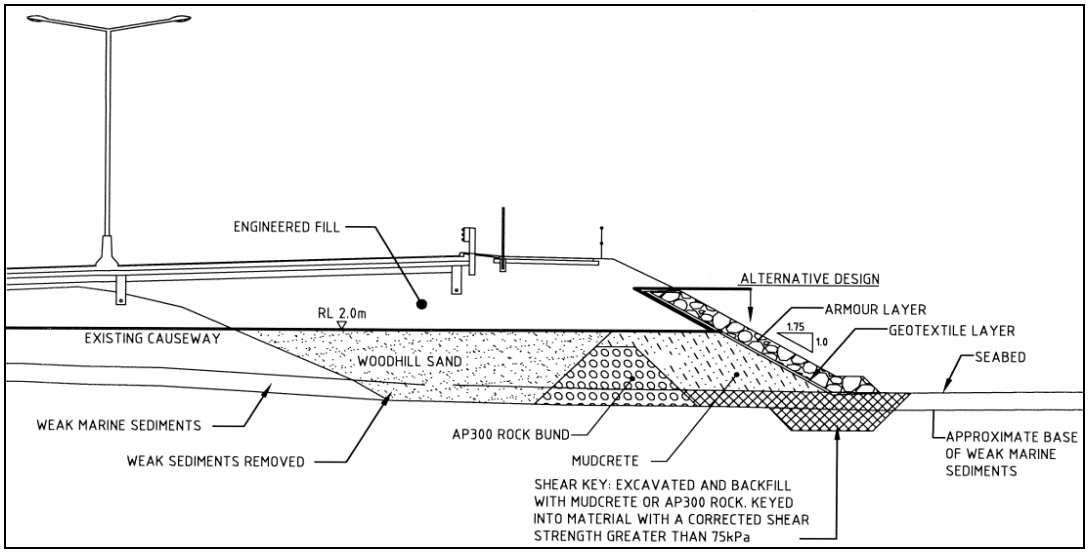


Figure 1: Typical Causeway Widening Cross-Section

### 3.1 CONFORMING DESIGN COMPARISION

The conforming design proposed utilising granular rock fill below RL2.0m and cohesive fill above this level (Opus and Meritic, 2002). The design involved two treatment concepts depending on the depth of marine sediments encountered as detailed below. Both treatments were combined with a shear key constructed using rock fill and keyed into the underlying Tauranga alluvium and Waitemata Group residual soils. The conforming design treatments are as follows:

1. For marine sediments less than 1.0m thick, large boulders greater than 1.0m were to be punched into the marine sediments and AP300 rock fill was to be placed above this level up to RL2.0m.
2. For marine sediments of thicknesses greater than 1.0m, the sediments were to be excavated and replaced with AP300 granular fill to RL2.0m.

The alternative design philosophy was considered to have the following advantages over the conforming design:

- The requirement for large boulders that are relatively scarce in the Auckland region is eliminated.
- The quantity of AP300 rock fill required below the water level is reduced.
- All marine sediments excavated are re-used in causeway construction eliminating the requirements associated with transportation and disposal of contaminated sediments offsite.
- The use of mudcrete increases the stability and reduces the risk of long-term settlements of the causeway. In addition steeper side slopes were used, therefore minimising the widening footprint.

### 3.2 KEY DESIGN AND CONSTRUCTION CONCEPTS

The key design issues for the causeway widening, with respect to the presence of weak marine and alluvial deposits, were:

1. The stability of the causeway, including performance under static and seismic load cases;
2. The potential for long-term settlement of the causeway and the differential movement relative to the existing causeway and new bridge structure, including the impact on services, ride quality and long-term maintenance requirements; and
3. The constructability of the causeway widening.

The design principles adopted to address these issues are described below.

#### 3.2.1 Removal of Soft Marine Sediments

All soft to very soft marine sediments with a corrected shear strength of less than 25kPa were removed from beneath the widening footprint. These sediments were removed to improve stability of the causeway and reduce consolidation settlements.

Pilcon Shear Vane testing was undertaken within the footprint at 10m intervals along the length of the causeway during design. This testing provided a depth versus strength profile of the underlying marine sediments and alluvial deposits for design, and identified the excavation depth required for the shear key. This testing generally indicated the underlying material had a corrected shear strength of at least 75kPa between 0.25 and 1.0m below the seabed. An infilled channel running across the causeway alignment was identified with over 3.0m of very weak sediments.

#### 3.2.2 Granular Fill Below the Water Table

Granular fill was used below the water table to enable compaction requirements to be met. This included the use of AP300 quarried rock to construct a bund behind the mudcrete face and Woodhill sand, a fine uniformly graded sand, to fill between the rock bund and existing causeway. Limiting the quantity of rock fill and utilising locally sourced sand reduced the cost of materials and transportation. The rock bund is required for the stability of the causeway, and was also utilised by the Contractor as a working platform during construction. The sand is confined between the existing causeway and the rock bund with a mudcrete facing to prevent migration and potential piping failures. The sand was compacted to a medium dense standard to reduce the potential liquefaction risk under the 1000 year return period earthquake event. The compaction standard was generally achieved by track rolling and tidal action and confirmed using Scala Penetrometer Tests.

#### 3.2.3 Shear Key

A shear key was excavated at the toe of the widening and backfilled with sections of mudcrete and AP300 rock fill. The shear key extends at least 0.5m into the underlying stiff alluvial deposits and residual soils with a minimum corrected shear strength of 75kPa. The shear key assists with the overall stability of the causeway widening and allows minimum

stability design standards to be achieved. The design provided flexibility to the Contractor by allowing mudcrete and rock fill to be generally interchangeable for the shear key backfill. Rock fill material was utilised when the marine sediments for mudcrete were not available.

### 3.2.4 Mudcrete

Mudcrete was used as the outer facing of the causeway bund and within the shear key. Cement was mixed onsite with excavated silts, clays and sands to form mudcrete, a relatively lightweight, high strength, cohesive fill product that does not require compaction. The strength of mudcrete improves the stability of the causeway and enabled the face to be constructed at a steeper angle reducing the causeway widening footprint. The lightweight nature of the fill is expected to reduce the long-term consolidation settlements of the causeway.

The marine sediments are contaminated with heavy metals from the harbour and untreated stormwater run off from the existing causeway. The use of mudcrete eliminated the requirements for storage, transportation and disposal of these contaminated, saturated sediments. Cement stabilisation binds contaminants such as DDT, lead, zinc, organics and TP4 into the product. As such the use of mudcrete is considered an environmental enhancement for this sensitive coastal setting.

### 3.2.5 Construction Program

The construction program was developed to reduce the post-construction settlements. The causeway embankment adjacent to the new western bridge abutment was constructed early to allow the majority of settlement to occur prior to construction of the abutment structure and piles. The paving along the causeway was delayed until the end of the project to allow the causeway to be re-levelled and reduce the long-term settlement effects on ride quality and on-going maintenance.

## 3.3 DESIGN STABILITY ANALYSES

The causeway was analysed at critical locations along the alignment based on a combination of the fill height and underlying soil conditions. The stability analyses were undertaken using the SLOPE/W software package to estimate the factor of safety (FOS) for different potential failure modes. The analyses were based on the Limit Equilibrium Theory utilising the Morgenstern-Price analysis method.

Soil strength parameters adopted for design were based on available field and laboratory testing from the site, and previous experience with these geological units. For the design mudcrete has been conservatively modelled with a shear strength of 150kPa and bulk density of 1.40 t/m$^3$. The mudcrete field strengths are described in Section 4.

The load cases modelled and minimum FOS against instability adopted for design are summarised below.

1. A short-term construction case modelled with total stress parameters and construction traffic loads requiring a FOS of 1.3.
2. A long-term construction case modelled with effective stress parameters and Transit New Zealand traffic loads, requiring a FOS of 1.5.
3. A seismic construction case modelled with total stress parameters and a seismic acceleration of 0.11g, requiring a FOS of 1.1. The seismic acceleration was based on the 1000 year return period earthquake event determined from a site specific hazard assessment (Engineering Geology Ltd, 2002) and was modelled using a pseudo-static approach.

Sensitivity analyses were carried out to determine the effect of changes in the design assumptions, including changes in soil strength parameters, materials, tidal levels and the rock bund profile. The analyses demonstrated that AP300 and mudcrete could be interchanged for sections of the causeway to provide flexibility for the Contractor.

# 4. MUDCRETE

Mudcrete is a relatively high strength, lightweight fill material produced by stabilising weak marine sediments with cement. The variability of the product is determined by both the construction methodology and variability of sediments (i.e. silt and clay contents). The field mixing techniques utilised for the causeway produced highly variable strength results, however a minimum strength can be specified and achieved in a reliable manner (albeit with zones much stronger than required).

Beca originally developed mudcrete for the reclamation constructed at the Auckland Viaduct for the Whitbread Round the World Race in 1994. Mudcrete has subsequently been utilised for the Americas Cup Village extension to the Auckland Viaduct and is currently being used for the Ports of Auckland Container Terminal Extension.

In addition to the causeway construction, the mudcrete was utilised for the stormwater retention ponds designed by the Client's Engineer, temporary mixing basins for mudcrete and to encase the double stormwater culvert that was extended under the new causeway widening.

## 4.1 MATERIAL CHARACTERISTICS

Mudcrete was utilised as a structural fill with specific requirements including a minimum Unconfined Compressive Strength (UCS) of 90kPa at 7 days, a minimum UCS of 200kPa at 28 days and an average UCS greater than 400kPa. Trial stabilisation batches were produced prior to construction to confirm the mixing technique and cement ratio.

The mudcrete was batched with the minimum 80kg/m$^3$ cement ratio for the majority of the causeway widening and generally achieved all specified strength requirements. As construction progressed a 65kg/m$^3$ mix was trialled and also achieved the required strength. The 28 day strength results from the northern causeway widening are presented in Table 1. These results indicate a large variability in mudcrete strength produced, with the UCS ranging from 114 to 3670kPa. This variability is a result of the uncontrolled nature of the sediments and water content, combined with the mixing technique.

| Properties | Bulk Density (t/m$^3$) | Moisture Content (%) | Dry Density (t/m$^3$) | 28 Day UCS (kPa) | | Young's Modulus (MPa) | |
|---|---|---|---|---|---|---|---|
| | | | | 65 kg/m$^3$ of Cement | 80 kg/m$^3$ of Cement | 65 kg/m$^3$ of Cement | 80 kg/m$^3$ of Cement |
| Average | 1.75 | 37.0 | 1.29 | 668 | 1460 | 273 | 542 |
| Range | 1.53 – 1.92 | 27.0 – 57.7 | 0.97 – 1.51 | 459 – 945 | 114 – 3670 | 51 – 459 | 22.6 – 1370 |
| Standard Deviation | 0.08 | 8.04 | 0.12 | 170 | 870 | 126 | 322 |
| All results based on 28 day UCS samples using standard 21 degree cure.<br>Bulk density, moisture content and dry density based on both 65 kg/m$^3$ and 80 kg/m$^3$ cement content samples.<br>Results based on ten 65kg/m$^3$ cement content samples and fifty five 80 kg/m$^3$ cement content samples. | | | | | | | |

Table 1: Northern Causeway Mudcrete Field Results

## 4.2 CONSTRUCTION METHODOLOGY

Mudcrete was produced by mixing sediments and cement within small on site mixing basins and insitu within the shear key trench excavation, creating 30 to 50m$^3$ per batch. The methodology adopted for mixing required little additional plant and resources, utilising hydraulic excavators to mix the product. The mixing basins were located in close proximity to the current work faces to improve efficiency and minimise haulage.

The sediments were placed within the mixing basin or excavated key trench with the volume accurately recorded. Prior to mixing, sediments were checked for consistency and large clumps of clay were premixed or removed. The cement quantity was calculated and spread over the surface of the sediments. The cement was tilled in with a hydraulic mixing head attached to an excavator; the mixing head is shown in Figure 2. Each batch was mixed until a uniform consistency was achieved. Mudcrete batched in mixing basins was transported to the placing excavator and placed at low tide within an hour of batching. The surface of the mudcrete was shaped using the back of the excavator bucket. The mudcrete generally gained enough strength for people to walk on the surface within a few hours and to traffic vehicles within one or two days. This allowed construction to proceed quickly. The cohesive nature of mudcrete prevented it from being washed away as the tide rose. In order to achieve an adequate bond between subsequent layers, the hardened surface of the mudcrete was roughened with a toothed excavator bucket prior to application of the next layer.

The water content of the sediments affects the strength of the final stabilised product, however it is not practical to control the water content. The construction methodology adopted the following steps for working within the tidal environment and reducing the water content of the sediments.

- No additional water was added to the mudcrete mix.
- The sediments were excavated at low tide and stored behind a mudcrete or sand fill outside the extent of tidal influence. If this was not possible sediments were drained at low tide prior to mixing or premixed with drier excavated materials to reduce the water.

- Mudcrete was generally placed at low tide or above the tide level to minimise the requirement to place it below water.
- The shear key constructed was generally excavated and either mudcrete placed or sediments stored within the key over the low tide cycle to prevent water becoming trapped within the shear key and to limit the risk of side collapse. Where the design required the shear key depths to increase, AP300 rock fill or insitu mixing was adopted to enable the key to be backfilled quickly.

Figure 2: Mudcrete Mixing Equipment

## 5. CONCLUSION

The UHB duplication and causeway widening project has been successfully designed by Beca and constructed by FCC, using an alternative causeway design philosophy. Key geotechnical issues addressed by the design include the presence of weak marine and alluvial deposits, slope stability and long-term settlements. The alternative design aimed to address these geotechnical issues and to improve constructability by utilising mudcrete for the shear key and outer facing, combined with a rock bund and locally sourced sand fill.

The use of mudcrete allowed excavated weak sediments to be re-used on site to produce a relatively lightweight, high strength fill and eliminated the requirement for disposal off site. The mudcrete was batched on site using small temporary mixing basins with a mixing head attached to a hydraulic excavator. All minimum strength requirements were achieved in the field, however the mudcrete strengths produced were highly variable due to the uncontrolled nature of the sediments and mixing techniques adopted.

## 6. ACKNOWLEDGEMENTS

The writer would like to thank Nicola Ridgely and Dr Do Van Toan of Beca Infrastructure Ltd for help through out the project and the review of this paper, and Fletcher Construction Ltd and Transit New Zealand for permission to publish this paper.

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# CASE STUDY OF QUARRY REHABILITATION VALLEY LAKE, NIDDRIE

**S. Colls**
*Golder Associates Pty Ltd, Melbourne*

## ABSTRACT

The Valley Lake Project involved the rehabilitation of a former quarry for residential development. Prior to rehabilitation, the quarry was partially filled with stockpiles of waste rock and overburden materials. A lake had formed over the quarry base. Rehabilitation involved the removal and processing of approximately 2.4 million cubic metres of stockpiled fill for placement as engineered fill, the creation of a feature lake and stabilisation of existing quarry walls.

This case study focuses on the design, construction and performance monitoring of engineered fill up to 35 m thick composed primarily of well-graded, predominantly granular fill with a maximum particle size of 600 mm.

## 1. INTRODUCTION

Quarrying of basalt at Niddrie Quarry began in the late 1920's and finished in 1976. During this time the open paddocks adjacent to the quarry were developed as residential areas. When quarry operations ceased, stockpiles of overburden material were located within and adjacent to the quarry pit. The base of the quarry had typically been excavated below the regional water table level and a lake had formed in the pit.

The proposed redevelopment comprised a residential estate with low, medium and high density housing areas and associated infrastructure. Initial bulk earthworks comprised the excavation of all uncontrolled fill for sorting, blending, conditioning and placement as engineered fill over natural subgrade within the former pit. A feature lake and parts of the rock wall surrounding the former pit were to be retained.

The initial earthworks were carried out between March 2002 and December 2003. Following redesign of proposed fill levels a second stage of earthworks commenced in March 2006 and is ongoing. This paper refers to the initial bulk earthworks.

VicUrban, formerly the Urban and Regional Land Corporation (URLC), were the owner of the site. Golder Associates Pty Ltd (Golder Associates) were the principal geotechnical engineer for the development. Breese Pitt Dixon Pty Ltd (BPD) were responsible for the civil design, survey and were Superintendent of the works. BMD Major Projects were the principal earthworks contractor for the initial bulk earthworks.

## 2. SITE DESCRIPTION AND GROUND CONDITIONS

The site occupied approximately 47 hectares with maximum dimensions of about 800 m (north-south) by 700 m (east-west). The site was located approximately 15 km north west of the Melbourne CBD, close to Tullamarine and Essendon Airports and three major freeways. Residential developments surrounded the site.

Prior to commencement of the bulk earthworks, the quarry pit occupied about 11.5 hectares and was up to 40 m deep. A lake, fed predominantly by groundwater, was present in the base of the pit. Stockpiles containing approximately 2.4 million cubic metres of uncontrolled fill (to depths of up to 30 m) were located within and adjacent to the pit. Near-vertical rock batters of up to 40 m in height surrounded the pit. The majority of the site was covered in vegetation varying from grass to mature eucalypts.

The surface geology at the site consists of Quaternary age basalt overlying thin irregular Tertiary age sedimentary deposits which in turn overlie Silurian age siltstones and sandstones. Quaternary age alluvial flood plain deposits occur adjacent to Steele Creek.

The 40 m high batter along the western side of the quarry provided a visual profile of the basalt sequence, which comprised two main basalt flows. The basalt graded from ash like tuffs to weathered vesicular basalt to massive

columnar-jointed basalt with increasing depth. Up to about 3 m thickness of natural residual basaltic clays occurred over the basalt. The deeper Tertiary age and Silurian age sediments were not exposed at the site.

Site fill stockpiles typically comprised overburden materials excavated during quarrying operations. The overburden materials were derived from the residual soils, ash like tuffs and weathered vesicular basalt and comprised materials ranging from rock-free clay to large basalt boulders of about 2 m maximum diameter.

Figure 1: Former quarry pit early in construction showing western rock batter, lake and some fill stockpiles

## 3. SITE HISTORY AND PROPOSALS FOR REDEVELOPMENT

Quarrying of the massive columnar-jointed basalt took place from the late 1920's until 1976. Overburden material was dumped in and adjacent to the main pit. It is understood that a combination of increasing pressure from adjacent residential developments, the penetration of the basalt layer in parts of the pit and increased groundwater inflows led to the cessation of quarrying activities.

A number of site investigations were undertaken following the end of quarrying, including those by MPA Williams & Associates (1984 and 1989) and Golder Associates (1994 to 1998). These investigations were aimed primarily at characterising the nature and extent of stockpiled fill, the stability of the exposed rock faces and groundwater characteristics of the site. Proposed site uses anticipated using the stockpiled fill and/or imported landfill to backfill a significant proportion of the pit. The removal of fill stockpiles to expose natural ground outside the quarry pit was expected to release land for residential subdivision. Local council (Mooney Valley Council) were keen to ensure strict engineering control of any areas of filling that were to be released for residential development.

Residential opposition to expected construction traffic on local streets helped shape future development constraints on the site. When the URLC became owners of the site it had become clear that importing significant quantities of landfill was no longer possible. In 2000, Golder Associates began a further series of investigations aimed at outlining the options available to the URLC for developing the site.

## 4. GEOTECHNICAL ISSUES

The reports by Golder Associates and others highlighted a number of areas of potential concern relating to residential development of this site. They included:

- The presence of deep stockpiles of uncontrolled fill of variable nature in the base of the pit.
- The base of the pit being up to 9 m below the current groundwater level leading to the potential for problems associated with draining the existing lake, saturation of the uncontrolled fill stockpiles and placement of engineered fill below the groundwater level.
- The presence of rock faces of variable composition and stability of up to 40 m height.
- The proximity of the site to residential areas.

URLC aimed to create a landscaped residential development in the pit using only fill stockpiled onsite and aimed to release the filled land for sale within a year of the completion of earthworks. The URLC wanted site classifications for shallow residential footings of M or H (reclassified from Class P) on engineered fill placed at depths of up to 35 m. This requirement was based on achieving a characteristic surface movement (made up of components of settlement and shrink/swell of the engineered fill) of less than 70 mm beneath footings.

## 5. GEOTECHNICAL DESIGN

For generally inert fill that has been subjected to a reasonable level of compaction, the rate at which settlement occurs will generally diminish with time. The higher the relative compaction, the less settlement that will occur and the sooner the URLC's aims could be attained. In broad terms it was anticipated that a high portion of the settlement would occur during the construction period with the amount of settlement decreasing proportionally with the log of time. Provided that tight engineering control of the fill could be maintained long-term settlements in the order of 1 % to 1.5 % of the thickness of fill material were anticipated. Settlement monitoring of the fill was considered essential to understanding the performance of the placed materials and as such was expected to be an integral part of the filling process.

Some of the initial proposals for filling of the pit (prior to the URLC development) had included improvement of the existing uncontrolled fill in the quarry hole. It was considered that the unpredictability and likely large, ongoing settlements associated with attempting to compact this fill *in situ* would not meet the URLC's aims. Instead it was proposed to remove all uncontrolled fill within the site and replace it, following reprocessing, primarily as the structural fill types described in Table 1.

Table 1: Grading requirements of primary structural fill types

| Fill Type | Purpose of Fill | Maximum Particle Size | Intermediate grading limits (percentage passing sieves) |
|---|---|---|---|
| Type A | Final layer of structural fill in which shallow footings, roads and services are located. | 100 mm | 53 mm : 70 – 100<br>2.36 mm : 10 – 100<br>75 micron (silt/clay) : 0 - 15 |
| Type B | Structural fill placed between the quarry base and the base of the Type A fill. | 600 mm | 300 mm : 70 – 100<br>75 micron (silt/clay) : 0 – 15 |
| Type C | Placement in housing zones designated medium density where piling to the former quarry floor was the anticipated founding solution. | 200 mm | 53 mm : 70 – 100<br>2.36 mm : 10 – 100 |
| Coarse Rock Fill | Placement as the first metre of structural Type B fill in areas of wet quarry base. | 600 mm | 2.36 mm : 0 – 10 |

Fill types were designed to utilise the wide variety of fill expected within the site stockpiles. It was expected that the majority of stockpiled fill could be excavated and placed as Type B fill. Type C fill was designed for materials excessively clayey for Types A and B fill. The specification for Type C fill was developed in consultation with piling contractors to ensure that future use of piling would be possible. Performance criteria for the primary types of structural fill are described in Table 2 and below.

A non-structural fill, Type D, was designed to accommodate rock too large for Type B fill and stockpiled fill that was too wet for placement as structural fill. Type D fill was placed beneath the proposed feature lake to provide a relatively high permeability that would allow groundwater to rise and fill the lake following completion of the earthworks.

Table 2: Performance criteria of primary structural fill types

| Fill Type | Minimum Dry Density Ratio (Standard compaction) | Maximum Compacted Layer Thickness | Survey Control Frequency |
|---|---|---|---|
| Type A | 98.0 | 0.3 m | Every third layer |
| Type B | N/A | 1.0 m | Every layer |
| Type C | 95.0 | 0.5 m | Every second layer |
| Coarse Rock | N/A | 1.0 m | Every layer |

It was expected that Types A and C fill could be tested with conventional density testing methods. As Type B fill was too coarsely graded for conventional testing an approved method for placement was developed based on the results of compaction trials. Trials were conducted using fill won from onsite stockpiles and placed with variations in layer thickness and compactive effort. Performance of trial compaction layers was assessed visually and by surface survey. The approved method adopted involved spreading and moisture conditioning the fill in two layers of equal thickness to a total layer thickness of 1 m, followed by a minimum of 8 passes with a vibrating smooth drum roller (static mass not less than 8.5 tonnes per metre width).

To reduce the risk of void propagation, especially in fill areas placed below the long-term groundwater level, Types A, C and the matrix (-19 mm) component of Type B fill were required to have moisture contents within 1.0 % Wet and 3.0 % Dry of the optimum moisture content for Standard compaction.

To establish the settlement characteristics of the constructed fill material and allow the timely release of land for development, settlement monitoring plates (SMPs) were installed in Type B and Type A fill. Where the depth of Type B fill placed over the quarry floor exceeded ten metres, SMPs were installed in pairs. One plate was installed approximately ten metres above the quarry floor (within the Type B Fill) and the other plate at the interface between Type B and Type A fill. Further SMPs were installed at the completed fill surface. SMPs were not installed within the Type C fill zones as it was anticipated that piled foundations would be adopted in those areas.

## 6. METHOD OF PLACEMENT AND ACCEPTANCE OF ENGINEERED FILL

The typical construction sequence for placement of structural fill on areas of the former quarry floor was as follows:

1. The lake was dewatered with dewatering continued as required.
2. Stockpiled fill was excavated to the level of the existing lake water for re-use as the appropriate fill type.
3. Stockpiled fill was excavated below the existing lake water level for re-use or disposal as Type D fill.
4. Loose or otherwise deleterious material was removed from the base of the excavation to achieve an approved subgrade. The subgrade was proof rolled if soils were encountered.
5. The exposed subgrade was surveyed.
6. If groundwater inflows were encountered, a single layer of Coarse Rock fill was placed. Type B fill was then placed with testing, survey and installation of SMPs as required.
7. If the base was dry Type B fill was placed with testing, survey and installation of SMPs as required.
8. On completion of Type B filling, SMPs were installed and Type A fill placed with testing and survey as required.

A variety of plant was used to spread and compact fill, including 825 sheepsfoot compactors, graders, dozers and vibrating sheepsfoot and smooth drum rollers. Type B fill was typically bulk excavated and transported by dump trucks with moisture conditioning occurring during loading and placement. Oversize rock was removed during loading or broken by excavators with rock breakers on the fill front. Type A fill was produced from screening of stockpiled materials together with some crushing of basalt cobbles and boulders. Type C fill was typically won from stockpiles of high clay content with oversize rocks removed during placement.

Figure 2: Placement of Type B fill won from stockpiles at right hand side of figure

Acceptance of Type A and Type C materials was based on conventional density and moisture content testing. Acceptance of Coarse Rock fill was based on an observation of the construction process, survey of the completed layer

thickness and proof rolling of the finished layer. Acceptance of the Type B fill was based primarily on conformance to the approved method for blending, conditioning, placement and compaction described in Section 5. During construction the requirement to use a roller at the completion of each 1 m layer was changed to a minimum of 4 passes of an 825 compactor per 0.5 m layer. Type B fill was placed under Level 1 supervision to assess conformance to the approved method. Test slots were excavated through completed layers to allow a visual assessment of grading and compaction.

All fill areas were surveyed at the frequencies given in Table 2 to confirm the placed layer thicknesses and to ensure that no uncontrolled fill was present in the completed fill area.

Figure 3: Placement of Type A fill (note figure 3 is taken from roughly the same location as figure 2)

## 7. SETTLEMENT MONITORING

Figure 4 shows the results of settlement monitoring from SMPs installed in fill placed over the south eastern part of the former quarry pit (to the right of Figures 2 and 3) in one of the deepest areas of fill. The depths of fill above and below each SMP are given in Table 3.

When reviewing the data presented in Figure 4 it is important to assess general trends rather than isolated weekly or monthly movements as the accuracy of the survey is such that a difference between consecutive readings could lead to a variation of between 10 mm and 15 mm in the accuracy of the survey alone.

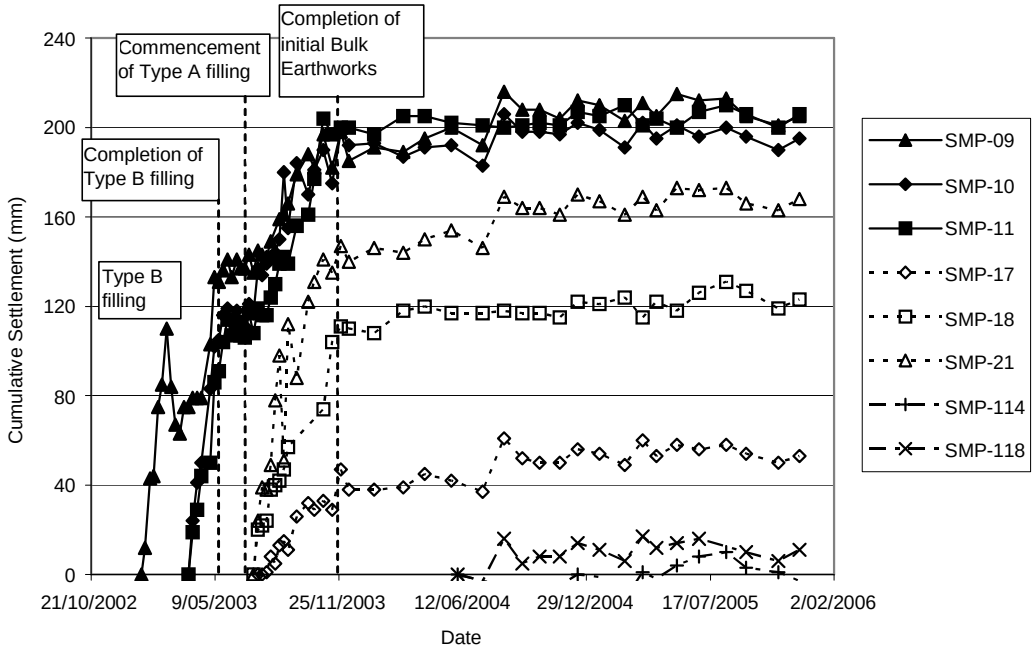


**Figure 4 – Selected results of settlement monitoring, Valley Lake**

Table 3: Installation details for settlement monitoring points shown in Figure 4

| Settlement Plate | Quarry Floor RL (m AHD) | Depth of Fill Beneath Plate (m) | Total Fill Surcharge Over Plate (m)* | Comments |
|---|---|---|---|---|
| SMP-09 | 15.52 | 10.14 | 17.43 | Paired with SMP-21 |
| SMP-10 | 19.11 | 9.74 | 18.50 | Paired with SMP-17 |
| SMP-11 | 19.15 | 10.12 | 21.86 | Paired with SMP-18 |
| SMP-17 | 19.08 | 25.63 | 2.55 | Paired with SMP-10 |
| SMP-18 | 19.27 | 27.63 | 4.37 | Paired with SMP-11 |
| SMP-21 | 15.56 | 24.94 | 3.12 | Paired with SMP-09 |
| SMP-114 | 16.10 | 21.44 | 0 | Surface monitoring |
| SMP-118 | 16.28 | 28.65 | 0 | Surface monitoring |

*To 8 December 2005

## 8. CONCLUSIONS

The results of the settlement monitoring indicated that total settlements in the order of 1 – 1.5 % of fill depth had been achieved. Minimal ongoing settlements were observed after about September 2004, nine months after the completion of the bulk earthworks. SMPs installed at the surface of completed fill areas indicated that surface movements of less than 20 mm had been observed for a period of more than a year. It should be noted that groundwater levels as measured in the feature lake reached a level within 0.5 m of the predicted maximum level (approximately RL 27 m AHD) over the time period shown in Figure 4. As the groundwater level has risen there has been no noticeable increase in the amount of settlement observed in the SMPs.

The settlement monitoring indicates that settlement of the fill generally follows the rate of placement, with the greatest rate of settlement generally observed during intensive fill placement. The rate of settlement then decreases and flattens off as the rate of placement decreases.

The Valley Lake redevelopment has shown that, given engineering control during construction, it is possible to rehabilitate land for residential development on shallow footings, despite numerous geotechnical challenges and the presence of deep fill zones.

## 9. ACKNOWLEDGEMENTS

The author extends his appreciation to VicUrban for permission to use technical data obtained during the development.

## 10. REFERENCES

Standards Australia, AS2870 (1996) “Residential Slabs and Footings – Construction”

Standards Australia, AS3798 (1996) “Guidelines on earthworks for commercial and residential developments”

Golder Associates Internal Reports, 1994 to 2005

# THE DESIGN AND PERFORMANCE OF A TEMPORARY SHEET PILE WALL

**Christopher J. Coulson**
*Maunsell Australia Pty Ltd*
*Melbourne, Australia*

## ABSTRACT

A property in inner Melbourne has recently been developed following the realignment of an existing subsurface stormwater drain that crossed the site. The realigned drain followed the site boundary, and required the design and installation of a soil retaining system along the perimeter of the project site.

A site investigation allowed a geotechnical model of the project site to be prepared. The stratigraphy and material properties were estimated from fieldwork and material testing. The geotechnical model provided the input for the design of a retaining structure.

A temporary propped sheet pile wall was designed to allow construction of the new drain. A number of underground services, as well as a masonry structure on shallow strip footings, exist adjacent to the northern and eastern site boundaries. These services and the footing are located within the zone of influence of the drain excavation, and would be affected by any adverse lateral deflection of the temporary wall.

The monitoring of the underground services, the adjacent structure, ground surface settlement, and lateral soil deflection was undertaken during the installation of the temporary works, excavation and construction of the drain. The results of the monitoring indicated the magnituce and deflection profile of horizontal soil movements behind the retaining wall were different than predicted during design. Back calculation of the elastic modulus of the soil and loads induced in the sheet pile wall suggest the soil mass did not transfer full earth pressures onto the wall during construction. Further analysis also indicated some soil properties adopted in the geotechnical model may have been more conservative than the actual soil mass behaviour would suggest.

Notwithstanding these points, the monitoring indicated the magnitude of soil movement behind the retaining wall was within the maximum deflection predicted during design, and the realigned drain was successfully constructed. The successful relocation of the stormwater drain has allowed the project site to subsequently be developed.

## 1. INTRODUCTION

This paper discusses the geotechnical aspects of the design and performance of a temporary retaining wall, used to support an excavation for the construction of a new stormwater drain around the perimeter of a property. The property is located approximately 5 kilometres east of the Melbourne CBD.

A brick invert / concrete slab roof stormwater drain was constructed along a natural drainage course in Hawthorn during the late 19th century, to direct storm water runoff to the nearby the Yarra River. This subsurface drain was located diagonally across the middle of the project site. The owner of the drain, Melbourne Water, would not allow the owner of the site to proceed with the construction of a multistorey commercial development with the drain located directly beneath the building. To develop the site, the site owner was required to relocate the drain along a new drainage easement along the eastern and northern site boundaries. The new easement would not be developed, but the remainder of the site would be available for use by the site owner. Figure 1 presents the project site with the existing and proposed drain alignments.

The project site had previously been used as an automobile service and petrol station. Underground fuel storage tanks had been removed prior to this investigation and the tank excavations backfilled.

Maunsell Australia Pty Ltd (Maunsell) was engaged by Roland Black and Associates (the Client) to undertake site investigations necessary to prepare design documents for the construction of the relocated stormwater drain. The new drain would tie into the existing drain on the southern and northern boundary of the site. With the new drain commissioned, the old drain could be removed, allowing the site to be developed.

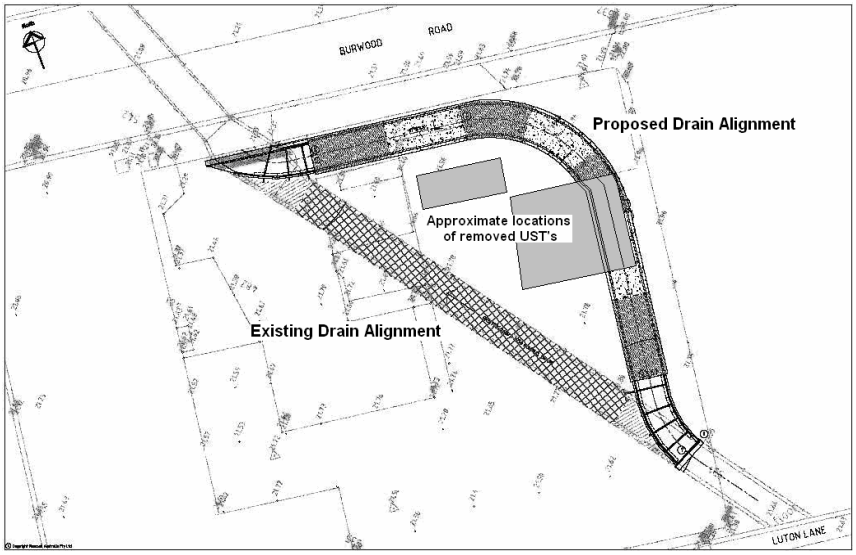


Figure 1: Project Site with existing and proposed drain alignments.

The project site is bounded by Burwood Road to the north, and a single storey masonry building to the east. Underground services, including water mains and optical fibre, exist below the southern footpath of Burwood Road, and are susceptible to damage when subjected to differential ground movement. The masonry building along the eastern boundary is founded on a shallow strip footing, and is also susceptible to differential ground movements. The underground fuel storage tanks used by the petrol station had been removed and backfilled prior to Maunsell becoming involved in the project, and the proposed drainage alignment passed through a section of the tank backfill.

A number of retaining systems were considered during the concept design stage, including permanent contiguous pile walls and sheet pile walls with props, anchors or rakers. Temporary propped sheet pile walls were selected as they require no temporary anchorages (which would extend beneath the adjacent road and building); and width of easement was minimised (the diameter of the bored pile would encroach on the available easement width).

The adopted design for the drain was an inverted precast U culvert section on an insitu concrete base. The maximum depth to the invert of the drain from existing ground surface level was approximately 4.0m. A cross section of the proposed temporary retaining system is presented in Figure 2, below. The struts would be located at 4m centres, allowing the drain units to be placed into the excavation between the struts.

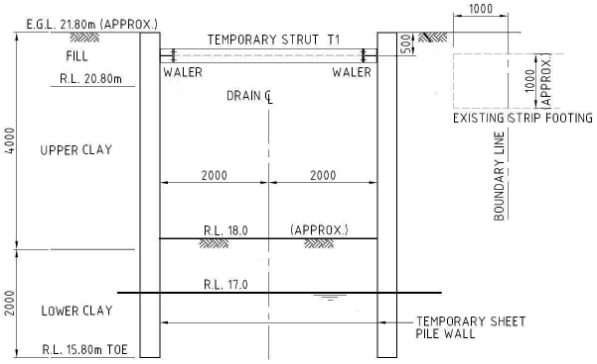


Figure 2: Cross section of temporary works along the eastern boundary with existing strip footing located adjacent to excavation

## 2. SITE INVESTIGATION

Site investigation fieldwork consisted of four test pits and six boreholes located along the proposed drain alignment. Insitu hand vane testing, Standard Penetration Testing (SPT) and hand penetrometer testing were conducted on materials encountered during the fieldwork. Laboratory classification testing (Atterberg Limit, Particle Size Distribution) of samples collected during the fieldwork was performed to confirm field logging on site, and to correct hand shear vane results for plasticity. The groundwater regime across the site was monitored using a number of standpipes already present on the site.

Published Geological Survey of Victoria literature indicates the geology of the site comprises Quaternary aged alluvial deposits overlying the Silurian aged interbedded siltstone and sandstone of the Dargile (Melbourne) Formation. The results of the site investigation confirmed the presence of Quaternary alluvium, and residual soils of the Dargile Formation, but the investigations did not extend deep enough to encounter the weathered rock. A layer of surface fill of varying composition was encountered consistently across the site. Locally, fill to depths of greater than a metre were encountered where underground storage tanks had been removed.

Figure 3 presents the geological long section along the eastern boundary of the site with the new drain invert and obvert superimposed on the section. The stratigraphy presented in Figure 3 has been inferred from the two boreholes shown on the long section, along with three test pits and two boreholes offset from the section.

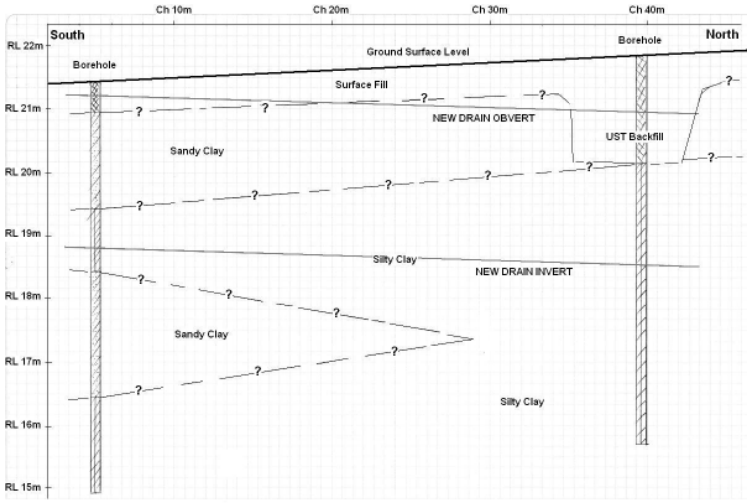


Figure 3: Geological long section, eastern property boundary (north to right)

The findings of the site investigation were used to construct a geotechnical model, which was used to design of the retaining walls. The stratigraphy of the model consisted of a surface layer of fill overlying two horizontal clay units.

The surface fill across the site varied between cohesionless sandy gravel, silty clayey sands, and stiff silty to gravelly clay, and was considered to typically be unengineered. SPT N values in the fill material ranged between 3 and 15, and hand penetrometer tests within cohesive fill material indicated undrained strengths of between 10 and 150kPa. To account for the presence of cohesionless material, which would be expected to be the critical load case in an undrained analysis, the surface fill in the model was assigned a cohesion of 0kPa and internal friction angle of 25 degrees.

The drained elastic deformation modulus of the surface fill in the model was estimated from the results of two standard penetration tests performed in clayey sand fill (N=4; N=6). Bowles (1996, p316) proposes a relationship between N and the elastic deformation modulus, E = 320 (N + 15), which yields a value of about 6MPa/m depth. The retaining wall computer program WALLAP recommend a similar value of 5MPa/m depth. A value of 5MPa/m depth was adopted for the model.

The areas of deep fill (in the location where underground storage tanks had been removed) were not directly considered in the model, as none of the deep fill occurred between the retaining wall and the critical services or strip footing. Figure 1 shows the location of the deep fill in relation to the inner and outer propped retaining walls. The critical external retaining wall was designed to restrict the movement of the soil adjacent to the underground services and strip footing using the insitu materials and surface fill only. No additional analysis involving the deep fill was undertaken.

The residual soil of the Dargile Formation was encountered in one borehole only, at a depth of 6 metres. The remaining material encountered was logged as alluvial (or similarly deposited) material.

The undrained cohesion of the alluvial material was estimated from field testing results. The results of the hand shear vane (corrected for plasticity) and hand penetrometer testing, plotted against depth, are presented on Figure 4. The hand shear vane test results show an increase in undrained cohesion with depth. Superimposed on the plot is a linear regression 'line of best fit' for the vane results. The line of best fit calculation includes two vane results that exceeded the capacity of the apparatus. These points are plotted at the maximum recorded value on the hand vane, although it is likely that the actual value of undrained cohesion for these points is greater than presented. The undrained cohesion estimated from hand penetrometer test results plot consistently higher than the hand shear vane tests, but also suggests the undrained cohesion increases with depth. The hand vane tests were considered more representative of the undrained cohesion, and formed the basis for the undrained cohesion for the geotechnical model.

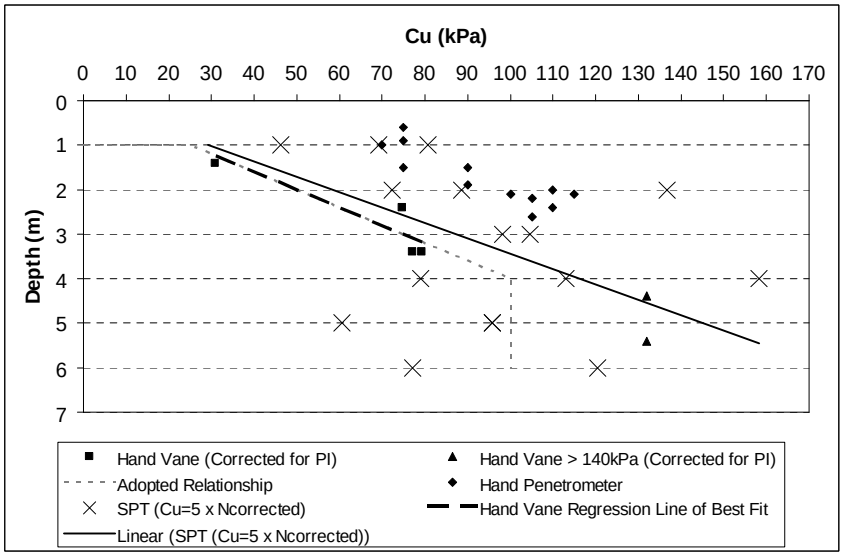


Figure 4: Undrained Cohesion of natural material with depth, estimated from field testing

The line of best fit for hand vane data in natural materials to a depth of 4 metres indicates cohesion increasing with depth at a rate of approximately 25kPa/m, a rate of increase also observed in the hand penetrometer testing.

Only two hand vane tests were performed on material below 4 metres depth, and as mentioned above, both of these tests exceeded the capacity of the hand vane. SPT testing was performed in boreholes across the site, including a number of tests below 4 metres depth. Bowles (1996, p165) cautiously proposes a linear correlation for SPT N value and the undrained compressive strength, $q_u$ (where $q_u = k \times N$; $s_u = q_u/2$, and k is a linear factor based on site correlations). For results collected from site, k = 10 provides a similar fit between the hand vane $s_u$ results and SPT data, as shown on Figure 4. Below 4 metres, the SPT data plots as a scatter, ranging between 60 and 120 kPa. Although scattered, the SPT data does not support the continued 25kPa/m increase of undrained strength below 4 metres, as indicated by the two hand vane test results. The undrained strength of the material below 4 metres depth was limited to 100kPa in the geotechnical model, based on the variability of the various test results.

Fell et al (2005, p277) state SPT results can be used to estimate the undrained elastic modulus of a soil, but caution the use as there is considerable uncertainty in the use of such relationships. The relationships proposed by Fell are presented graphically, and consist of a large scatter of test results displayed on log-log plots. The undrained elastic modulus for the cohesive material in the model was instead estimated using empirical relationships based on the undrained cohesion of the material.

Bowles (1996, p317) suggests a factor of between 100 and 500 be multiplied to the undrained shear strength of clays and silts to estimate the undrained elastic modulus, while WALLAP recommends a factor of between 270 and 500 for stiff to very stiff cohesive soil (Su between 75 and 150kPa). A factor of 300 was adopted for design, resulting in an undrained elastic modulus of 7.5MPa + 7.5MPa/m for depths between 1 and 4 metres, and 30MPa below 4 metres.

The stratigraphy and adopted properties of the geotechnical model is presented in Table 1 below.

| **Unit** | **Depth to top of unit** | **Unit thickness** | **$\phi$' (Fill) $\phi_u$ (Clays)** | **c' (Fill) $c_u$ (Clays)** | **E' (Fill) $E_u$ (Clays)** |
|---|---|---|---|---|---|
| | m | m | degrees | kN/m$^2$ | kN/m$^2$ |
| Fill | 0 | 1 | 25 | 0 | 5000 |
| Upper Clay | 1 | 3 | 0 | 25 + 25/m | 300 x $c_u$ |
| Lower Clay | 4 | Not defined | 0 | 100 | 30000 |

Table 1: Geotechnical Model - Stratigraphy and Material Properties

Groundwater was observed in all standpipes across the project site. The water level was approximately 5 metres below the ground surface level, although some perched water was encountered. The model considers the groundwater level to be at five metres below the ground surface, which is below the invert of the drain.

## 3. TEMPORARY WORKS DESIGN

The stratigraphy and material parameters adopted in the Geotechnical Model were used as inputs into the finite element analysis program WALLAP, to design size of the pile sections, the founding depth of the piles and the strut details.

Various grades of sheet pile section were considered during the wall analysis for serviceability and ultimate load conditions. The serviceability condition addressed limiting lateral wall and soil deflection to within 15mm for the adjacent building and underground services. The ultimate condition verified the global stability of the wall. The ultimate condition was also checked using the slope stability program SLIDE.

The above analysis with PU8 sheet pile section resulted in a maximum elastic deflection of the wall of 13mm, and with a pile toe depth of 6 metres from ground surface level, the wall satisfied the ultimate limit state stability case. The design analysis predicted the maximum wall deflection to occur towards the base of the excavation, maximum bending moments in the wall would be 35kNm/m run of wall, and that axial compression loads in the strut members would reach 50kN/m run of wall.

Following the preparation of design drawings, one of the preferred piling contractors proposed the use of smaller PU6 sheet pile sections. This contractor was awarded the contract with a non-conforming tender, and accepted the risk of damage to the adjacent services or building as a result of any excessive soil movement caused by deflection of the wall.

## 4. CONSTRUCTION

Ground surface settlement points were located along the length of the temporary excavation immediately adjacent to the sheet pile wall. An inclinometer was located behind the sheet pile wall along the northern boundary. The location of ground surface settlement points and the inclinometer are presented on Figure 5. The settlement points and inclinometer were monitored for the duration of construction and backfill of the new drain.

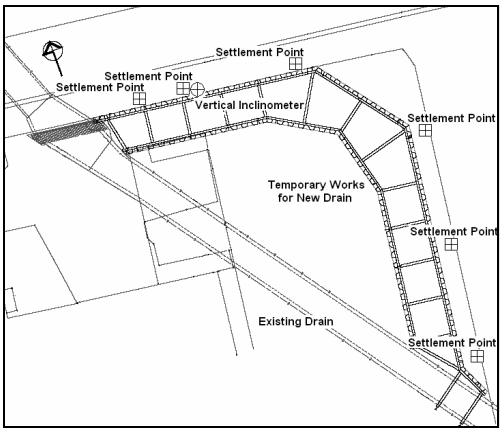


Figure 5: Location of monitoring locations relative to the temporary works layout

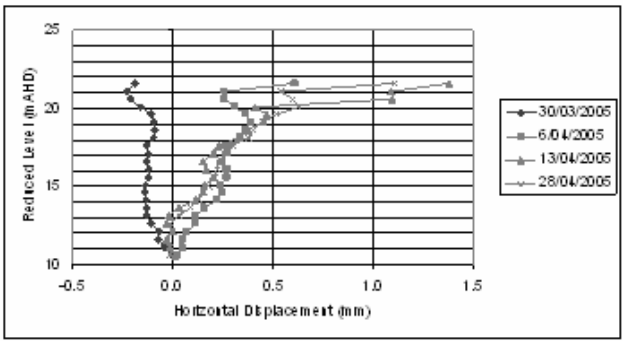


Figure 6: Results of inclinometer monitoring

The results of settlement point survey data indicate settlement along the top of the temporary wall was less than 2mm during construction. The inclinometer recorded a maximum deflection at the surface of less than 1.5mm. Results of the inclinometer monitoring are presented in Figure 6.

## 5. DISCUSSION

The results of the surface monitoring and the inclinometer indicate significantly less soil deflection than expected. Only one inclinometer was installed, and with no other direct measurement of lateral soil or wall deflection, the accuracy of measurements and application of results across the site can be questioned. The small recorded surface settlements recorded along the wall support the possibility of the inclinometer data being representative of actual wall deflections. There was no monitoring of axial loading in the struts, and it is unknown if the struts were preloaded when installed.

If the inclinometer results are representative of the actual wall deflection profile, a numerical analysis using structural theory predicts the corresponding bending moments generated in the sheet piles probably ranged between 2 and 10kNm/m run of wall. The design bending moment for the sheet piles was 35kNm/m run of wall. The active earth pressures acting on the wall to generate the recorded deflections are less than 5 kPa. This limited transfer of load from the soil mass to the back of the wall may indicate there was enough cohesion in the soil for it to remain predominantly self supporting during the construction of the drain.

If the soil did behave as an undrained cohesive self supporting soil mass during construction, elasticity theory can be used to predict the bulk modulus required to generate the lateral deflections recorded. For the case where the soil deflects 1.5mm and does not apply an active earth pressure to the wall, the constant bulk modulus of a homogeneous material lies between 40MPa and 50MPa. It is not unreasonable to consider the possibility of a near surface gap of 1.5mm forming between the soil and sheet pile caused by soil disturbance during driving of the piles. As active earth pressure is transferred to the wall, the back calculated elastic modulus of the soil mass reduces, but wall deflection increases, and the wall deflection profile changes, displaying the influence of the propped strut as it attracts load.

An elastic modulus of 40 to 50 MPa is representative of an overconsolidated to highly overconsolidated clay. The undrained modulus adopted in the geotechnical model was estimated from the undrained cohesion, and increased linearly from 7.5MPa at the base of the fill, to 30MPa at a depth of 4 metres. The derivation of the undrained cohesion in the upper clay unit relied on a linear regression through the lowest shear vane result ($c_u$=31kPa at 1.4 metres depth). If this result is ignored as an outlier, then the remaining vane, hand penetrometer and SPT results presented on Figure 4 suggest a less conservative relationship between $c_u$ and depth may exist (for example, constant $c_u$ of 75kPa between 1 and 4 metres may be more representative of the field testing results). If 75 kPa is adopted, the relationships proposed for undrained modulus by Bowles (1996, p317) and WALLAP (Geosolve, 1995) estimate $E_u$ somewhere between 20MPa and 38MPa. Maintaining the adopted value of $c_u$=100kPa below 4 metres, Bowles (1996, p317) and WALLAP estimate $E_u$ to range between 27MPa and 50MPa. The higher values for $E_u$ predicted by these methods are similar in magnitude to range of undrained modulus estimated in the elastic back analysis described above.

If the inclinometer results are reliable, the back analysis suggests the fill material in the upper metre displayed cohesive properties, unlike the purely frictional material assigned in the geotechnical model. This may not unexpected, as the site investigation encountered a range of materials in the surface fill, and the geotechnical model adopted the critical undrained load case. The inclinometer was also located beneath a concrete pavement, suggesting the near surface material may have experienced a greater level of compaction than much of the remaining fill on the site. Any increase in the density would increase the undrained strength and modulus of the material.

## 6. CONCLUSION

The inclinometer results recorded during construction suggest the actual wall deflections varied significantly to those predicted during the design. Monitored results indicated maximum soil deflection occurring near the top of the excavation, and a maximum recorded deflection only about 10% of the predicted deflection. The design predicted maximum wall deflection towards the base of the excavation. A back analysis of the recorded soil deflections indicates little, if any, lateral earth pressure was transferred to the sheet pile wall. Assuming the monitoring results are accurate, the deflection profile of the soil indicates the soil had sufficient undrained cohesion during the period of construction to remain at least partially self supporting, resulting in minimal if any active load being transferred to the wall.

Based on a fully self supporting soil mass, the elastic modulus of the soil was back calculated to range between 40 and 50MPa. The geotechnical model adopted values ranging from 7.5MPa to a maximum of 30MPa. The test data used to estimate both the undrained cohesion and elastic modulus of the cohesive soils for the geotechnical model showed a scatter, and with hindsight the adopted values maybe conservative. A reasonable re-assessment of the data results in a range of undrained elastic moduli for the soil that are not unlike the values predicted in the elastic back analysis.

## 7. ACKNOWLEDGEMENTS

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# GEOMORPHOLOGY AND FAULT ZONE KINEMATICS OF THE MT FYFFE SEGMENT OF THE HOPE FAULT, SOUTH ISLAND, NEW ZEALAND

**Roseanne Coulter**
*New Zealand Geotechnical Society*
*URS NZ, Christchurch New Zealand*

## ABSTRACT

The northeast-trending transpressive Hope Fault is the most active structure of the Marlborough Fault System, transferring stresses from the Hikurangi Subduction zone to the transcurrent Alpine Fault.

The Mt Fyffe segment is expressed as a zone of faulting and deformation up to 1 km wide. Fault scarps can be divided into three spatially distinct styles; strike-slip, reverse, and uphill facing. In the southern part of the segment deformation is expressed as a series of en-echelon pop-outs linked by oblique shears. The pop-outs comprise semi-parallel zones of the three types of fault scarp. The northern part of the segment is dominated by reverse faulting. Generally the fault traces show an overall pattern of deformation similar to the 'collapsing fault wedge model', with the more active features at the foot of the range indicating an eastward propagating wedge. The Mt Fyffe segment is further complicated by changes in slip rate and the ratio of lateral to vertical movement. This change in deformation style is related to changes in fault zone orientation and stress transfer to the adjacent Jordan thrust through obliquely oriented reverse splays.

Near surface fault zone kinematics recorded by tectonic geomorphic landforms are interpreted to reflect the role of topographic loading and the related fault zone break-out along the range front.

## 1. INTRODUCTION

This study is based on geomorphic mapping of the Mt Fyffe segment of the Hope Fault and presents a tectonic geomorphic model and analysis of spatial variation along the segment. The purpose of analysing this segment is to develop a better understanding of fault activity and kinematics that can be applied to paleoseismology and hazard analysis.

New Zealand lies on the boundary between the Pacific and Australian plates, to the north the Pacific plate is subducted at the Hikurangi Margin, and to the south the Australian plate is subducted at the Puysegur Trench (Figure 1). The two subduction zones are separated by the 1000 km long right-lateral Alpine fault. Stress is transferred from the Alpine fault to the Hikurangi margin through an approximately 150 km wide zone of transpressive deformation, the Marlborough Fault Zone (MFZ).

The MFZ comprises an assemblage of discontinuous faults with variable slip vectors and four larger, segmented right-lateral strike-slip faults, from north to south the Wairau, Awatere, Clarence, and Hope. Near their southwest and northeast terminations the faults generally splay and form thrust systems (Pettinga et at 1998). The Hope fault forms the southern boundary of the zone, and currently has the highest horizontal slip rate.

As well as acting as a transition zone between a subduction and a transcurrent plate boundary, the MFZ also accommodates a change in slip partitioning from the non partitioned Alpine fault (Norris and Cooper, 1995) to the spatially distinct reverse and strike-slip faulting seen to the north (Little and Jones, 1998). The MFZ displays increasing partitioned faulting to the north east (Little and Jones 1998). Transition in kinematic behaviour corresponds in a broad sense to southward locking of the Pacific and Australian plates beneath the MFZ.

The Hope fault stretches 220 km from the Alpine fault to north of Kaikoura, and is the second fastest slipping onland fault in New Zealand (Berryman et al, 1992; Pettinga et al, 1998). It has a mean slip rate of 23 mm/yr, with a $M_{max}$ of 7.5 at a 200 year recurrence interval. This magnitude of earthquake would produce displacements of greater than 4 m, and pga's of 0.5 to 0.6 g at Kaikoura township (Stirling et al 2002). State highway One also passes within a few km of the fault trace (Figure 1). The largest historical earthquake on the Hope fault was a $M_{max}$ 7.3 in 1888. The Hope fault has been the subject of many studies including structural deformation, paleoseismology, and basins (see Eusden et al, 2000). Variations in fault behaviour and structural geometry define four segments along the Hope fault; the Taramakau-Hope River segment; the Conway segment; the Mt Fyffe segment; and the Seaward segment. The segments are linked through fault splays which are reflected in changes in topography.

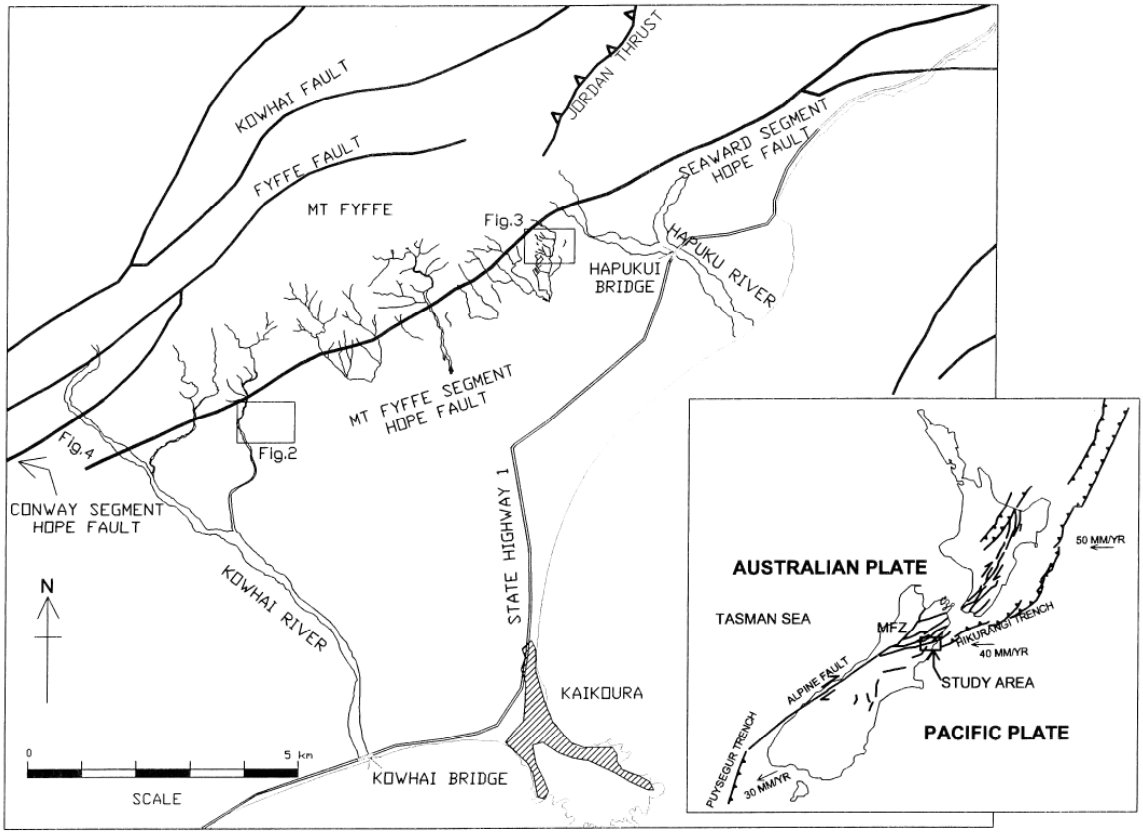


Figure 1: Regional setting and location map of study area.

## 2. MT FYFFE SEGMENT

The transpressive Mt Fyffe segment is a range bounding fault which extends about 12 km from the Hapuku River in the northeast to the Kowhai River in the southwest, where the fault steps about 150 m to the east, and the Fyffe fault splays inland (Figure 1). To the north topography becomes more subdued as the Mt Fyffe segment links into the less active Seaward segment.

The Mt Fyffe segment forms a slight arc, changing in strike from 275° in the southwest, to 235° in the northeast, with a dip of about 70°. The hanging wall of the fault zone comprises deformed Torlesse greywacke with a Tertiary limestone and sandstone anticline at the north-eastern end. Tertiary deposits (mainly sandstone) are exposed in remnant or possibly uplifted hills on the flood plain, which has a thickness of gravel of at least 60 m.

The surface expression of the fault is represented by discontinuous fault traces and folding forming a zone up to 1 km wide, obscured by the active erosional boundary between range and fan.

The Holocene slip rate and ratio of lateral to vertical displacement changes along the segment from 16 ± 5 mm/yr (min) and 150:1 at Goldmine Creek just north of the Kowhai River (Van Dissen, 1989), to 4.8 ± 2.7 mm/yr and 2:1 at the Hapuku Terraces in the north (Figure 3) (Knuepfer, 1984; Van Dissen, 1989). Fault plane curves and irregularities, equivalent to restraining and releasing bends and step-overs, add considerable complexity to the structural deformation observed.

## 3. GEOMORPHOLOGY

The structural geology and geomorphology of the study area was mapped using air-photo interpretation and field mapping. The photos were enlarged to 1:5000 to 1:7000 and used as base maps. Bedrock geology was taken from Lensen (1975) and field mapping.

Sixteen locations were mapped in detail along the segment. Over 200 active structures were mapped, and 600 more interpreted from aerial photographs. The tectonic geomorphology of two of the sites, Floodgate Creek and Hapuku River terraces is discussed below.

### 3.1 FLOODGATE CREEK

A series of alluvial fan terraces formed by Floodgate creek straddle the propagating Mt Fyffe Segment (Figure 2). They record extensional, strike slip and compressional active tectonic features. The creek catchment area has been shuttered to the northeast relative to its fan several times.

The extension features comprise two up hill facing fault scarps and one up hill facing monocline. The biggest scarp is 10 ± 2 m high with at least 140 m lateral offset, the other scarp is about 50 m north and is displaced 1 m vertically and 10 m dextrally (about 10:1 lateral to vertical ratio on both faults), and develops into a monocline to the east. The monocline is another approximately 50 m north of the smaller scarp, and is on the metre scale.

The middle section of the deformed fan has a series of short reverse faults, about 20 m long, and up to 14 m high. The appearance of fault segmentation is enhanced by stream incision. These faults are inferred to be the result of propagating back thrusts.

Down slope of the reverse faults is a zone of monoclines or fold propagating faults, indicating the back thrusts are actively propagating to the east. The monoclines form more rounded 'scarps' compared to the sharper breaks in slope that result from reverse faults. Below the monoclines the streams are noticeably less incised.

The uphill facing scarps taper into an obliquely oriented wedge shaped zone of small (1-2 m high, and 10 – 15 m long) fault scarps, which are interpreted as shears.

Together the tectonic geomorphic features form a lens shape, or pop-out feature, tapering towards Floodgate Creek in the west and Luke Creek in the east. The structures with the most offset occur to the north, and the youngest features to the south. The shears link Floodgate Creek pop-out to the adjacent Luke Creek pop-out, which has structures oriented almost parallel, but stepped (called en-echelon) about 300 m north of Floodgate Creek. The formation of these en-echelon pop-outs is likely related to the relative orientation of the fault zone to plate motion.

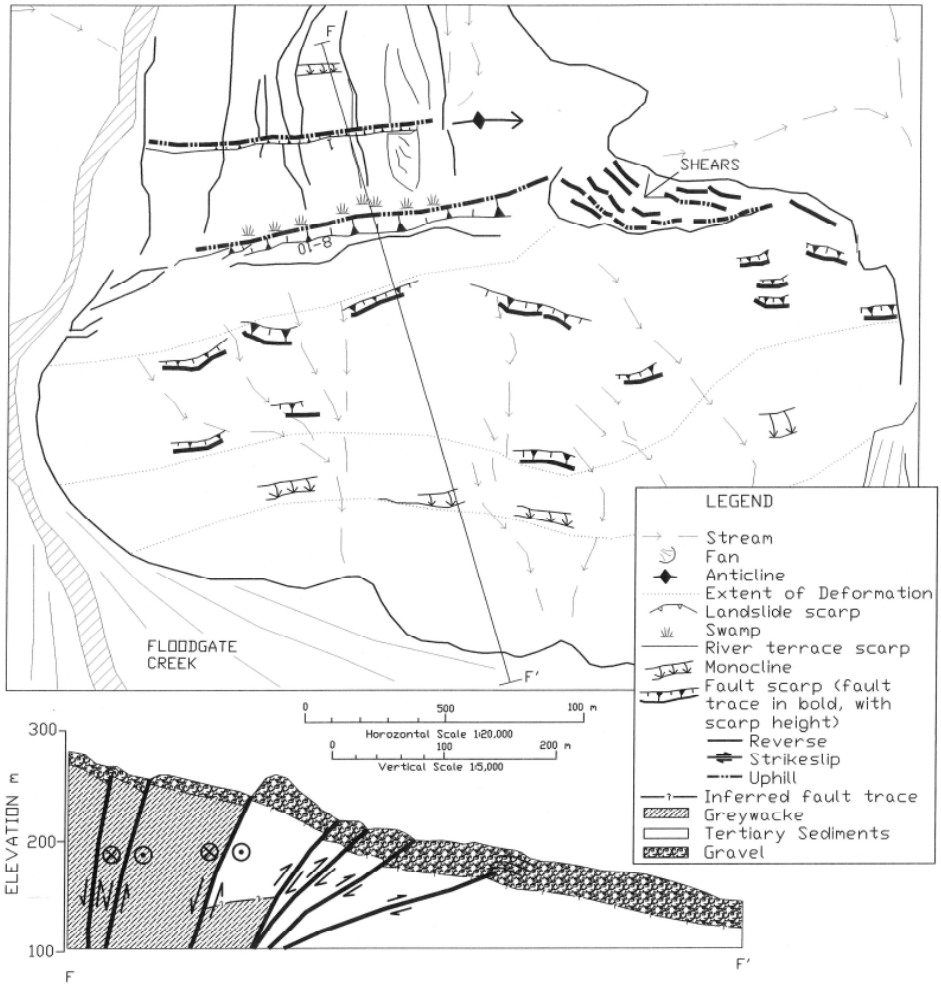


Figure 2: Geomorphic map of Floodgate Creek Terraces and schematic geologic section

### 3.2 HAPUKU RIVER TERRACES

At this site, several of the Hapuku river terraces have been offset and deformed by splays of the Mt Fyffe segment (Figure 3). The terraces are displaced by up to 26 m vertically, lateral displacement is harder to determine. Towards the northeast the fault zone becomes more segmented, with multiple splays only metres apart. The terrace on the up thrown side of the fault has also been folded, and a graben has formed sub parallel to the fault trace due to subsequent hanging wall collapse.

To the southwest, the fault forms narrow duplexes resulting in a steep flower structure, shuttered ridges, swamps, deflected streams and bulging. At the very western edge of Figure 3 two fault scarps with strikes almost north south are interpreted as bending moment splays, related to stress transfer to the adjacent Jordan Thrust. Deformation is dominated by compression.

Knuepfer (1984) determined a horizontal slip rate of 1.7-2.5 mm/yr whereas Van Dissen (1989) came up with 2.8-7.5 mm/yr horizontal slip rate. The next phase of this project involves trenching this site to gain better control on slip rates.

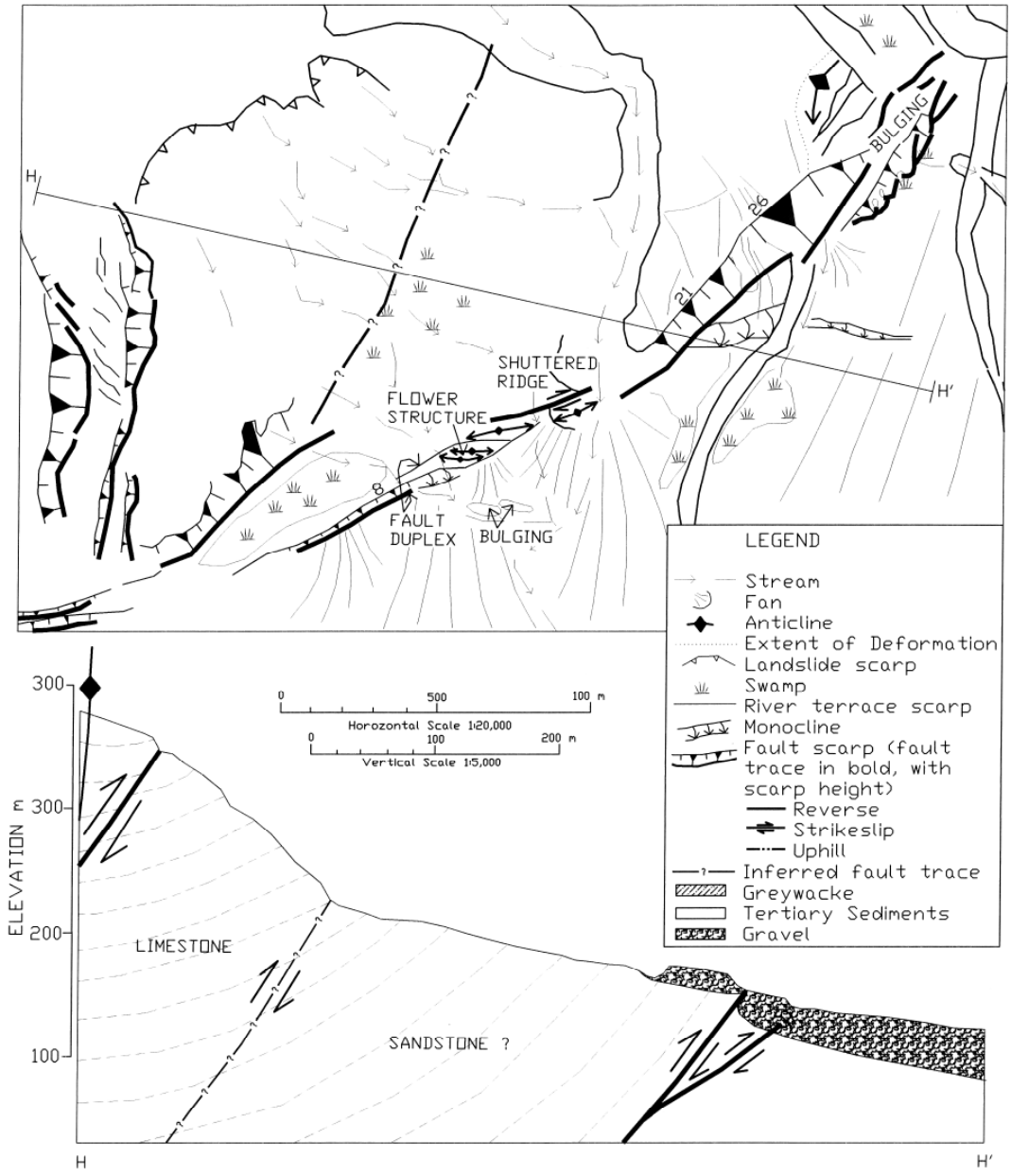


Figure 3: Geomorphic map of Hapuku River Terraces and schematic geologic section

## 4. INTERPRETATION – SPATIAL KINEMATIC MODEL

The spatial kinematic model was developed from analysis of fault type, relative location of active geomorphic features and fault strike. Both the strike-slip and up-hill facing scarps are located in the northern part of the deformation zone (Figure 4), in steeper terrain, sometimes cutting through basement at the surface. The reverse faults, monoclines and shears are all smaller; more segmented features located more at the foot of the range, and deform gravel at the surface.

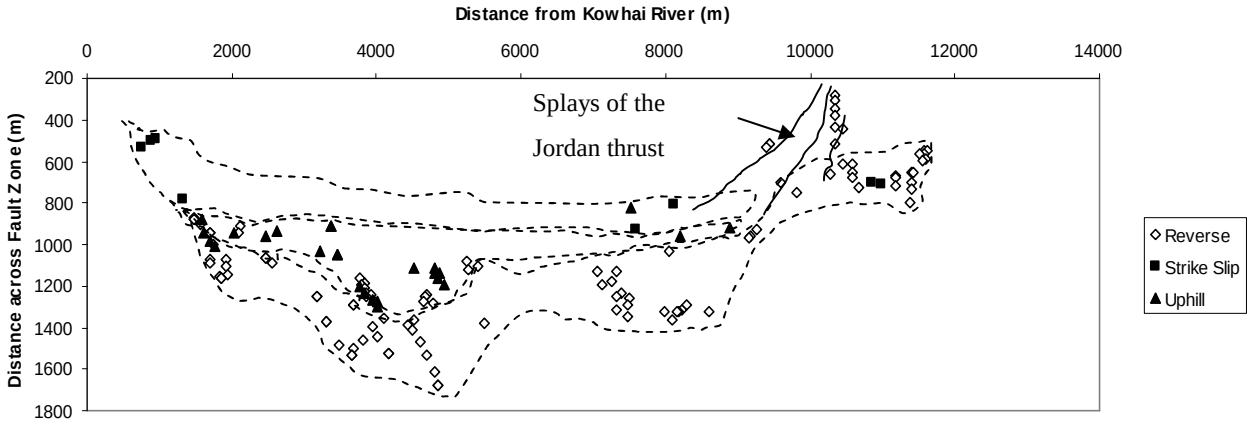


Figure 4: Graph showing spatial relationships between structural zones

The strike-slip faults form the 'main' trace of the segment. Characteristically the strike-slip faults form the longest and straightest segments and have a mean strike closest to that of the plate vector (257° DeMets, et al 1994), and a ratio of lateral to vertical off set of about 50:1.

Reverse faults define the advancing edge of the eastward propagating pop-outs (Figure 2). In the northeast the zone of reverse faults steps back several times, narrowing the width of the active Mt Fyffe segment. This is related to stress transfer to the Jordan thrust.

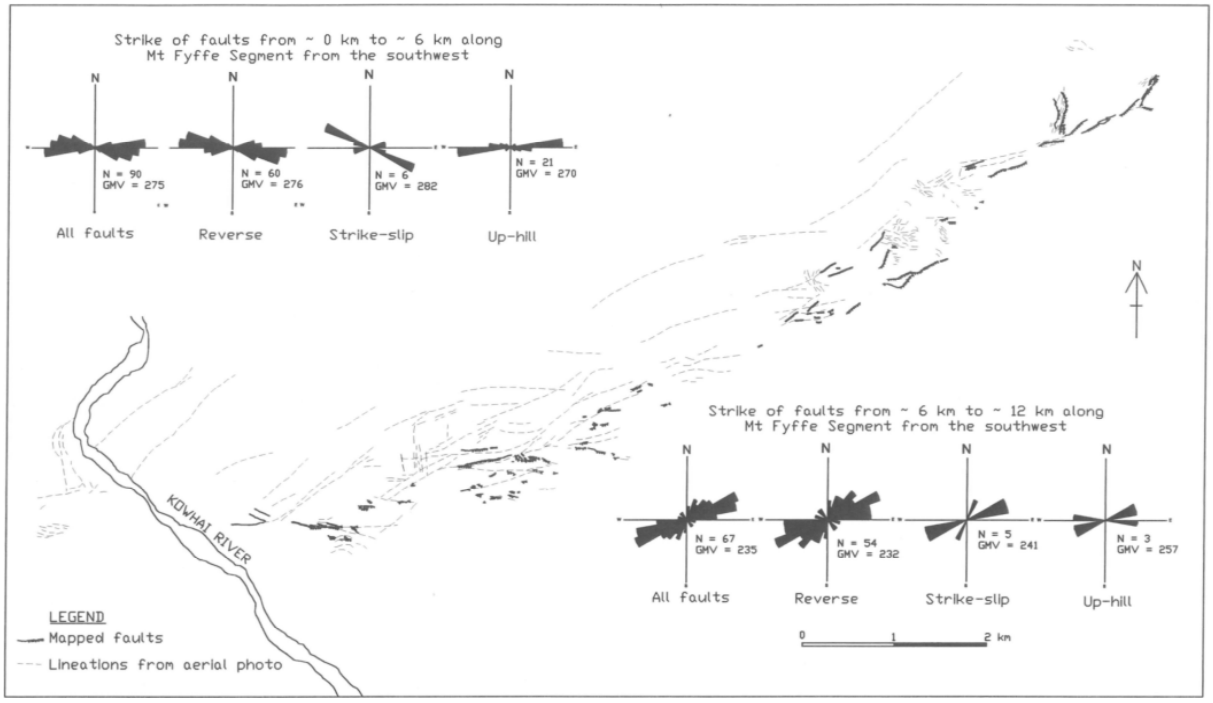


Figure 5: Structural map with rose diagrams of fault strike

Up-hill facing scarps do not fit so easily into this basic model. The up-hill facing scarps mapped in the Mt Fyffe segment are the least common fault type, forming a tapering zone between the strike-slip and reverse faults which narrows towards the Hapuku River. Two of the most likely explanations for these features are; ridge rents (mapped on other faults in the MFZ eg McCalpin, 1996) or the collapsing wedge model developed by Eusden et at, 2005. In the

wedge model strike slip faults are reactivated as normal faults after a shift in deformation into the footwall through back thrusts. This model is complicated, and would require further study to prove.

Figure 5 is a structural map of the segment, showing Rose diagrams of each fault type for the first and second 6 km of the segment. All of the fault types change in strike by about 40 degrees over this distance. This change in strike could be a rupture propagation boundary – with a collapsing wedge style of deformation to the southwest, and dominantly compression deformation to the northeast. This has important implications for segment rupture length and related hazard assessment.

## 5. SUMMARY

- The Mt Fyffe segment of the Hope fault is a 1 km wide by 12 km long zone of active deformation
- There are three fault types, strike-slip, reverse and up-hill, and they form spatially distinct zones.
- The three zones form a wedge shape, narrowing at each end of the segment and locally at smaller rivers, which define en-echelon pop-outs. The strike-slip faults occur closest to the hanging wall of the fault zone, up-hill in the middle, and actively propagating reverse faults at in the footwall.
- Faults in the south western half of the segment are spatially arranged similarly to the 'wedge model' (after Eusden et al 2005) and have a mean strike of 275°. While those in the north eastern half are dominated by reverse faults and have a mean strike of 235°. This change in strike and deformation style is related to stress transfer to the Jordan thrust, and could represent a rupture propagation boundary.
- Defining the zone of active deformation is useful for future town planning and existing lifelines. Future paleoseismology work will hopefully produce a $M_{max}$ for the Mt Fyffe segment, which can be used in hazard analysis and design criterion.

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# THE TERRACE TUNNEL APPROACH WALLS, WELLINGTON

**Beverley Curley**
*Opus International Consultants Ltd*
*Wellington, New Zealand*

## ABSTRACT

The Wellington Urban Motorway, which includes the Terrace Tunnel, is a vital lifeline for Wellington City. The tunnel approaches are supported by 16 m high retaining walls, with anchors tied back into drives in the hillside behind the walls. Transit New Zealand decided that a pro-active approach to maintenance of these walls was required.

A maintenance strategy was developed to ensure the satisfactory long-term performance of the walls, and involves a programme of inspection, testing, maintenance, and the assessment of the earthquake performance of the structures.

A total of 263 anchors support the approach walls. Methods of testing and inspecting the anchors included: endoscopic and visual inspection, checking residual pre-stress, load testing, removal of corrosion protection to check anchors and reinstatement. Inspection and testing of the Macalloy bar tie-back anchors that are protected by a yellow heat-shrink jacket system indicate that they are in good condition, with only very slight corrosion. The corrosion protection is being enhanced.

Four piezometers were also installed to monitor groundwater. Cleaning and enhancing of the existing, along with drilling of drainage holes from the anchor drives has helped lower the groundwater pressures in the hillside behind the walls, and has reduced groundwater inflows into the anchor holes.

## 1. INTRODUCTION

The Wellington Urban Motorway section of State Highway 1 (SH1) is a key road into Wellington City, and includes the Terrace Tunnel and its approach walls.

The Terrace Tunnel approach walls and anchors were designed in the early 1970s and construction was completed in the summer of 1977-1978.

Transit New Zealand (Transit NZ) commissioned Opus International Consultants Limited (Opus) to develop a pro-active maintenance strategy for the Terrace Tunnel Approach Wall Anchors. The maintenance strategy was developed to ensure that the approach walls are satisfactorily maintained during their design life and perform adequately during a seismic event. The new maintenance regime developed by Opus has been implemented.

## 2. LOCATION & SITE DESCRIPTION

The Terrace Tunnel is located on SH1 in Wellington New Zealand. The Terrace Tunnel has a north-south alignment, with the approach walls located to the western side of the road. The tunnel and its road approaches cross the Terrace Fault Zone and the in-filled valley of the Kumototo Stream. The west side of the valley originally rose 38 m, at a slope of 35°, to Kelburn Park (Riddolls & Perrin 1975).

The Terrace Tunnel has retaining walls at the southwest and northwest approaches. The northwest retaining wall is a maximum of 16 m high and 184 m long, and retains the hillslope below Kelburn Park. The southwest wall is a maximum of 12 m high and 23 m long, and retains land that contains both commercial and residential properties. Both walls have anchor drive tunnels running the entire length approximately 22 m behind the face of the retaining wall.

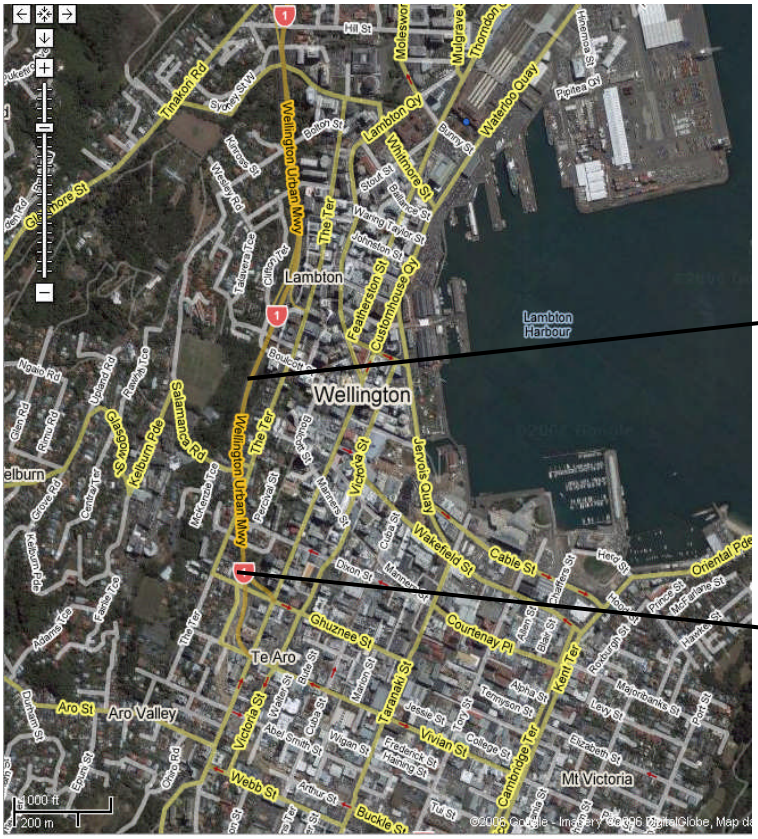


Figure 1a: Location of the Terrace Tunnel

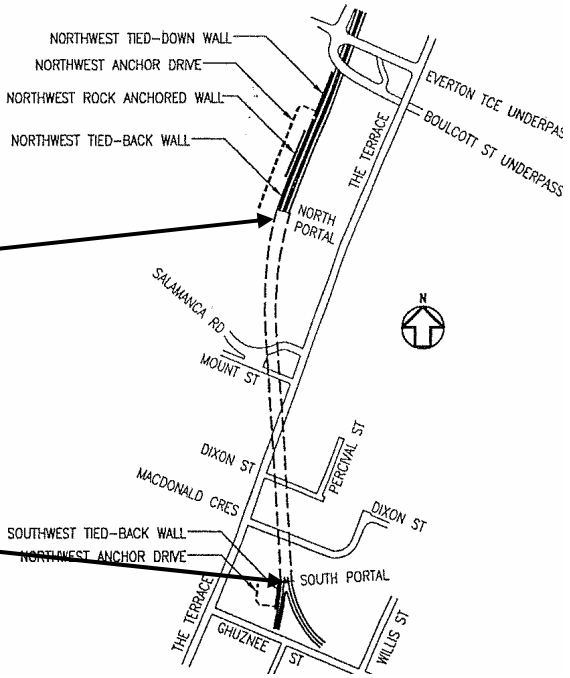


Figure 1b: Terrace Tunnel Approach Walls

## 3. GEOLOGY

The site is comprised of three main geological units. These are the Wellington Belt Greywacke comprising interbedded sandstones, siltstones and mudstones (argillite) that are well indurated and highly fractured with various degrees of weathering; alluvium, silt and loess that are associated with a erosion surface; and fill (Institute of Geological and Nuclear Sciences, 1996).

The bedrock (Greywacke) between faults in the Wellington area is generally heavily disturbed by past tectonic movements along the faults, and this is exhibited by the complex fractured nature of bedrock with closely spaced joints, shear and crush zones.

The Terrace Fault Zone consists of intensely fractured and crushed rock and fault gouge materials. These materials were encountered during the original investigations and excavation for the construction of the southwest wall. The northern motorway approach to the tunnel is constructed on fill.

## 4. RETAINING WALL ARRANGEMENT

Two designs were employed for the Terrace Tunnel retaining walls; concrete tied back, and concrete tied down walls. The southwest retaining wall is comprised entirely of a tied back wall (23 m in length). The northwest retaining wall is predominantly a tied back wall (126 m in length). There is also a lower height rock anchored tied down wall (58 m in length) in the northern section, furthest from the Terrace Tunnel portal and with a lower height. For both tied-back walls the anchors are tied-back to a 1 m thick concrete beam in the anchor drive tunnels constructed in bedrock behind the walls. The northwest tied back wall also has a short section of rock anchored wall above the highest section of the tied back wall.

Figure 2: Terrace Tunnel Northwest Approach Wall

The anchors employed in the tied-back approach walls comprise 32 mm diameter Macalloy bars anchors (rolled in the UK) with a yellow jacket protective coating (applied in Australia). The tied-down section is supported by 35 mm diameter Macalloy bar anchors, and 12.5 mm diameter strand anchors.

Both the southwest wall and northwest wall (tied back section) are comprised of three rows of anchors from the wall face to the anchor drive tunnels behind the walls. The northwest wall has two anchors at each position, totalling 230 anchors. The tied down northwest retaining wall has 16 anchors and the upper rock anchored wall has 17 anchors. The southwest retaining wall has a single twin-bar anchor at each position, totalling 19 anchors.

The anchor heads both at the wall face and in the anchor drives were covered with protective steel boxes which were filled at the time of construction with Visconorust, as special anticorrosive grease imported from the U.S.A (Gray, 1977).

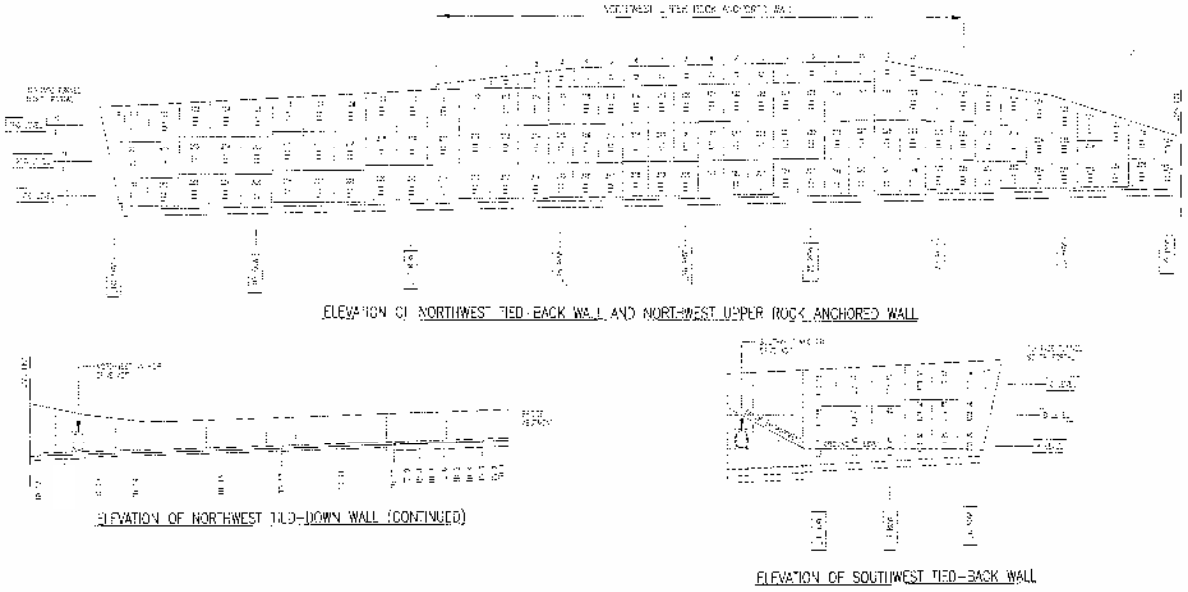


Figure 3: Layout of Approach Walls at Wall Face

## 6. MAINTENANCE OF THE TERRACE TUNNEL APPROACH WALLS

The maintenance of the Terrace Tunnel Approach Walls involves checking the pre-stress in the anchors, load testing rock anchors, cleaning anchor head drainage holes, checking for corrosion, and maintaining the drainage holes to the anchor drive.

As part of the maintenance strategy development records of previous inspections and maintenance activities were collated. A programme of inspection and maintenance was recommended following the compilation of this strategy. Based on these recommendations, Transit NZ let two maintenance contracts for the Terrace Tunnel Approach Walls. The first contract was for the installation of piezometers to monitor groundwater levels in the hillsides behind the walls, and to maintain and improve the drainage in the Terrace Tunnel Anchor Drives. The improvement in drainage was designed to reduce groundwater levels in the material behind the retaining walls to increase stability and reduce pressure on the walls. The second contract focused on the anchors supporting the retaining walls and involved inspection, testing and corrosion protection. These contracts were carried out between May 2005 and April 2006.

## 7. INSTRUMENTATION & MONITORING

As part of the 2005/2006 works, standpipe piezometers were installed to enable monitoring of the groundwater levels behind the retaining walls and to check the effectiveness of the anchor drive tunnel drainage works. The piezometers were installed by Webster Drilling & Exploration Ltd, in three locations behind the northwest wall, and one behind the southwest wall. The locations of the boreholes were chosen to monitor groundwater levels in the hillside beyond the anchor drive tunnel, between the wall face and the anchor drive tunnels, and behind the tied down wall. The drilling of the boreholes (one at the southwest and one at the northwest approach walls) between the wall face and anchor drive tunnels required a precise location between anchors and accurate drilling to avoid hitting any anchors. Additional information on ground conditions were also determined from the drilling of the boreholes. Two piezometers were installed in each borehole to allow the monitoring of perched water tables.

Two groundwater horizons were monitored; a shallow level approximately 10 m to 15 m above the roof of the anchor drive tunnel, and a deep level approximately 10 m deeper than the shallow level. For the installation of the piezometers 90 m of cored drilling was carried out at the 4 locations. Whilst groundwater levels fluctuated, the levels recorded in 4 of the piezometers reduced between 1 – 2 m in the six months after installation.

## 8. ANCHOR DRIVE TUNNEL DRAINAGE

Drainage holes were created during the construction of the anchor drive tunnels to provide drainage of the bedrock behind the retaining walls. The holes were drilled from the anchor drive tunnels outwards into the bedrock. Water drains into the anchor drive tunnel and collects in a sump, which is joined to the city's underground drainage network. As-built drawings stated the drainage holes in the walls of the anchor drive tunnels were 7.6 m in length, and 4.5 m from the ground surface in the roof (equivalent to 21.5 m in length for a portion of the holes). However, during the contract it was found that all of the drainage holes in both anchor drive tunnels were shorter than indicated, and were essentially acting as weep holes through the concrete tunnel lining.

Prior to the commencement of any works in the anchor drive tunnels a survey was undertaken recording the length, material observed and the amount of seepage of each hole in the tunnels. The holes were also individually numbered. In the northwest anchor drive tunnel 438 holes were recorded in the walls. In the southwest anchor drive tunnel 52 holes were recorded in the walls. The hole lengths varied from 0.0 m to 6.0 m regularly during the works. Material observed in the drainage holes included grout, calcite and greywacke.

The amount of seepage from each hole was recorded on and classified according to the rate at which water was flowing from the hole. The seepage was classified as either dry, damp, dripping, slow flow, moderate flow or fast flow.

The drainage holes in the anchor drive tunnels were cleaned by a combination of high pressure water (approximately 3000 psi at nozzle head), reaming and drilling. The high pressure water cleaning removed a large amount of calcite that had built up in and around the drainage hole, and removed loose debris. The high pressure water jet was also used to remove stalactites that had formed and calcite that had built up below the holes on the side walls. Seepage in the anchor drive tunnels increased after high pressure water cleaning. Drilling was carried out in the existing hole to re-open

partially blocked holes to their original diameter and length by removing the more stubborn calcite and debris. The lengths of 97 of the drainage holes were increased by drilling.

Drilling resulted in a significant improvement of the anchor drive drainage, demonstrated by increased flow of groundwater through the drainage holes, and lower groundwater levels recorded in the piezometers.

A number of the drainage holes had partially collapsed. These drainage holes were redrilled and slotted PVC pipes were placed into these holes. The pipes were wedged into the hole and not grouted in to allow for easier removal during future maintenance.

The amount of calcite being released and transported by the groundwater was greater than expected with calcite growing on the drilling rigs after 2.5 months in the anchor drive tunnels. Stalactite growth had resumed and calcite was observed to be building up in the cleaned and drilled holes. Because of the fast build up of the calcite, the time gap between recommended cleaning (with high pressure water) was reduced, and the cleaning was repeated recently.

Figure 4: Stalactite Growth

Figure 5: Southwest Anchor Drive Tunnel

## 9. ANCHOR TESTING & MAINTENANCE

The testing and maintenance of the anchors in the 2nd contract was carried out by Construction Techniques Ltd (Contech), (who carried out the original installation), and included;

- Checking residual pre-stress in selected anchors,
- Load testing anchor bars,
- Lowering selected anchors into anchor drive,
- Withdrawal of entire length of selected anchor bars,
- Reinstatement of anchor bars, plates and re-stressing the anchor,
- Modification of anchor heads in anchor drives,
- Removal and renewal of corrosion protection of anchor heads,
- Cleaning of anchor head drainage holes,
- Endoscopic inspection of selected anchor bars behind head.

During testing of the northwest tie-down anchors, one anchor failed. This anchor was replaced by a 40 mm Macalloy bar immediately adjacent to the failed anchor. Due to this failure, all subsequent tie-down 35 mm Macalloy anchors were tested and locked off at a slightly lower prestress load.

As part of the recent contracts 6 anchors were completely withdrawn to check for corrosion. To enable this to occur SH1 at the Terrace Tunnel had to be completely closed on three separate occasions (all night time closures), as the length of the anchor bars exceeded the width of the 4 lane highway.

Light rusting of the bar thread near the anchor heads was found under some previous heat shrink repairs, but this did not extend under the original "yellow jacket" corrosion protection system. Two couplers of the withdrawn anchor bars were exposed and uncoupled and found to be in good condition.

All anchor heads within the anchor drive tunnels had the existing tape corrosion protection removed and replaced with primer, mastic and densotape.

Corrosion had occurred to the anchor head cover boxes at the southwest wall face in the bottom row, as they are located below ground level and exposed to groundwater. All 6 of the boxes in this row had corroded sufficiently to allow the grease within the box that protects the anchor head to escape. All of these cover boxes were replaced with new galvanised steel boxes and packed with denso mastic instead of grease to protect the anchor heads and anchor plates from corrosion.

The condition of the anchor bars after nearly 30 years was excellent. The condition of the anchor heads was also generally good. It was noted that the presence of the grease did make inspection and maintenance difficult. Also where any corrosion had occurred in the anchor head boxes in a number of instances the grease had escaped. It was decided subsequently that all grease should be removed from the anchor drive tunnels. All anchor heads in the drives and those exposed during the contract at the wall face were covered with primer, mastic and densotape. The good condition and successful testing of the anchors allowed the time period between inspections to be increased. It should be noted that inspections and maintenance of the anchors is staggered with one third of the anchors being attended to at a time.

## 10. CONCLUSION

The recent inspection and maintenance contracts allowed the Maintenance Strategy to be updated and a number of assumptions regarding the condition of the asset and the timing of future maintenance to be confirmed or amended.

The good condition of the anchors supporting the approach walls is encouraging for the future performance of the walls, and the recent enhancement to the anchor head corrosion protection will continue to protect the anchors.

The reduction in groundwater levels behind the walls has a two fold benefit. Firstly the decreased amount of water in the bedrock has improved stability of the slopes behind the walls; pore water pressures are reduced leading to greater stability in seismic and non seismic conditions. Less groundwater in the material behind the walls also reduces the likelihood of corrosion in the anchors.

All of these benefits lead to a more reliable structure and road.

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# GEOTECHNICAL ASPECTS OF THE CONSTRUCTION OF THE WIVENHOE DAM SPILLWAY

**Cynthia de Bok**
*Coffey Geotechnics Pty Ltd*
*Brisbane*

## ABSTRACT

The Wivenhoe Dam Flood Security Upgrade involved the construction of an additional spillway through the right abutment of the main dam. The spillway is cut through massive sandstones and siltstones of Jurassic and Upper Triassic age.

This paper covers geotechnical design and construction issues encountered during construction, including mapping of up to 17m high cut faces and the treatment of the spillway walls, including the design of anchors and shotcrete wall lining.

During the construction of the spillway the design of the spillway was significantly modified as a result of exposed conditions. Variations from anticipated geological conditions included the presence of fill up to 7.8m thick found over more than 100m length of the right hand side wall of the spillway chute, and the presence of a thick layer of extremely weathered sandstone and highly weathered siltstone underlying this fill.

The Alliance form of contract ensured that due to the presence of the designers on site, the design could be modified when required in conjunction with the construction team.

## 1. INTRODUCTION

The Wivenhoe Dam is located 80km northwest of Brisbane on the Brisbane River and is the largest water storage dam in South East Queensland. The dam was completed in 1985 as a flood mitigation and water storage dam. Updated rainfall predictions, which increased estimates for extreme flood events, resulted in unacceptable flood risk for Wivenhoe Dam. The Wivenhoe Dam with one primary spillway at the centre of the dam was not able to pass the latest estimate of the Probable Maximum Flood (PMF). The existing spillway capacity was insufficient, which led to the potential for overtopping of the dam to occur during extreme events. Therefore the Wivenhoe Dam Flood Security Upgrade Project was created to design and construct works to allow the Wivenhoe Dam to pass the PMF.

In the design phase, many options were evaluated including the construction of a spillway at each saddle dam and construction of a secondary spillway at the right abutment of the main dam. The option that was chosen was:

- Excavation of a secondary spillway at the right abutment of the existing dam with the inclusion of three fuse plug embankments and a bridge across the spillway for Brisbane Valley Highway traffic,
- Upgrade of the existing gated spillway to accommodate an increase in the Maximum Flood Level (MFL), and
- Upgrade of the crest of the dam to reduce piping risk and raise the existing concrete traffic barrier.

This option increased flood discharge capacity from about 13,000$m^3$/s to about 28,000$m^3$/s.

The secondary spillway is located on the right abutment of the existing dam, which is on the west side of the dam. Figure 1 shows the layout of the secondary spillway. The upstream side of the spillway is to the north. The construction of the secondary spillway comprised the following:

- Excavation of a 165m wide spillway chute to 64m RL, up to 20m deep,
- Placement of dowels, mesh and shotcrete lining on the right hand side (RHS, west) and left hand side (LHS, east) spillway walls (training walls),
- Construction of a concrete ogee crest and cantilever dividing walls between the three fuse plugs,
- Construction of three fuse plugs; each comprising an earth fill dam with a sloping clay core, sand and gravel filters and rock fill supporting shell,

- Construction of a two lane bridge across the spillway

This paper covers some geotechnical aspects of the construction of the secondary spillway excavation and training walls.

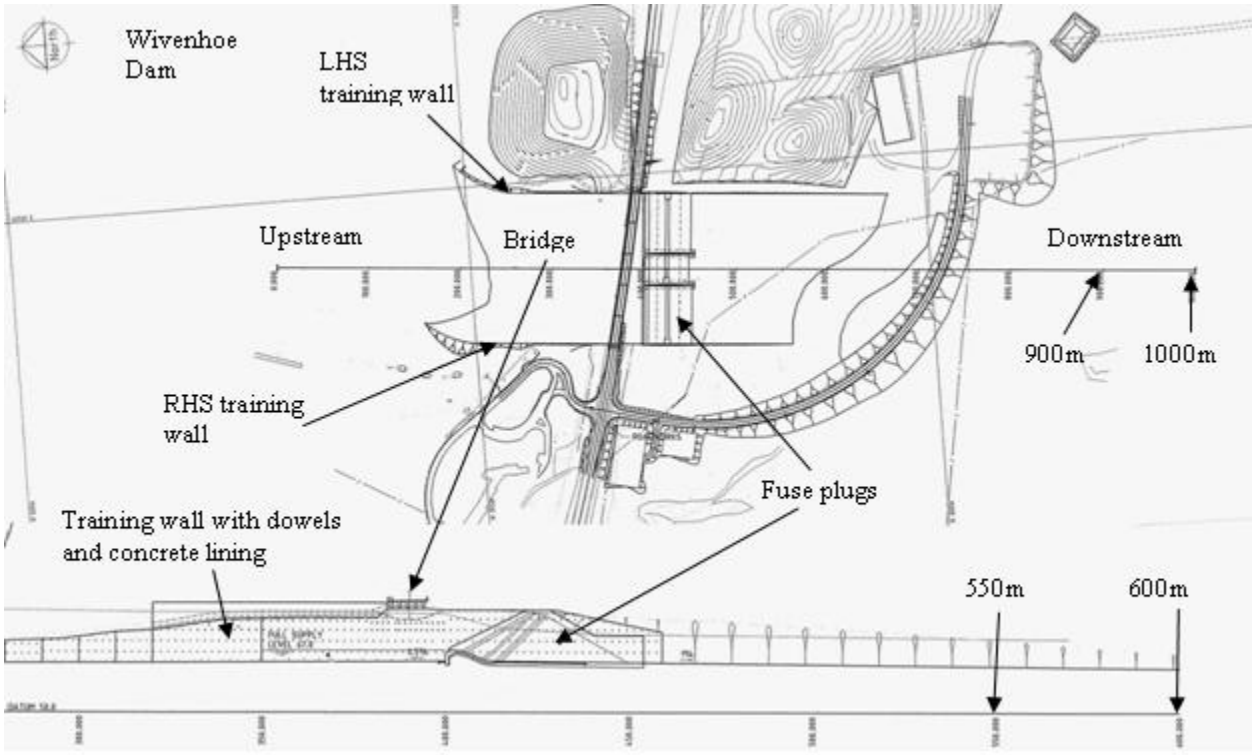


Figure 1: General arrangement of secondary spillway.

## 2. GEOLOGY

The regional geology of the Wivenhoe Dam area is predominantly composed of weathered sedimentary rocks of Triassic to Jurassic age.

The right abutment area where the secondary spillway was constructed generally comprises residual soil and some localised fill, underlain by weathered sandstone and siltstone beds of the Lower Jurassic Marburg and Upper Triassic Woogaroo Subgroups. The bedding is generally thick with occasionally thinner layers of siltstone or thinly interbedded layers of sandstone and siltstone and is orientated near-horizontal or dips to the south and south east at around 2° to 5°. The sandstone is generally low to medium strength, becoming medium to high strength further towards the base of the excavation. Some minor faults were observed orientated northeast to southwest of which the dip could not be measured. The joint spacing is generally several meters.

## 3. GEOLOGICAL MAPPING

Throughout the excavation process exposed geology to be covered by shotcrete after construction of the spillway was mapped by an on site geologist and photographed to:

- Verify the integrity of the foundation as a water retaining medium,
- Identify and treat erodable materials and seams within the rock mass,
- Collect geological data for as built records,
- Adapt the design to the encountered geology, if different from expected geology.

The mapped areas included the spillway walls and wall footings, and the ogee crests (at the fuse plugs). All collected data during mapping was presented in CAD drawings and the photographs were archived.

During the construction phase fill was encountered on the right hand side (RHS) of the spillway upstream from the bridge. The fill appeared to comprise layers of sandstone fill, with occasionally a clay layer or sandy clay layer. At

some locations the bottom layer of fill appeared to be a grey soft sandy clay layer up to 0.5m thick. The fill, which was shown to be up to 7.8m thick, was placed during the original construction of the dam to construct the access road to the office and associated buildings.

The fill is underlain by highly weathered (HW) sandstone and extremely weathered (XW) sandstone/siltstone. The XW sandstone/ siltstone is generally a red high plasticity clay up to 5m thick. The HW sandstone on the RHS training wall is underlain by siltstone. The siltstone was HW and of very low to low strength. The bedding orientation as measured in the cleaned footing foundation generally dips to the northwest at less than 20°. Test results on a sample taken from the footing indicate that the siltstone is of medium plasticity and is highly dispersive.

Figure 2 shows the boundaries between the different layers of fill, sandstone and siltstone as encountered in the RHS wall.

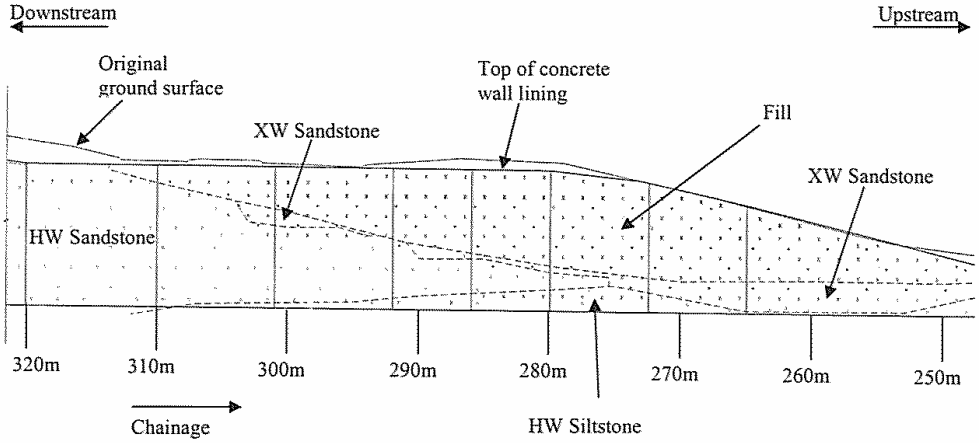


Figure 2: Encountered geology at the RHS training wall.

## 4. DESIGN AND TREATMENT OF SPILLWAY WALLS

### INITIAL DESIGN

The spillway wall height varies from about 20m high at the bridge location to 0m at the upstream and downstream ends of the outer wall. The training walls were designed to be vertical through the bridge location, and to have a 1H:10V slope downstream from the ogee crest and upstream of the bridge. Less steep slopes (1H:2V) were used upstream from the 1H:10V slopes.

The ground investigations carried out before the construction phase indicated that the training walls were located in an area of highly to moderately weathered sandstone. Based on assessed geotechnical parameters of the sandstone and stability analysis it was expected that the stability of the training walls would be sufficient without any additional measures.

However the spillway walls needed to be protected from the high velocity flows during operation of the spillway. 3m long N32 anchor bars (dowels) were grouted at 2m horizontal and 3m vertical intervals in a 90mm diameter hole drilled at 30° below the horizontal. A welded reinforcing mesh (consisting of N20 bars at 200 centres) was attached to the wall using the dowels and the walls were covered by 100mm to 300mm thickness of shotcrete.

### DESIGN CHANGE 1 – FILL ENCOUNTERED

The excavation of the spillway and the treatment of the training walls on the downstream side of the bridge were carried out in 2004 and were generally carried out according to the design. In 2005 the excavation of the spillway upstream from the bridge was started. This was the last section of the spillway that required excavation.

In April 2005 fill was encountered on the RHS training wall. The exposed fill was up to 5m thick and was encountered along 45m length of the training wall. None of the boreholes carried out in earlier phases of the project had encountered more than 1m depth of fill and therefore the extent and thickness of this deeper fill was unknown.

Test pits were excavated from the then existing excavation floor to determine the base of the fill. Highly weathered sandstone was encountered at 2.2m depth below the excavation floor at 67.8m RL. Based on this information and an outcrop further upstream showing the base of the fill, it was expected that the fill would gently slope down from 68m RL at CH290 to 67.2m RL at CH255 (see Figure 3, geotechnical model 1). Based on the construction plans for the original Wivenhoe Dam, it appeared that the fill was placed for the construction of an access road to accompany building of the dam. However no information of the placement of the fill was found and therefore the fill was assumed to be uncontrolled.

Based on this information a new design for the RHS training wall was required to stabilise the fill. The fill was thought to be up to 5m thick and would be the top part of an 11m high training wall. Several options such as gravity wall, L-shaped retaining wall, anchors, dead man with anchors, soil nailing and a shotcrete wall tied back with anchors were assessed. Due to the pressure of further activities at the top of the training wall, timing issues and the nature of the fill, the option of a shotcrete wall tied back with anchors was further developed.

The principle of the shotcrete tieback wall is that the mesh and concrete wall is tied back into the sandstone with anchors to hold the fill in place. Due to the nature of the fill no strength could be assumed in the fill and therefore the anchors were to be anchored into the underlying sandstone. Assessments showed that a 3m bond length would be sufficient, as this would equal the anchor bar strength.

Assessments indicated that the top anchor at 73.5m RL should be 14m long for 3m embedment length in sandstone if placed at 30° below the horizontal and 11m long for a placement of 45° below the horizontal. Anchors placed at 30° below the horizontal were preferable, but 14m long anchors would require special transport. Therefore, the longest length of anchor that can be transported using normal road transport was chosen, which is 12m long. Due to this constraint the anchors were placed at 45° below the horizontal. Analysis of anchors both in shear and tension were carried out and this proved that anchors at 45° below the horizontal were sufficient.

Slope stability analysis using 12m long anchors at the top row and the soil and rock parameters shown in Table 1 indicated that 5 rows of anchors placed at 1.5m horizontal and vertical spacing with length from top to bottom of 12m, 10m, 8m, 6m and 5m would be sufficient.

On 17 May 2005 the installation of the anchors was started, only 2.5 weeks after the discovery of the fill.

| Unit | Phi (degrees) | Cohesion (kPa) | Allowable Bond Stress (kPa) |
|---|---|---|---|
| Fill | 33 | 0 | 0 |
| XW sandstone | 40 | 5 | 50 |
| HW sandstone | 40 | 200 | 750 |

Table 1: Geotechnical parameters of geotechnical units

## DESIGN CHANGE 2 – EXTREMELY WEATHERED SANDSTONE ENCOUNTERED

On 20 May 2005 two more test pits were carried out further upstream from the previous one to provide information to refine and check the current design. In the test pits extremely weathered (XW) sandstone up to 5m thick was encountered upstream from CH290 (See Figure 3, geotechnical model 2). Due to this discovery all the previous calculations had to be checked to analyse the effect of the XW sandstone. The geotechnical parameters used for the XW sandstone are shown in Table 1. The analyses indicated that the anchor lengths used in the design were sufficient and on 25 May 2005 all the anchors above 70m RL were installed.

## DESIGN CHANGE 3 - CHANGE IN FILL BOUNDARIES

On 28 May 2005 further excavation exposed red clay that was thought to be the extremely weathered sandstone. This resulted in an assumed fill boundary at about 69m RL instead of about 66.5m RL (see Figure 3, geotechnical model 3). It was also determined that the bond stress instead of the bond resistance was used in the stability calculations. The bond resistance used in further stability calculations is shown in Table 2. All analyses were redone incorporating these changes. Because all anchors above 70m RL were already installed anchor lengths could only be changed for anchors

below 70m RL. The lengths of most anchors were increased by 1 to 2m and an additional fifth row of anchors of 4 to 6m lengths was introduced to satisfy the design criteria.

| Unit | Phi (degrees) | Cohesion (kPa) | Allowable Bond Resistance (kN/m length) |
|---|---|---|---|
| Fill | 33 | 0 | 0 |
| XW sandstone | 40 | 5 | 15 |
| HW sandstone | 40 | 200 | 200 |

Table 2: Geotechnical parameters of geotechnical units

**DESIGN CHANGE 4 – SHOTCRETE STRENGTH INCORPORATED IN ANALYSIS**

In all previous slope stability calculations the strength of the shotcrete was not incorporated. As the shotcrete wall is a part of the retaining structure, it was decided to include the strength of the shotcrete in the slope stability design. The shotcrete wall was incorporated as a 100mm thick wall with a cohesion of 1000kPa. This change improved the previous slope stability assessments.

Figure 4: Boundary of Fill and XW sandstone on RHS training wall.

**DESIGN CHANGE 5 - CHANGE IN FILL BOUNDARIES**

By 20 June 2005 the RHS spillway was further excavated to below 69m RL. The red clay bands previously observed were now fully exposed and it appeared that this clay was part of the fill and not the XW sandstone as first thought. The new geotechnical model is shown in geotechnical model 4, Figure 3. Based on these findings the wall design had to be reassessed. Despite the changes the design criteria were still met. A photo showing the boundary between the fill and the XW sandstone is shown in Figure 4.

**DESIGN CHANGE 6 – SILTSTONE ENCOUNTERED**

At the end of June 2005 the base of the spillway was exposed, which therefore exposed the full depth of the wall on the RHS of the spillway. It showed the presence of an 5m thick siltstone layer below the sandstone underlying the fill (see Figure 3, geotechnical model 5). The siltstone is highly weathered. Several anchors which were thought to be bonded in sandstone were now shown to be bonded in siltstone. An allowable bond resistance of 30kN/m length in HW siltstone was assumed and the slope stability of the affected area was reassessed. The analyses showed that the design criteria

were still satisfied. Though overall stability was satisfied additional anchors were placed through the siltstone layer for extra support.

In July 2005 the mesh was placed on the walls and the walls were shotcreted.

## 5. CONCLUSIONS

During the construction phase of the Wivenhoe Dam upgrade project several geotechnical difficulties were encountered, such as fill being present at the RHS training wall instead of sandstone. Being in the construction phase, it was essential that any geotechnical issues were solved without delaying the construction team. The alliance project structure allowed this to happen and had the following advantages:

- An on site integrated project team including geotechnical staff ensured a quick turn around on design issues and identification of variations in design assumptions in a timely manner. This allowed design staff to be proactive in developing design changes and avoiding on site delays.
- The alliance delivery method allowed these issues to be dealt with in a cooperative environment as opposed to the traditional adversarial contract.
- Cost effective, practical and pragmatic designs were achieved by having immediate feedback on safety, material availability and costs from construction staff based in the same office as the design staff.
- Designs involving the use of similar technology (eg the use of longer anchors instead of concrete walls) across the site provided major cost benefits.

Despite a major variation between the expected and actual ground conditions the project was finished ahead of schedule and within budget. A photo of the complete secondary spillway is shown in Figure 5.

Figure 5: The completed secondary spillway at the right abutment of the Wivenhoe Dam

## 6. ACKNOWLEDGEMENTS

I would like to thank the Alliance members, the owner SEQ Water, Leighton, Department of Commerce, MWH, and other Coffey staff for their input during the geotechnical challenges. I would like to thank Phil Styles and Barton Maher in particular for their support during the project.

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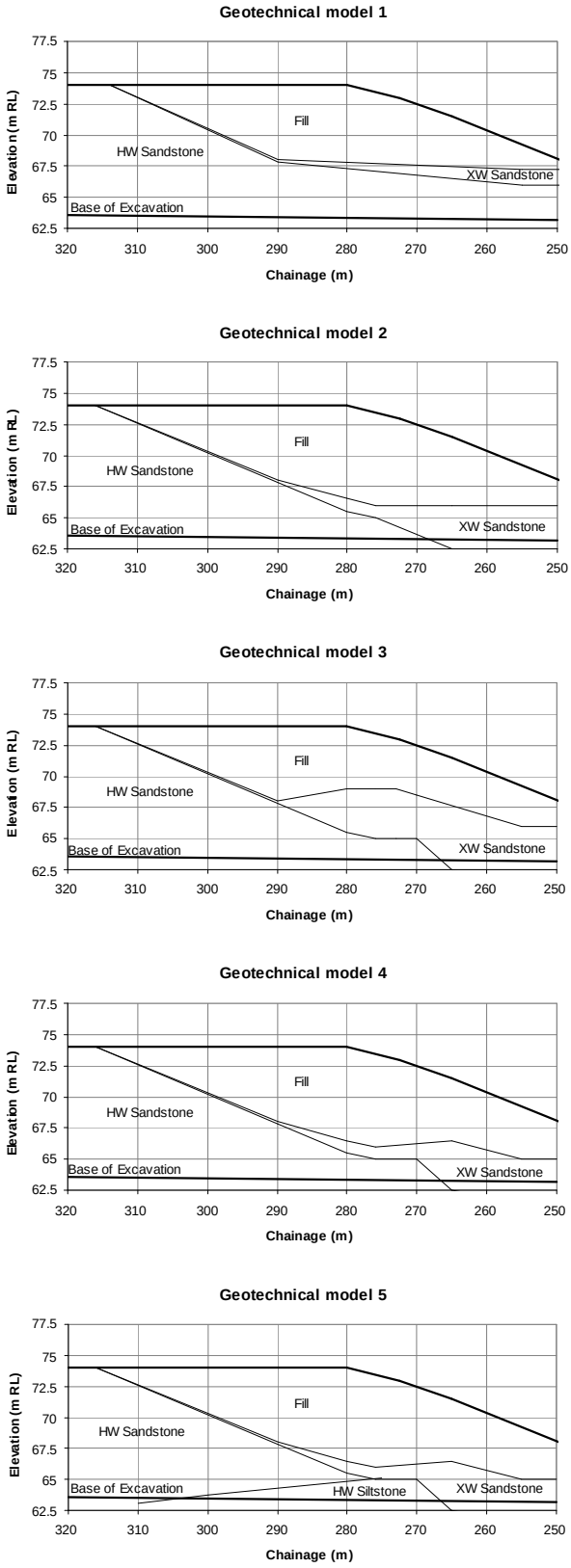


Figure 3: Changes in geotechnical model during construction.

# VERIFICATION OF ROCK SOCKET DESIGN DURING CONSTRUCTION

**C.P. Every**
*Northern Gateway Alliance*
*Orewa, New Zealand*

## ABSTRACT

The design of rock socket piles requires a good knowledge of the subsurface ground conditions for each pile or group of piles. The amount of geotechnical information available will determine the engineer's confidence in the design as well as the level of conservatism and choice of appropriate strength reduction factors for the design. An assumption about the quality of the rock socket construction also needs to be made at the design stage.

This paper outlines the methods employed to verify the rock socket design on the ALPURT B2 project. The ALPURT B2 project is the extension of Auckland's Northern Motorway, from Orewa to Puhoi, on State Highway 1.

Brief outlines of the investigation phase, rock socket design and pre-construction pile load tests are provided. Details of the design verification methods used during construction are presented along with methods of reviewing and modifying the pile design at the time of construction.

## 1. INTRODUCTION

ALPURT B2 is the extension of Auckland's Northern Motorway from Orewa through to Puhoi. This extension will form the "northern gateway" to the greater Auckland urban area. Design, construction, consenting and overall project delivery has been undertaken by the Northern Gateway Alliance (NGA).

The 7.5 km motorway includes two eco-viaducts passing over the environmentally significant Nukumea and Otanerua valleys, twin cantilever viaducts over the Waiwera River estuary and tidal mud flats, overhead bridges at Hillcrest Road and Orewa Interchange, widening of the existing Titfords Bridge at the northern end of the project, and twin tunnels through Johnstone's Hill.

The ALPURT B2 route passes through the geological boundary between the Pakiri Formation and the East Coast Bays Formation, both of which are part of the sedimentary Waitemata Group. The Pakiri Formation comprises older stronger Waitemata Group materials with included volcaniclastic sandstones and conglomerates. Pakiri Formation rock strength is variable with unconfined compressive strengths (UCS) ranging from 1MPa to over 20MPa. East Coast Bays Formation (ECBF) comprises extremely weak to weak sandstone and interbedded extremely weak to weak siltstone.

A total of 277 permanent piles are required for the ALPURT B2 project. Pile diameters range from 600mm to 1800mm and piles depths range from a few meters to over 25m.

Most piles (with the exception of Hillcrest over-bridge and Titfords bridge widening piles) were drilled using a 60 tonne CMV piling rig. The rig is capable of drilling up to 2.2m diameter bored piles, to a depth of 50m.

Figure 1. Artists impression of the twin Waiwera Viaducts and Johnstone's Hill tunnels.

# 2. ROCK SOCKET DESIGN METHODOLOGY

Bored pile foundations for the ALPURT B2 Project were designed using a 'complete socket design' (CSD) method (Sinclair and Every, 2006). This design method assumes that the pile socket derives its capacity from all surrounding rock (shaft friction and end bearing). A brief summary of the CSD method and the key rock parameters required are given below.

## 2.1. DETERMINATION OF ULTIMATE SIDE SHEAR CAPACITY

To determine the rock side shear capacity ($f_s$), three primary factors are considered, these being:

- Unconfined compressive strength (UCS), denoted by $q_u$
- Rock quality factor $\beta$. Rock quality factor is related to the rock mass quality (RQD) or rock mass factor 'j' ($j = E_{mass}/E_{intact}$) by the following proposed relationships:

  $\beta = 0.35+0.0065 \times RQD$ or $\beta = j^{0.23}$
- Roughness coefficient '*b*', which is dependent on roughness and cleanliness of the socket sidewall. '*b*' ranges from 0.6 to 1.9 for a very smooth and rough socket respectively (Pells, 1999).

From the above factors, a relationship for ultimate side shear has been developed:

$$\frac{f_s}{p_a} = b.\beta\left(\frac{q_u}{p_a}\right)^{0.5} \quad (1)$$

This is the dimensionless form of the equation where $p_a$ is atmospheric pressure (101.3 kPa).

## 2.2. DETERMINATION OF ULTIMATE END BEARING RESISTANCE

A relationship for the ultimate end bearing capacity ($q_b$) has been developed based on data from Lianyang Zhang & Einstein (1998). The primary factors used to determine the end bearing capacity are the unconfined compressive strength ($q_u$) and the rock quality factor $\beta$. Roughness coefficient *'b'* is no longer required. From these factors, a relationship for ultimate end bearing has been developed:

$$\frac{q_b}{p_a} = 22.2 \times \beta\left(\frac{q_u}{p_a}\right)^{0.5} \quad (2)$$

This is the dimensionless form of the equation where $p_a$ is atmospheric pressure (101.3 kPa).

## 2.3. COMBINED END BEARING AND SIDE SHEAR CAPACITY

The CSD method assumes that Serviceability Limit State (SLS) loads may be carried by a combination of side shear resistance and a proportion of end bearing (in the elastic range, without sideslip). The required socket length and associated settlements are calculated using design charts provided by Pells. Ultimate Limit State (ULS) capacity is determined by addition of the ultimate side shear capacity plus the ultimate end bearing capacity, with appropriate strength reduction factors applied.

Rock mass modulus $E_{mass}$ was determined from statistical analysis of site-specific UCS vs $E_{intact}$ plots with an adjustment for RQD. No insitu rock stress-strain tests were carried out.

## 2.4. FINAL ROCK SOCKET DESIGN

The final rock socket length was taken as the maximum of either; the rock socket length calculated using the CSD method above including negative skin friction loads, or the rock socket length required for uplift capacity (tension), or a self imposed design minimum pile socket length of 3D (where D=pile diameter), or special requirements of the external peer reviewer.

### 2.5. PRE-CONSTRUCION PILE LOAD TESTING

A test programme comprising two 1.5m diameter (full scale) test piles was undertaken in the Waiwera Estuary during August – October 2005. Test loads in excess of 40MN were reached using Osterberg Cells (O-Cells) embedded in the rock socket of two test piles. As a result of the pile load tests, significant cost savings were made on the permanent foundation piles for the Waiwera Viaduct. Figure 2 shows equivalent top load displacement graphs from the test piles.

Full details of the pile load tests are described by Every et al (2006).

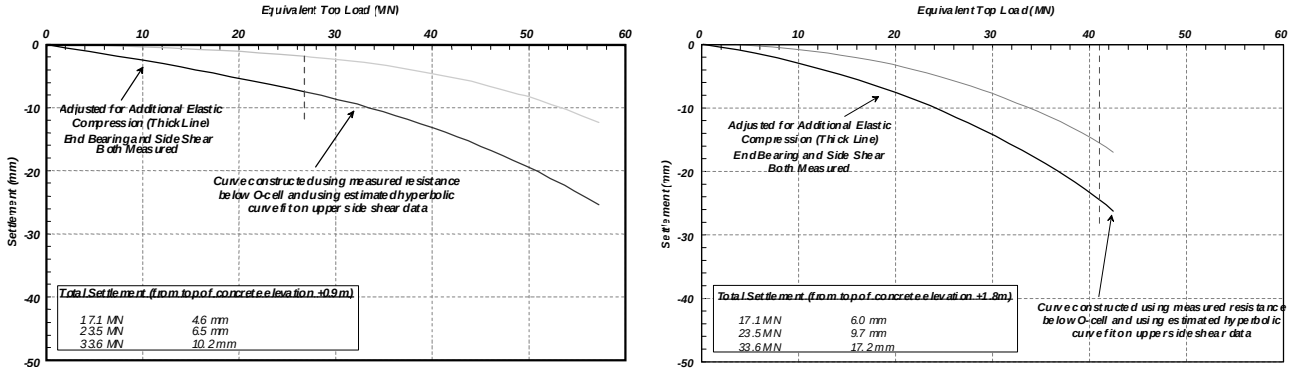


Figure 2. Top load displacement graphs from the two test piles at Waiwera Viaduct

## 3. PILE SOCKET DESIGN VERIFICATION

Verification of the pile socket design was an essential part of the complete socket design and construction process. It was important to be able to vary the designs according to the actual conditions encountered.

### 3.1. PILE INSPECTIONS

Pile inspections were carried out by the NGA geotechnical engineer and NGA piling engineer. Any variations to the design parameters due to unforeseen ground conditions needed to be quickly checked so that the constructor was not unduly held up. A pile design spreadsheet was developed to help speed up this process and changes to the pile socket design could normally be made on-site with little or no delay to the constructor.

Design parameters that needed to be checked during construction included:

- Top of rock level,
- Socket roughness ($b$),
- Rock mass quality or RQD ($\beta$),
- Rock strength ($q_u$),
- Geological and construction defects.

#### 3.1.1. TOP OF ROCK LEVEL

The top of rock level affects the overall pile length and also the spring stiffness values assumed in the structural model. This was significant in the NGA bridge structures which are designed for seismic loads (0.24g) and large bending moments and shear forces in the piles.

If the actual top of rock varied significantly from the assumed design rock level, either an extension was added to the cage or additional longitudinal bars were added. The structural model was also checked to ensure that this did not significantly affect other parts of the bridge structure.

#### 3.1.2. ROCK STRENGTH

Rock strength was tested by the way of UCS tests from investigation and proof bore holes and by point load tests on selected cuttings from the pile hole. A point load UCS to $I_{s50}$ correlation factor of 24 was initially chosen. However results from the point load tests on pile cuttings and UCS tests carried out during the investigation phase indicated that this correlation may be closer to 13 for the Waitemata Group rocks.

Changes in rock strength affect the available rock socket capacity and rock stiffness. Rock strengths less than assumed in the design have a direct effect on both the geotechnical and structural model by reducing the available socket capacity and stiffness of the rock socket. Where the actual rock strength was found to be lower than assumed, the capacity of the rock socket was recalculated using the CSD method described earlier. The roughness and RQD also needed to be confirmed prior to recalculating the capacity. In some cases, no modification to the socket was required where the roughness or RQD were better than assumed and the overall capacity was not reduced.

### 3.1.3. SOCKET ROUGHNESS

For design purposes, the initial design roughness chosen for all piles was R3 ($b$=1.4). Observations during piling indicated that the grooving equipment was able to achieve a roughness equivalent to R4 ($b$=1.9). However in some ground conditions the grooving tool became blocked with clay. When this occurred, the grooving tool was cleaned and grooving was re-attempted. If it was still not possible to groove effectively, any smear on the pile socket was removed using a water blaster attachment connected to the kelly bar and the pile socket length was redesigned using a lower roughness coefficient.

The fine-grained siltstones of the Waitemata Group caused excessive smear in piles drilled with little or no ground water present. Piles with high water tables generally did not have a smear problem as the agitation of the cleaning bucket and grooving tool would wash the smear off. The use of a water blaster proved to be a very quick and cost efficient way of removing smear from the socket sidewalls. The water blaster comprised a petrol powered water blaster, which was also used to clean the steel reinforcing cages, and an attachment on the bottom of the kelly bar. Cleaning took only several minutes and used approximately 100 litres of water.

### 3.1.4. ROCK MASS QUALITY

Rock mass quality given by fracture spacing or RQD was the most difficult factor to quantify to determine during construction.

Rock mass quality was determined by borehole logs and confirmation was done by carrying out down hole inspections or, where possible, using a CCTV camera.

In most cases the RQD did not need adjustment during construction. On a couple of occasions the actual RQD was clearly lower than assumed in design and the pile capacity was recalculated. Changes to the pile length were made as required.

### 3.1.5. GEOLOGICAL AND CONSTRUCTION DEFECTS

Any imperfections in the pile socket due to geological defects or construction defects were also checked. These include (Tomlinson, 1999, Poulos, 2005):

- Piles bearing on competent rock but with fractured rock or weak layers below the toe. The risk of this was minimised by carrying out pre-construction proof boreholes on selected piles and pile groups to a depth of 3D below the toe of the pile(s).
- Soft toe due to inadequate base cleaning. The risk of soft toe was reduced by ensuring that the base was cleaned and tested using a lead weight or plumb bob immediately prior to the concrete pour. The maximum acceptable sediment depth for the ALPURT B2 piles was 50mm (typically 5mm to 20mm depth was recorded).
- Lumps of clay falling from the walls of the borehole into the concrete as it is being placed. This was remedied by extending casing to competent rock.
- Infiltration of ground water causing gaps or honeycomb in the concrete. This was remedied by correct tremmie pouring of concrete or where water inflows were high, allowing the water level to stabilise prior to pouring concrete
- Hardened soil or concrete on the inside of the lining can cause the concrete to arch and be lifted as the casing is withdrawn causing voids in the concrete. This was remedied by using a concrete with sufficient workability (150 mm to 230 mm slump) and ensuring that the inside of the casings were clean before they were installed.

At Waiwera River estuary we were unable to pump most of the piles dry for inspection. However, based on our earlier experience at Otanerua Viaduct, where we were able to inspect all the piles, we knew that the drilling, grooving and cleaning methods consistently produced a good quality rock socket. Therefore at Waiwera, using the same piling rig and piling crew, we were confident that we had a good rock socket and it was only the rock levels, rock strength and base cleanliness that needed to be checked during the pile inspections.

### 3.2. METHODS OF INSPECTING THE ROCK SOCKET

NGA engineers were located on the construction site and all piles were inspected to some degree. Piles were inspected during the drilling and immediately prior to concrete pouring. Pile check-sheets were completed for all piles. These contained information regarding the diameter and depth of the pile, the size of ULS and SLS design loads, the assumed geotechnical ground conditions, and the rock design parameters. Inspections comprised the following:

#### 3.2.1. SPOIL SAMPLING AND POINT LOAD TESTING OF CUTTING SAMPLES.

This is a convenient way of confirming the top of rock and rock strength ($q_u$) but involves time and some uncertainty. Samples from the drilling auger or bucket are disturbed and in very weak to weak rock it can be difficult to determine the top of rock level. After some experience with the CMV rig, it was possible to tell when the machine was 'working harder' to drill and this was sometimes the only indication of being in good rock. Irregular shaped samples for the point load tests (PLT) needed careful selection to carry out the tests.

#### 3.2.2. PHYSICAL DOWN-HOLE INSPECTIONS USING BOSUN'S CHAIR OR INSPECTION CAGE.

This is the best way of observing the completed rock socket. Rock mass quality or RQD ($\beta$), rock strength ($q_u$) and socket roughness ($b$) can all be assessed during the down hole inspection. However inspections require a long setup time and personnel trained in confined access safety procedures. The minimum pile diameter we could safely inspect was approximately 900 mm diameter. This allows an 800 mm diameter inspection cage to be installed over the uncased length of the pile. Figure 3 shows a down hole inspection set up at Otanerua Viaduct.

#### 3.2.3. CCTV VIDEO RECORDING.

A CCTV camera system was developed by NGA engineers. The camera attached to the kelly bar and was operated by a control panel and laptop computer at the surface. This inspection procedure reduced the time to complete an inspection to approximately 20 minutes per pile. Confirmation of the roughness ($b$) of the sidewall and construction quality was easily determined with the CCTV camera. Another advantage of this method is the elimination of the health and safety risks associated with the physical down hole inspection. A disadvantage of the CCTV inspection is that it does not give the rock strength and it is difficult to determine the rock mass quality. Figure 4 shows the NGA CCTV camera.

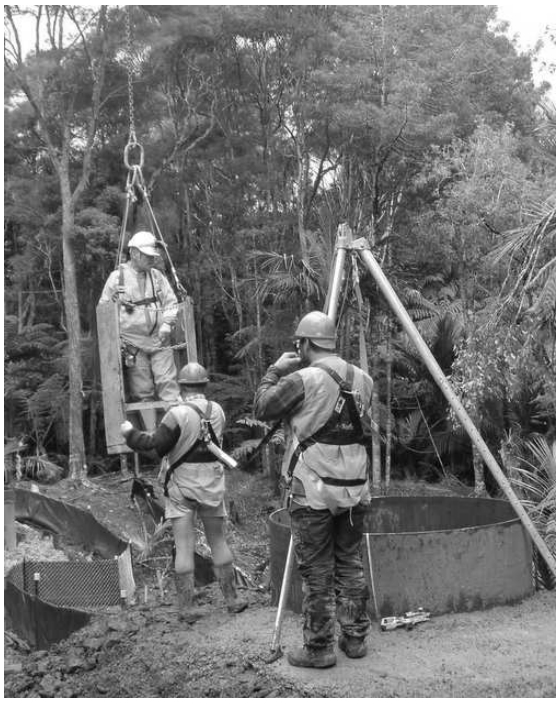

Figure 3. Physical down-hole inspection of pile.

Figure 4. CCTV inspection camera attached to kelly bar.

### 3.2.4. DIGITAL STILL IMAGES OF THE PILE SHAFT USING A DIGITAL CAMERA

This simple method was used prior to the introduction of the CCTV camera. The camera was lowered down the hole on a measuring tape with the timer set. It was possible to determine the roughness of the sidewall with this method, however the overall quality of the rock socket was very difficult to determine.

## 4. CONCLUSIONS

Verification of the rock socket design an essential part of the complete socket design and construction process for all foundation piles.

The supervising Geotechnical Engineer needs to have an understanding of the pile design assumptions and key design parameters that need to be confirmed during construction. These parameters include the depth to rock, rock strength ($q_u$), roughness ($b$) and rock mass quality or RQD ($\beta$) . The allowable variation in these parameters should be known and if changes to the design are required, this should be done quickly using a spreadsheet or hand calculations so as not to unduly hold up the constructor.

The various methods used to check these parameters include down-hole inspections, CCTV video recording, spoil sampling and testing, and digital still camera photos.

Geotechnical engineers should also be aware of the risks during concrete pouring, including contamination of concrete by sidewall collapse, honey-combing caused by ground water infiltration and voids caused by arching of the concrete when the casing is removed.

## 5. ACKNOWLEDGMENTS

The author would like to thank the assistance of Stephen Crawford (Northern Gateway Alliance), Tim Sinclair (Tonkin & Taylor Ltd), and the alliance participants comprising Transit New Zealand, Leighton Contractors, Fulton Hogan, URS New Zealand, Tonkin & Taylor and Boffa Miskell.

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# PARALLEL BEDDED CLAY SEAMS IN THE AUCKLAND URBAN ENVIRONMENT

**Paul Fletcher**
*Foundation Engineering Consultants Limited*
*Auckland, New Zealand*

## ABSTRACT

Pre-existing deep-seated planar landslides located within the southern outskirts of the Auckland urban area present many geotechnical implications for the City's rapidly expanding housing developments. Auckland is home to approximately 35 percent of the population of New Zealand and is one of the fastest growing areas in the country.

In particular, parallel bedded clay seams present within the underlying Waitemata Group weak rock horizons are thought to be the cause of major historical translational and subsequent circular slope failures. These failures are evident when looking at the geomorphology of the affected areas in south Auckland and have major implications for slope stability within these areas today. Urban expansion is currently stalled along the boundary of this area due to the difficulties and costs involved with developing the land.

This paper will discuss investigation, potential remedial works and the outcomes of land development within this troublesome geological formation, referencing a number of land development sites contained within the southern city fringe.

## 1. INTRODUCTION

Parallel bedded clay seams (PBCS) occur within the Waitemata Group rock mass that is common to a large proportion of the Auckland urban environment as shown in Figure 1. The Waitemata Group is a sedimentary formation and has been dated back to the Miocene Epoch (20 million years ago). This tertiary group rock mass comprises predominantly flysch sequences (alternating horizons of sandstone and mudstone) and falls within the strength range of hard soils to weak rock.

Derived from deep marine sediments, deposition is thought to have come about by turbidity (density) currents, resulting in alternating or interbedded sandstone/ siltstone/ mudstone sequences. Total thicknesses up to 500 metres have been measured.

Insitu weathering of the usually dark-grey bedrock material has created in most locations, an overburden comprising mixtures of silts, clays and sands, being predominantly orange, brown and grey in colour and often containing hard dark-brown iron oxide (limonitic) concentrations, (NZGS, 1992 [1]).

This formation was subjected to tectonic forces during the late Otaian period which acted to induce folding and inclinations within the rock mass (Isaac et al 1994[2]) as explained further below. Further to this, four main faults influence the SLZ. These include the Whitford Fault, Brookby Fault, Drury Fault and the Alfriston Fault. Lineaments and smaller faulting within the SLZ are responsible for ridge and gully formations.

Tectonic actions are believed to have brought about flexural slip within the Waitemata Group bedrock. This action is similar to what occurs within any laminated body undergoing flexure. Horizons within the overall body of the rock move relative to the horizons above and below. The force expended as these planes rub past one another during flexure can cause alteration of the contact surfaces resulting in thin layers of highly plastic clay being formed (PBCS), these clay seams remain at residual soil strength and are thus prone to movement even in situations of favourable bedding (dip opposing slope angle).

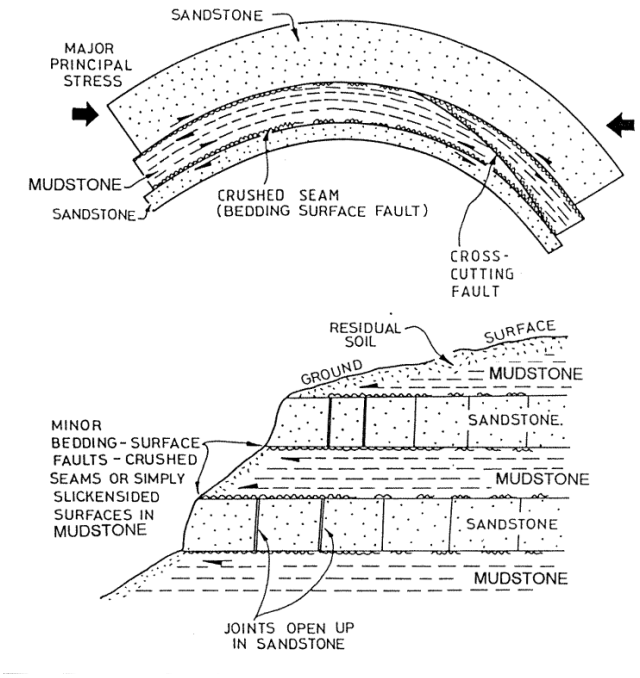


Figure 1

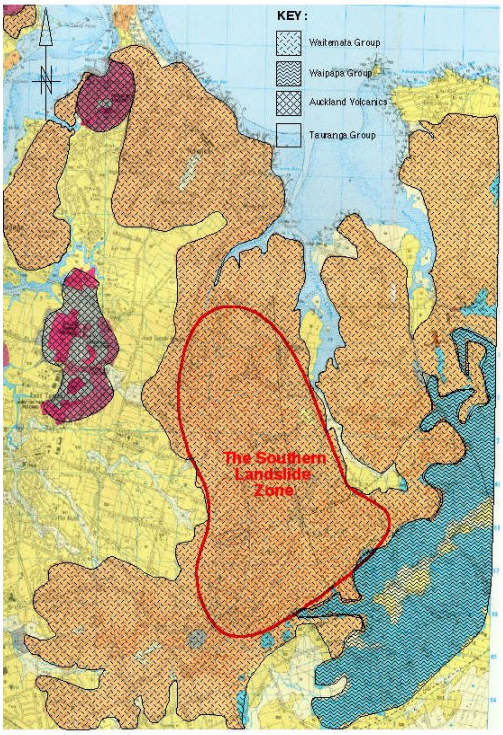


Figure 2

PBCS are formed within the bedrock by this process and in cases where the rock is subsequently tilted, these seams produce a failure mechanism that differs from that which is usually attributed to Waitemata Group materials. The majority of observed failures around the Auckland Region and within Waitemata Group materials occur within the weathered overburden soils, followed by (in order of decreasing incidence) failures along the interface between rock and soil, within the transitional zone between soils and rock and finally by failures within the rockmass. Typically slope failures are driven by saturation of the overburden or by the flow of water along the soil/ rock interface at depths of less than 10 metres. Being sited within the bedrock, PBCS are typically at depths of between 10 and 20 metres below existing ground levels and in some cases deeper.

## 2. SPATIAL DISTRIBUTION

Parallel bedded clay seams have been recognized as a major causative factor in the highly unstable topography evident within the SLZ, this 60 square kilometre zone extends east from East Tamaki Heights to Whitford. Many studies have been undertaken in this area, which has been termed “the southern landslide zone” (SLZ) (Chapman 1998 [3]). This area of Auckland has for the last 50 to 100 years been utilized predominantly as a pastoral zone for grazing cattle and sheep with bush covering the steeper gullies.

However, with the rapid growth of the Auckland region, pressures are now being exerted to allow the urban fringe to expand into this area.

The SLZ as shown in Figure 2 (Kermode 1992 [4]), can be broadly characterized as an area of Waitemata Group rocks and weathered soils which have been subjected to tectonic forces resulting in a wavy, undulating landscape. Lineaments including streams, valleys and ridges have formed parallel to, and also branching from faulting and contribute to the structural geology of the region.

Recent planning changes have lead to rezoning of large areas of southern Auckland. One of these areas includes the southern landslide zone and therefore a broad area of land affected by PBCS. In recent times Foundation Engineering Limited has been involved in a number of land development
projects in this area.

## 3. INVESTIGATION AND MODELLING

Geotechnical investigations within the SLZ require careful attention to detail to ensure that the geological model is correct and are typically undertaken in the following stages:

1) Site walkover: this is undertaken to get a feel for the scale of the site and site features as well as an initial impression of geomorphology
2) Desktop study: this involves researching previously investigated sites in close proximity to the subject site, viewing aerial photographs and drawing up a preliminary geomorphological map
3) Geomorphological mapping: this involves a site visit to accurately map the features observed in the aerial photographs and to add or refine those features that are visible on site
4) Field investigation planning: at this stage a geological model is postulated for the site based upon the previous steps. The field investigation is planned based on this model
5) Field investigation: a suite of machine boreholes, hand augers and trial pits is undertaken to ascertain the characteristics of the soils and underlying rock mass, samples are taken where appropriate.
6) Review: at this stage the field investigation to date is reviewed and checked against the postulated model, any variations lead to changes in the model and may lead to further investigation. The existing model is proved or disproved at this stage and a final model is produced based on the information gathered to date.
7) Testing: samples taken from the field investigation are tested to ascertain soil parameters for stability analysis
8) Reporting: once the model has been finalised it is run through slope stability analysis to determine the scope of works required to reach a factor of safety (FOS) of 1.5. Work necessary to reach this FOS is detailed and a report issued
9) Earthworks monitoring: during the earthworks all excavations are logged and findings compared to the model to ensure that site conditions are as assumed from the investigation, alterations made as needed.

The outcome of this system is a subdivision constructed with a FOS against slope instability of 1.5 under anticipated conditions.

## 4. GEOTECHNICAL PROPERTIES

Waitemata Group rocks within the wider Auckland area have typical parameters of:

| | |
|---|---|
| Natural Water Content | 17 – 88% |
| Sand Content | 2 – 46% |
| Silt Content | 31 – 54% |
| Clay (<2μm) Content | 10 – 58% |
| Porosity | 10 – 45% |
| Bulk Density | 1600 – 2400 kg/m$^3$ |
| Liquid Limit | 25 – 70% |
| Plastic Limit | 20 – 50% |
| Liquidity Index | 0.1 – 0.74 |
| Plasticity Index | 20 – 60 |
| Uniaxial Compressive Strength (UCS) | 0.5 – 10 MPa |
| Cohesion (c) | 0 – 50 kPa |
| Friction angle ($\phi$) | 17 - 64˚ |

Many failures within the Waitemata Group Rock mass are structurally related. Planar failures occur in natural slopes or in slope excavations where bedding or joint sets have an average dip direction similar to the slope face dip direction.

In order that sliding occurs on a single plane, the following geometrical conditions recognised by Hoek and Bray (1977[6]) must be satisfied:

1. the dip direction of the planar discontinuity and the dip direction of the slope face must be within $\pm$20 degrees;
2. the dip of the planar discontinuity must daylight from the slope face, therefore, must be less than the dip of the slope face;
3. the dip of the planar discontinuity must be greater than the angle of friction of the surface; and
4. the lateral extent of the potential failure mass must be defined either by lateral release surfaces that do not contribute to the stability of the mass, or by the presence of a convex slope shape intersected by the planar discontinuity.

Within the southern landslide zone the Waitemata group rocks are typically inclined due to faulting and folding and current landscapes tend to reflect the underlying structure of the rock. Ring shear and shear box testing undertaken on samples recovered from PBCS within the SLZ have retumed values of c'=0 to 3 and $\phi$'=10 to 18. Thus the identification of PBCS within the rock mass has a large impact on the stability analysis of the slope.

## 5. EFFECTS ON LAND DEVELOPMENT

To understand the effect of parallel bedded clay seams on land development the reader must also have an appreciation of the legislative framework surrounding land development within the areas affected. Land development within Manukau City is affected by Auckland Regional Council guidelines, national legislation and also council specific guidelines. The net affect of these various pieces of applicable legislation is that pre and post development landforms are effectively constrained to relative similarity due to the necessity to protect streams, flora and fauna in their current states. Bulk earthworks approaches generally including the cutting down of ridges and filling of gullies are no longer accepted by the various councils and thus less extensive stabilisation methods have evolved.

The effect of PBCS on instability mechanisms is to create translational slides. A translational slide may progress infinitely if the surface on which it rests is sufficiently inclined and as long as the shear resistance along this surface remains lower than the driving force (Varnes 1978[5]). This is in contrast to rotational slides where the rotary movement of the toe tends to restore equilibrium to the unstable mass and the driving momentum decreases during movement.

Any alteration of the natural slope forms must therefore be undertaken carefully following modelling of the slope to ensure that excavations or filling does not upset the equilibrium of these slopes and reactivate historical translational slides.

### 3.1 CURRENT LAND STABILISATION TECHNIQUES

Drainage is generally the first approach to land stabilisation in this area. Lowering ground water levels allows for a reduction in the weight of the slide prone blocks of land increasing overall factors of safety, however, these drains are often not able to be constructed to drain the deeper clay seams due to their depth. Drainage is time consuming and costly. To effectively drain the heavy overburden clays found in this area drains need to placed at close centres to relatively large depths, often of the order of 5-7 metres. Drainage also needs to be positioned so as not to impede on the development potential of the residential lots created. Maintenance of these drains is also of concern in a built up subdivision. Typically counterfort drains are utilised in these soils as shown in Figure 3.

Maintenance of these drains is difficult due to the lack of access, this is compounded once development of the land has been undertaken. Construction of these drains is therefore carefully undertaken under supervision to ensure that the prescribed methodology is followed.

Other drainage types such as bored drains with relief wells have been utilised where greater drainage depth was required. These drains require on going maintenance and are therefore unattractive to local councils. Therefore, drainage alone is not permitted by the various Auckland councils as a solution for stabilizing slopes.

Mechanical stabilization utilising methods such as soil nailing and soil anchoring are not effective protection against PBCS influenced slides as PBCS are commonly identified at depths between 10 and 20 metres below current ground levels, below the effective depth of these methods

Buttress fills are the most commonly used tool to stabilize PBCS. Buttress fills involve excavating a portion of the slope to expose the rock below the clay seam, the excavation is then well drained and backfilled with engineered fill as shown in Figure 4. This serves to buttress the toe of the failure surface(s) and interrupt the continuity of the clay seam(s). Buttress fills are sized based on computer slope stability analysis and sited to provide support to the

maximum number of potential lots. However, great care must be taken during construction not to activate movement in the slope by reducing toe support during excavation. This necessitates staged construction of the buttress fills in small sections and unfortunately this is expensive. Buttress fills can cost thousands of dollars per lineal metre to construct and as such are only economical in areas of high land value.

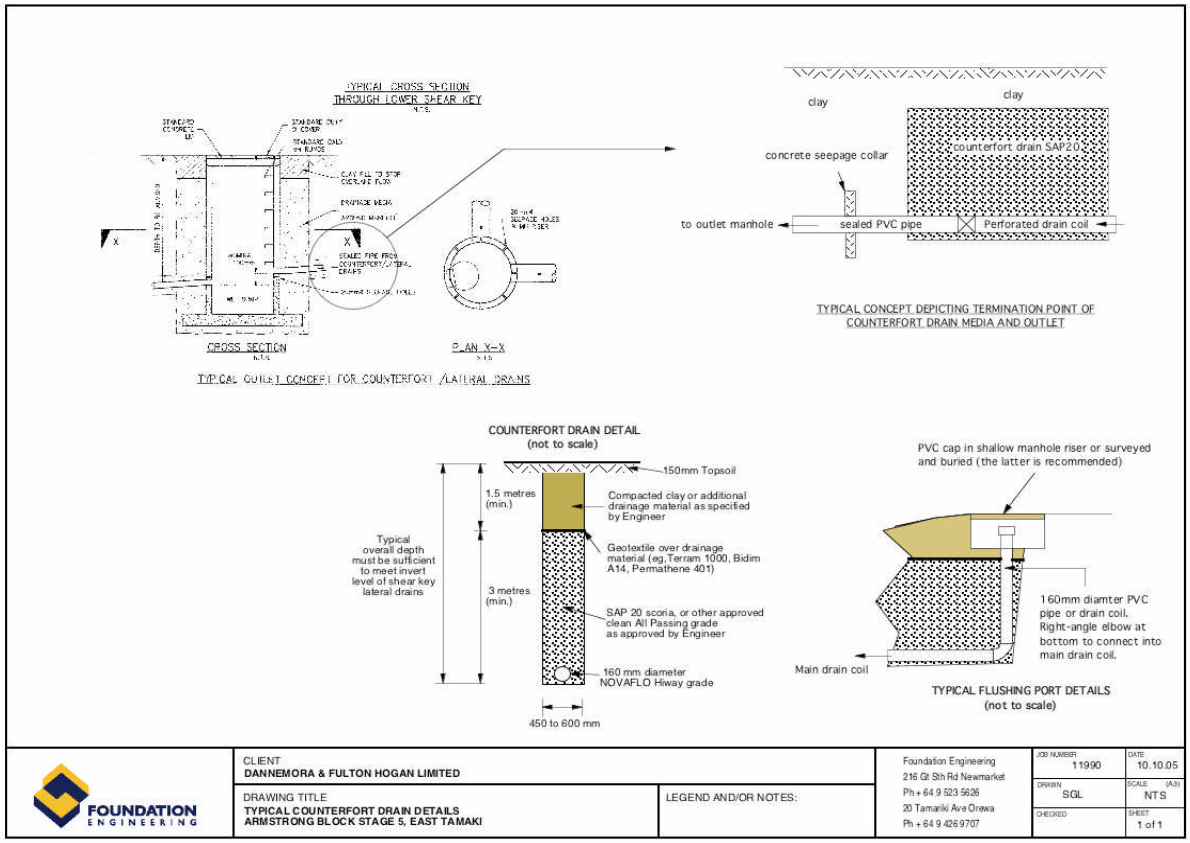


Figure 3

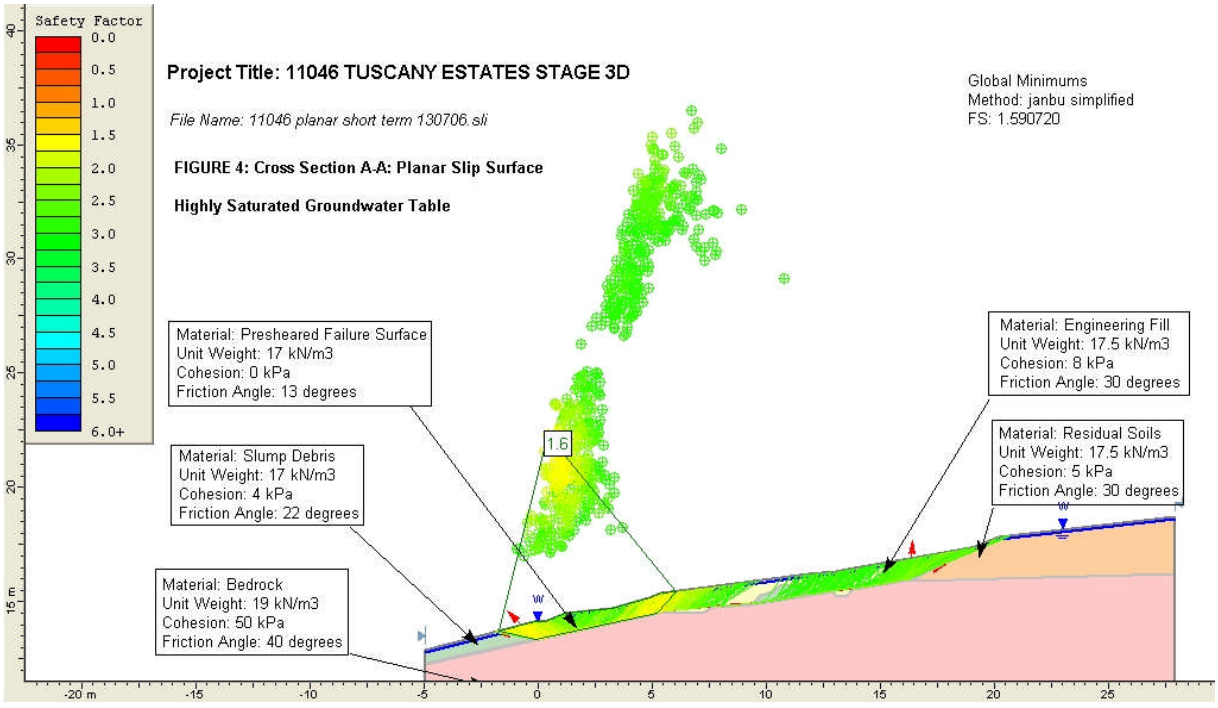


Figure 4

## 6. CONCLUSIONS

Solving the problems PBCS pose for residential development will open the way to expand Auckland further to the southeast through the southern landslide zone and although much research has been undertaken regarding this problem, we have yet to report any significant breakthroughs in the economical treatment of PBCS.

The problems of access associated with the depth of these features exponentially increases costs involved in stabilisation. As such the majority of the Southern Landslide Zone remains in pasture awaiting property prices to rise to offset the high costs of land development.

However, by undertaking detailed investigation and thorough design processes to ensure an accurate model, we can reduce the cost of stabilising this land by eliminating excessive conservativism in design. It is therefore imperative that the developers of any such land are well informed regarding the nature of the SLZ at an early stage. This understanding of the site and the costs involved will ensure that stable subdivisions can therefore be constructed within the SLZ at a reasonable cost.

One possibility for lower cost solutions in the future is deep soil improvement. This is undertaken by special machinery that drills columns into the soil and weak rock and stabilizes the soil with lime and/or cement among other compounds as it progresses. This effectively creates columns of improved strength soils, disrupting the continuity of the PBCS and stabilising the slope. However, the setup cost of such machinery and lack of applicable performance histories in the New Zealand environment has so far proved a stumbling block for this technology.

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# ASSESSMENT OF CUT SLOPE IN ROCK, STATE HIGHWAY 20, AUCKLAND, NEW ZEALAND

**Rhys Graafhuis**
*Maunsell Limited*
*Auckland, New Zealand*

## ABSTRACT

The new State Highway 20 motorway extension passes the basaltic volcano of Mount Roskill. Large cuts are to be excavated into the northern slopes of Mount Roskill, raising the risk of a wedge of basalt lava sliding on the underlying loose scoria. In order to assess the risk of sliding, the lava was mapped by drilling a series of percussion holes through the lava into the underlying scoria. The geology of the area is locally variable with complex relationships between the geological units. Investigation techniques are outlined, together with results of the mapping, construction risks and suitable stabilisation measures.

## 1. INTRODUCTION

Transit New Zealand is planning to extend State Highway 20 (SH20) westward from its termination at Hillsborough Road, to near Richardson Road in Mt Roskill, Auckland City. Associated with the extension are various works to connect to and upgrade existing roads. The proposed extension is approximately 5.3 km long Figure 1: shows the proposed alignment past Mount Roskill.

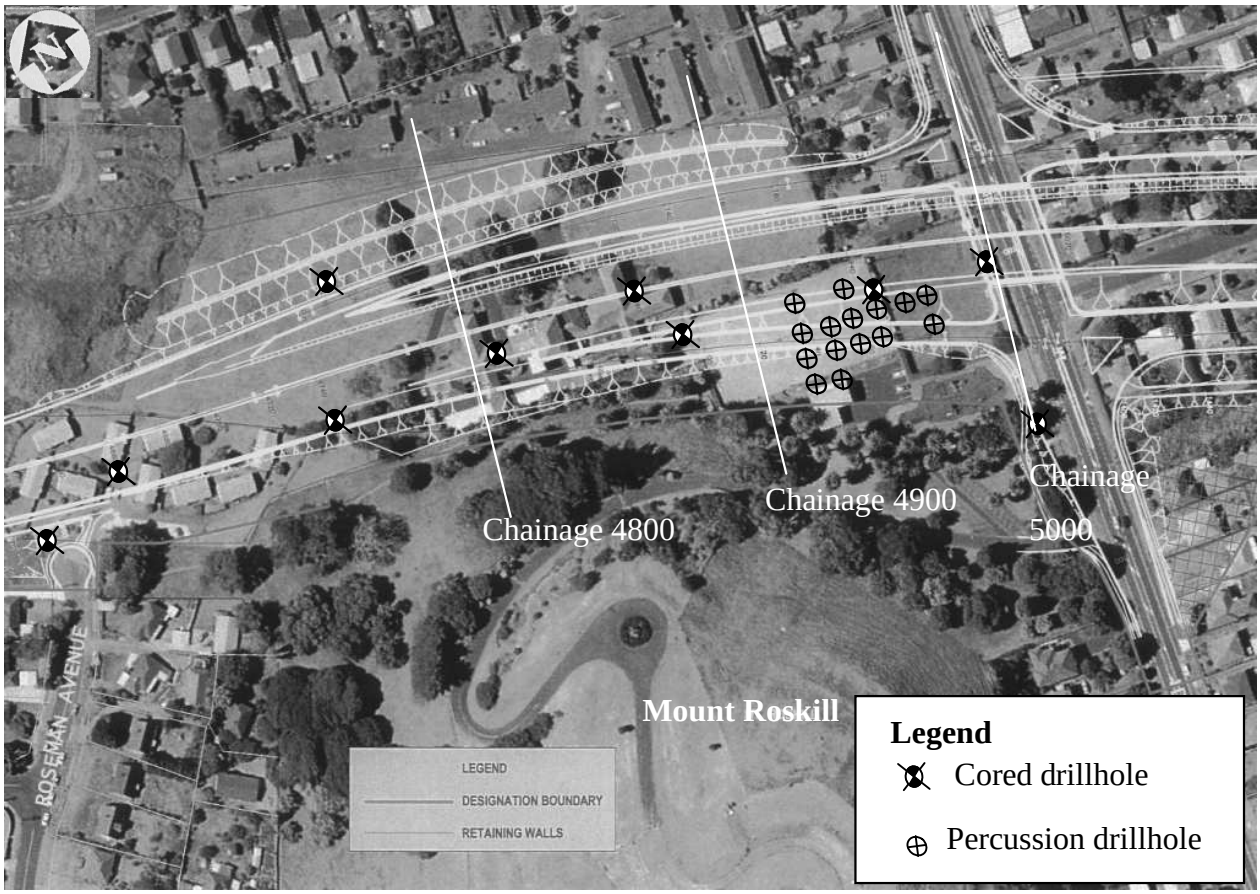


Figure 1: Aerial photo with location of SH20 alignment, drillholes and grid of percussion holes

Construction of SH20 involves the excavation of a 10.5m high cut face into the flanks of the basaltic volcano of Mount Roskill. The cut face will expose basalt lava, tuff and scoria. The stability of some of these volcanic materials in cut slopes is the subject of this paper.

## 2. LOCAL GEOLOGY

The new SH20 extension generally follows the Oakley Creek valley, which is cut into the basement sedimentary rocks of the Waitemata Group that outcrop throughout the Auckland region. Waitemata Group comprises weak bedded siltstone and sandstone deposited approximately 25 million years ago. The uplift of the Waitemata Group (2 - 7 million years ago) resulted in a long period of erosion resulting in series of ridges, terraces and valleys, including Oakley Valley (Burns *et al*, 2003). Mount Roskill comprises part of the Auckland Volcanic Field, which consists of numerous small basaltic volcanoes within an area of approximately 360km$^2$. These volcanoes have erupted approximately 3km$^3$ of material covering 100km$^2$. Most of the erupted material is olivine basalt comprising lava, scoria, ash, lapilli and non basaltic lithic tuff (Kermode, 1992). Around 30 thousand years ago Mount Albert, 3km northwest of Mount Roskill, erupted. The tuff ring, scoria cone and lava flow blocked Oakley Creek and resulted in the deposition of the Mount Albert Alluvium in the damned valley upstream of the volcano. Shortly afterwards Mount Roskill erupted, building a tuff ring, followed by a small scoria cone built from two main fountaining vents as evidenced by two shallow craters on the summit. Breach of the scoria cone resulted in lava flowing over the Mount Albert Alluvium both downstream to the Mount Albert volcano, and upstream as a lava lake, substantially filling the valley. This lava dam resulted in a second period of alluvial sedimentation upstream (Mount Roskill Alluvium). Following this other volcanoes within the Auckland Volcanic Field have been active and produced a thick cover (4 - 5m) of airfall tephra over the area (Burns *et al*, 2003). Local geological units relative to Mount Roskill are shown in Figure 2. A cross section down Oakley Valley showing the relationship of geological units is shown in Figure 3.

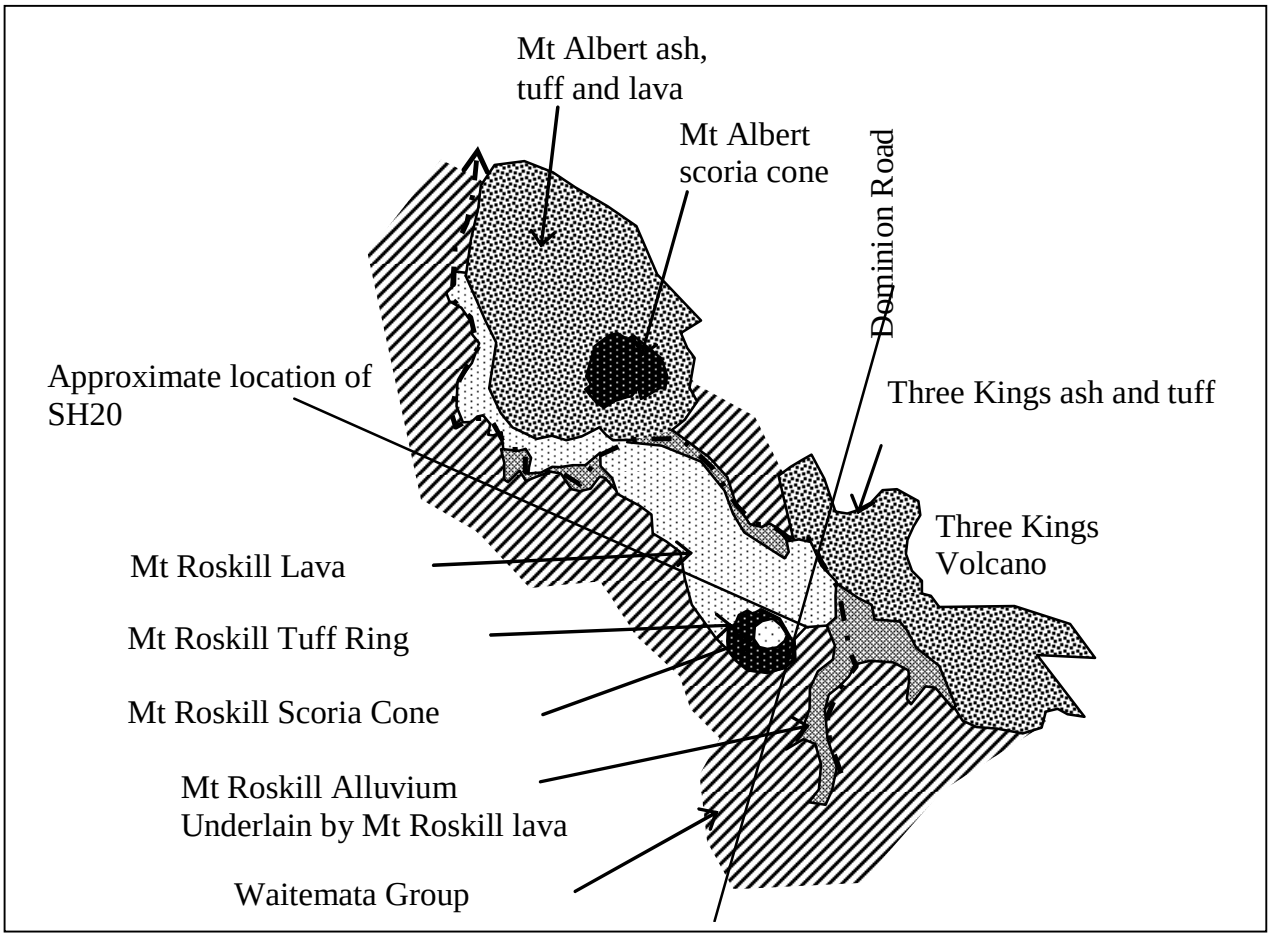


Figure 2: Local Geology map (adapted from Burns *et al*, 2003)

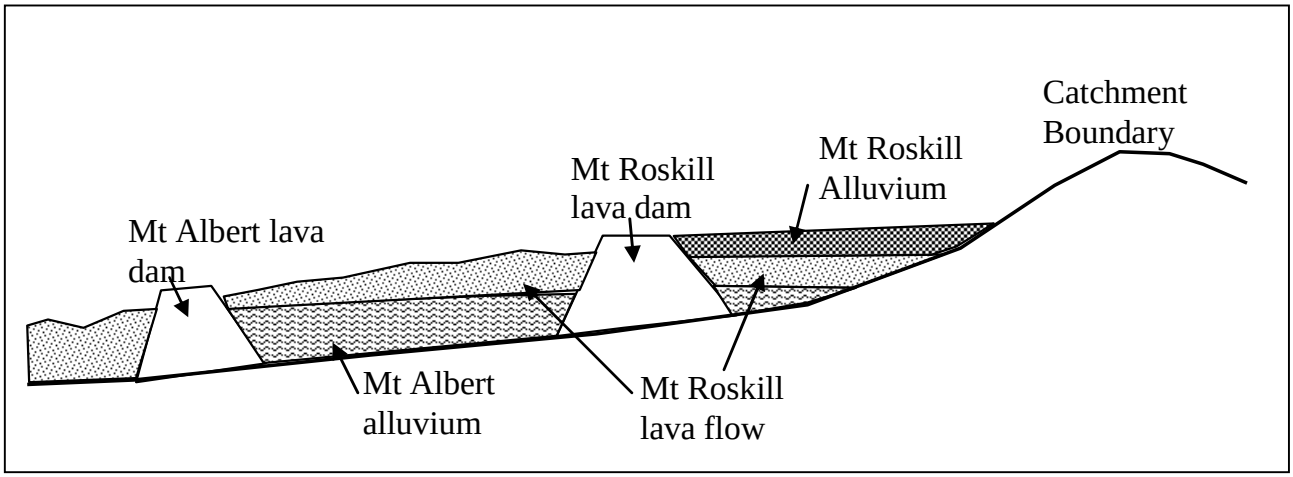


Figure 3: Cross section down Oakely Creek Valley (not to scale)

## 3. INVESTIGATIONS

Initial investigations comprising 11 cored drillholes highlighted the risk of a wedge of basalt lava overlying the Mount Roskill scoria cone sliding on the loose scoria after or during excavation of the cut slope (Figure 4). In order to assess the likelihood of this failure occurring, a series of percussion holes where drilled in a grid pattern to assess depth and extent of the basaltic scoria cone. The cored drillholes in the area provided a reasonable assessment of the local geology, while the percussion drillholes allow relatively efficient mapping of the scoria - lava surface.

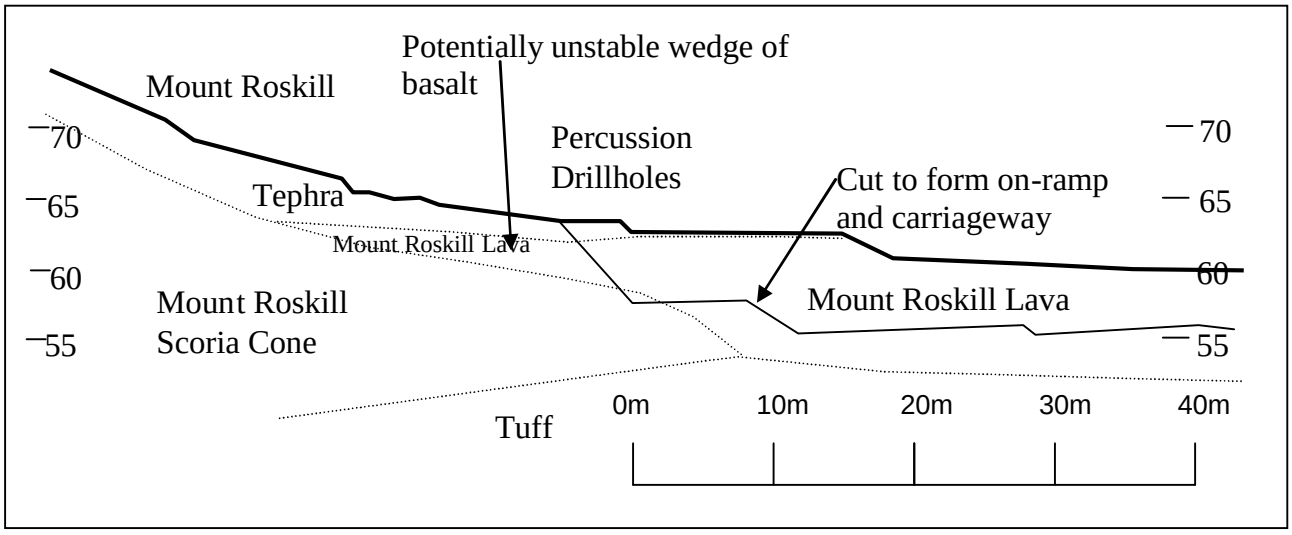


Figure 4: Cross Section of Mount Roskill and location of potentially unstable basalt wedge.

While percussion drilling does not allow accurate logging due to the lack of and disturbed nature of the cuttings returned, they do allow a large number of holes to be drilled quickly in rock, and if local geology is well understood, geological surfaces are easily identified by noting the depth of changes in drill rate, lack of or presence of returns to surface and loss of air circulation.

## 4. INTERPRETATION OF GEOLOGICAL UNITS FROM PERCUSSION DRILLING

Geological units were interpreted by comparing the percussion drilling with the logged geology from nearby cored drillholes. Outlined below are typical characteristics of the geological units during percussion drilling.

Typically the weathered brown ash at the surface was cohesive and drilled rapidly with consistent air and cutting returns to surface.

The basalt lava was characterised by slow drilling, although zones of faster drilling (drill-breaks) were common. The drill-breaks often coincided with a loss of air circulation and were interpreted as vesicular lava or partially welded scoraceous zones between thin lava flows marking the fluctuating / pulsating stages of an eruption. Slow drilling was interpreted as moderately strong to strong basalt of a thick flow.

Typically groundwater was encountered within the lava and coincided with short but rapid drill breaks. Often when groundwater was encountered a small 'geyser' formed at the surface, persisting for a minute or two before subsiding. This is interpreted as drilling through a groundwater filled vesicular or scoraceous zone or possibly a lava tube and the subsequent levelling of the head of groundwater.

The scoria at the base of the lava was noted by a sharp increase in the drill rate and a complete loss of air circulation. Occasionally the base of the lava flow was followed by variable drilling rates with no loss of air circulation. This was interpreted as the tuff ring comprising variable pyroclastic material, comprising a situation that only occurred towards the edges of the scoria cone.

In general based on the drilling 4m to 5m of brown ash and weathered rock overlies basalt lava. As expected, basalt lava thins up the slopes of Mount Roskill and thickens to the north. The basalt lava appears to be of better quality (slower to drill) towards the western side of the grid of percussion holes. The eastern side of the percussion holes were notable by variable drill rates and a lack of air returns to surface.

The results from the percussion holes indicate that the basalt lava / scoria interface in the area of percussion drilling is deeper than the carriageway and onramp design levels and should not be encountered in the final cut slopes in the grid area (Figure 5). Based on the results around 2 to 5m of basalt lava will outcrop low on the cut face and be overlain by between 3 to 5m of weathered ash and rock. While suitable stabilisation measures may need to be implemented upon exposure of the wedge, (such as rock anchors and netting), it is considered that the wedge will not present a significant problem and will be best remediated once encountered in the cut face.

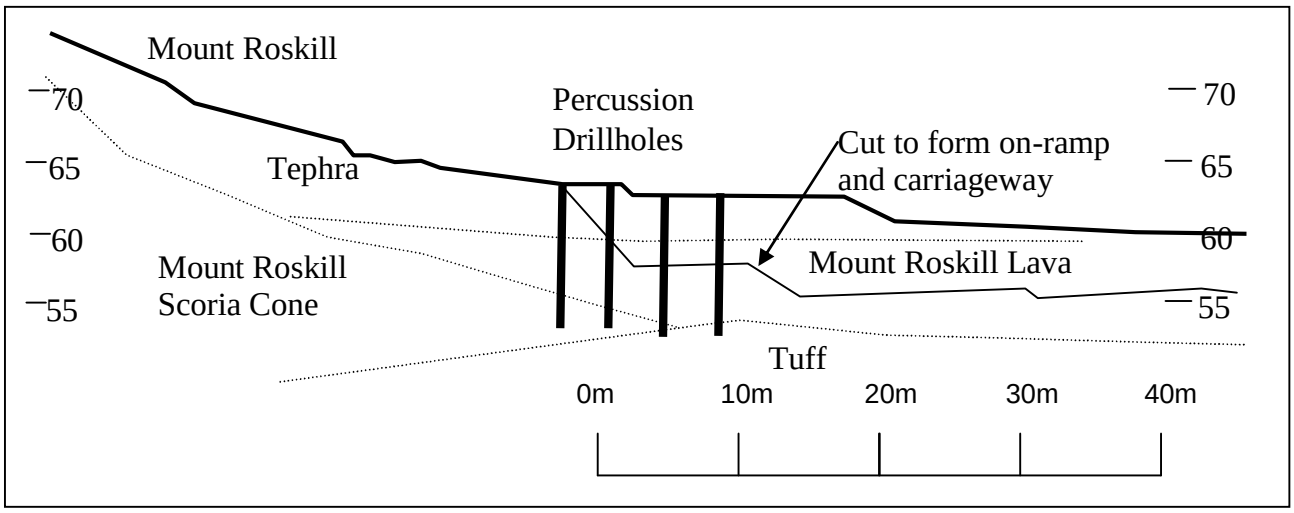


Figure 5: Cross section of Mount Roskill and location of percussion drillholes, showing the lava scoria interface deeper than the onramp design level.

## 5. INFLUENCE ON DESIGN

A number of factors where considered when allocating design parameters for the Mount Roskill cut slope, including stability of similar cuts slopes around the Auckland region, and laboratory test results.

The upper 4 to 5m of the cut slope will be in brown ash. Ash soils are prone to aggregation into silt and sand sized particle on wetting and drying. Exposed surfaces are therefore susceptible to erosion and it is important to protect slopes, for example with a vegetation cover. Ash soils in the Auckland region are typically excavated to long term stable slopes of between 1V:2H and 1V:3H (Meritec, 2004

Cut slope stability and treatment of basalt lava varies widely throughout Auckland, cuts range from fully faced and supported at less than 2m height to unsupported sub-vertical cuts in the order of 30m. Controls on the stability of cut slopes in basalt include the thickness of individual flows, degree of vesicularity and weathering of the rock, nature of materials between flows, orientation of interflow surfaces, joint patterns, particularly the development of columnar joints, nature of any joint infilling and slope height. Slopes cut in the thicker, subhorizontal flows that have well

developed columnar jointing, tend to be the more stable. Cuts in such cases can be higher and at steeper angles. Highly vesicular basalt tends to produce a poorer rock mass that has distinct zones of weak rock and rubble. Thin flows also tend to produce a poor, closely jointed rock mass with weak interflow zones of ash and rubble that can lead to toppling failures and block slides within individual flows (Meritec, 2004).

Scoria is considered to vary locally from loose to interlocked or partially welded within thin lava flows, affecting local stability. Stable cut slope angles in scoria typically range between 1V:1H and 2V:1H. Deposits may be poorly bedded to massive and bedding angles may be steep. The grain size of scoria exposed in the access road cuts on Mt Roskill typically varies from coarse sand to coarse gravel. The clasts are generally well interlocked and are difficult to disaggregate by hand. Exposure of cut slopes tends to result in the removal of fines and, long-term, face degradation by erosion. Local erosion and undercutting occurs where groundwater flows are concentrated (Meritec, 2004).

## 6. CUT SLOPE DESIGN RECOMMENDATIONS

Ground conditions for the Mount Roskill cut faces typically comprise ash overlying a combination of basalt, scoria, and tuff respectively. Recommended stable cut batters are 1V:3H in ash and 1V:1.5H in scoria and tuff. Cuts on Mount Roskill will be no steeper than 50 degrees (except in basalt) and typically will be retained with soil nails and faced with a lattice basket system to retain topsoil and allow vegetation of the slope. Cuts in thick basalt lava are expected to be stable at angles near vertical for cut heights up to 4m, although a 1.5m wide (minimum) 'fall zone' is recommended to minimise the risk of isolated rocks falling onto the carriageway. Cuts greater than 4m high will require a combination of protective cover and/or full support to ensure long-term stability. Poor quality vesicular or scoraceous basalt will require local support comprising bolting and netting (Meritec, 2004).

## 7. OBERSERVED GEOLOGY FROM INITIAL STAGES OF CUT

The first stage of the cut into the side of Mount Roskill (Photo 1) has allowed a comparison with the interpreted geology from the percussion holes.

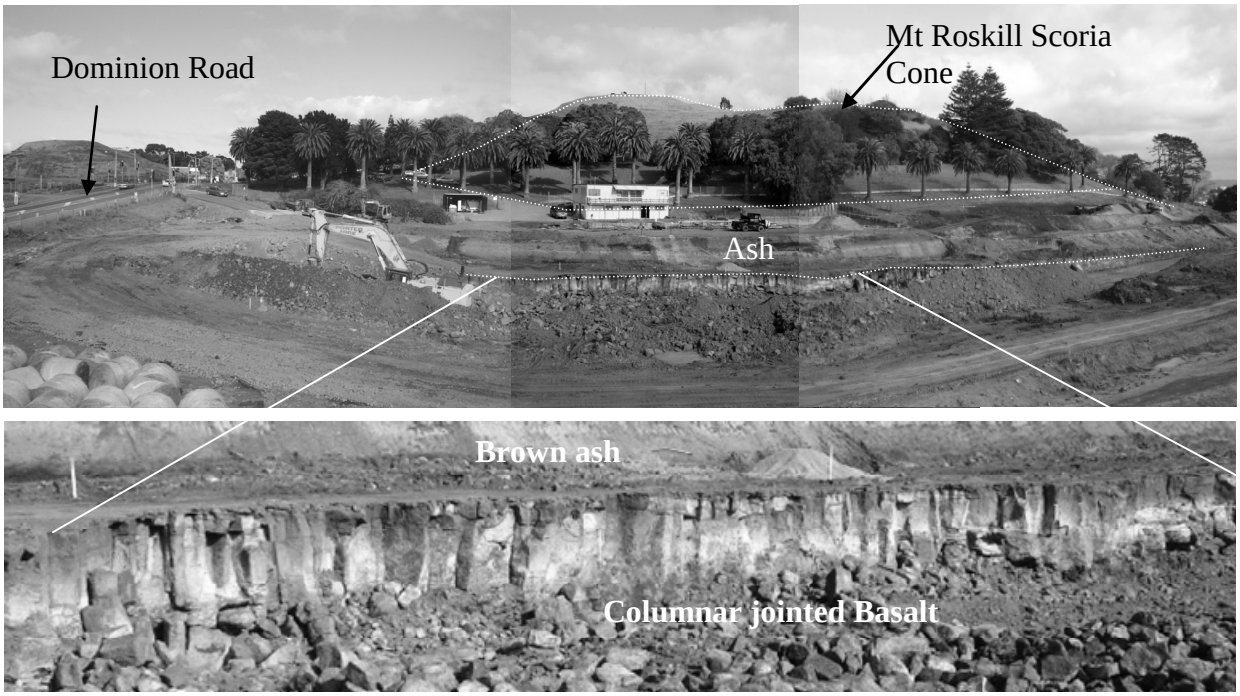


Photo 1: Mount Roskill cut slope exposing brown ash and columnar basalt

The excavated slope comprises cohesive brown ash, over columnar basalt, over thinly bedded irregularly fractured vesicular lava with thin beds of poorly welded to loose scoria. The columnar lava is weathered dark orange brown and is highly vesicular at the surface (top 10cm) and becomes less vesicular and weathered with depth. The joint surfaces in the lava are covered with clay veneer, probably washed by percolating rainwater. The large joints between the columnar basalt may have been cause by blasting of the rock during excavation.

The contact between the columnar lava and the underlying fractured lava and scoria is relatively steep (Photo 2), indicating a high potential for localised toppling of small blocks of jointed lava. This type of small localised failure along joint surfaces is considered more likely than large scale failure of lava wedge which requires a large intact rock

mass. Currently the cut is approximately 4 – 5m deep and it remains to be seen whether a substantial thickness of lava will be encountered or the scoria cone will be encountered.

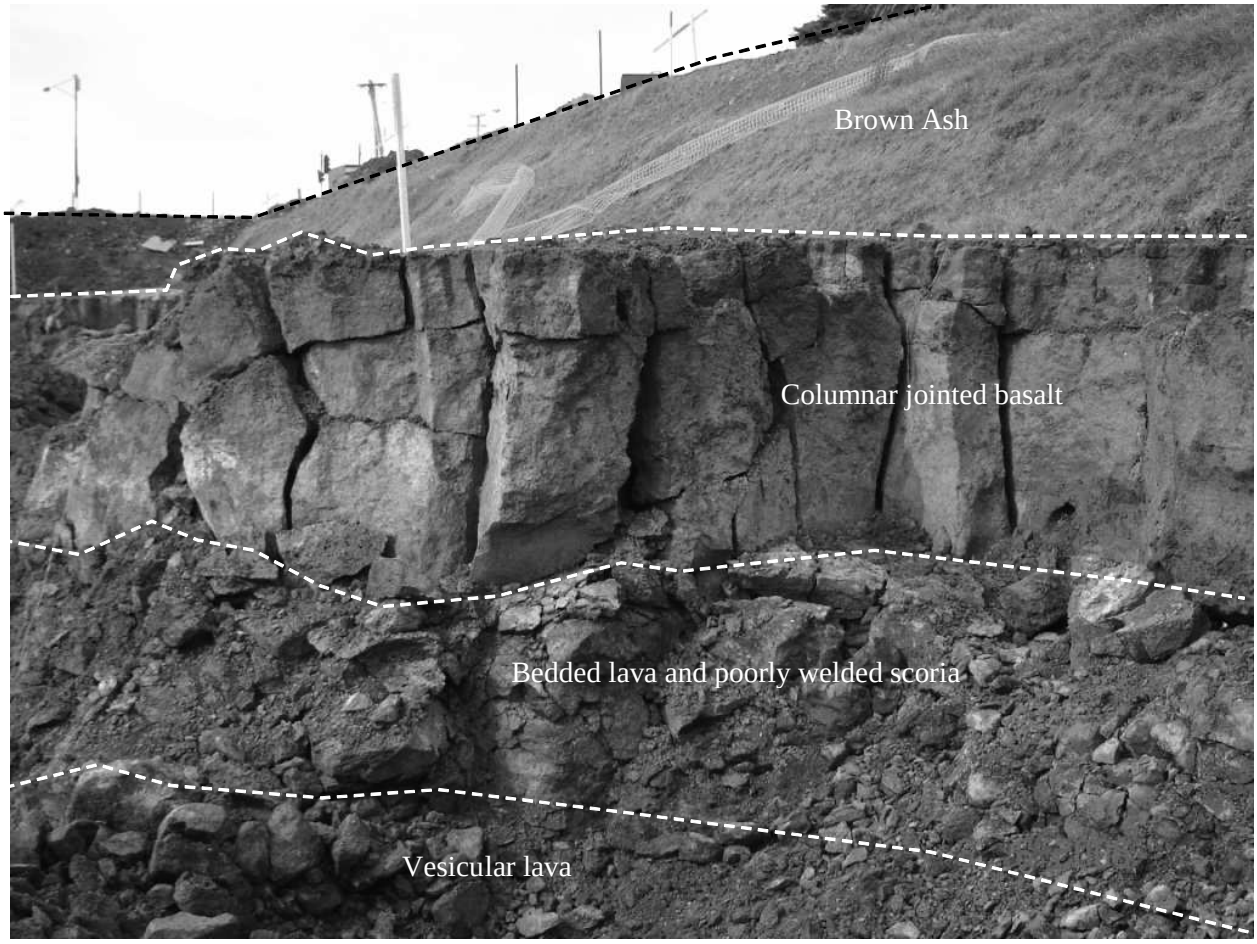


Photo 2: Columnar Basalt Lava and bedded fractured lava and scoria

## 8. CONCLUSIONS

Initial drillholes highlighted the risk of a wedge of lava failing in a cut face. A number of differing investigation techniques including cored drillholes, percussion drilling and field mapping has now concluded that there is a low risk of failure of the lava wedge because the lava / scoria interface within the area of percussion drilling is deeper than expected and there is a lack of a large intact rock mass. Small failures of jointed basalt on a steeply inclined surface are considered to be the primary risk of failure. This will need to be remediated with localized stabilization measures such as bolting and netting. Recent excavation of part of the cut slope has confirmed the findings from the percussion holes. Percussion drilling provides a fast and economical investigation method for mapping geological surfaces, so long as local geology is well understood.

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# MANAGEMENT OF A LARGE SCALE WEDGE FAILURE IN OPEN PIT MINE WITHOUT ECONOMIC LOSS

**Alastair Grubb**
*Ernest Henry Mining Pty Ltd*
*Cloncurry, Queensland*

## ABSTRACT

On the 17th December 2005 a 120m high, 550,000t failure occurred in the northwest corner of the Stage 6 pit at Ernest Henry Mine (EHM). On March 17th 2006 a second failure event occurred. Prior to both failures the area had been evacuated and demarcated. Neither failure resulted in a complete wall collapse, and at the time of writing this report the failed material was still moving. Mining to the wedge toe was able to continue without incident through careful management, intense 24hr prism monitoring and flexible mining practices. Currently, mining has progressed past the wedge and has achieved final design. This paper attempts to outline the development and management of the failure, and the challenges presented.

## 1. INTRODUCTION

Ernest Henry Mine (EHM) is a large open pit situated in 38km NE of Cloncurry, NW Queensland operated by Xstrata Copper. The mining method is stage cut-back mining utilising drill, blast, load, and haul mining sequence on 16m benches.

During October 2005 anomalies in pit wall prism monitoring results lead to the discovery of a large area of instability in the NW corner of the Stage 6 pit. A combination of monitoring and field work defined a large 120m high wedge forming on two structures. Primary monitoring was conducted using an automated prism monitoring system to adequately track changes and developing movement behaviour.

On December 17th 2005 the 550,000t wedge failed showing movement of up to 7m. The failure did not result in a complete wall collapse and slow movement continued after the failure. In the following months rain, blasting and mining at the toe resulted in increased movement. A second large movement event of several metres occurred on March 23rd 2006 following 177mm of rain over three days. Again no collapse resulted and minor movement (creep) is ongoing. Total movement to date has exceeded 16 metres.

The management of the failure by the EHM geotechnical team, working closely with Pells Sullivan Meynink Pty Ltd (PSM), allowed mining to continue in the area with only minor changes to the working pit design. Currently mining has successfully progressed below the failure and the final design has not changed, Figure 1.

## 2. GEOLOGY AND STRUCTURE

At the location of the failure, approximately 60m of Mesozoic and Tertiary sedimentary cover material unconformably overlies a slightly weathered, moderately altered mafic and intermediate volcanic Proterozoic rock mass, Figure 1.

The failure was bounded by two wedge forming structures and a tension crack. General descriptions of the structures are provided below. A stereographic projection of mapping data for the NW corner is shown in Figure 2.

### 2.1 FAULT 1

Fault 1 (F1) is a well defined, continuous and relatively planar fault structure in the west of the Stage 6 pit with an overall orientation of 40˚/060˚. A high level of confidence is held regarding the location and orientation of F1. On a pit scale the F1 is interpreted as an anastomosing system. Variations in dip and azimuth are seen and individual faults 'pinch out' and splay from others. On the scale of the failure it is reasonable to assume the structure is planar.

The infill is clay and rock gouge with thickness varying from 0.2 to 1 metre. Observation has shown movement occurred on the F1 footwall contact. Zero cohesion is considered reasonable.

The rock mass above the fault appears slightly weathered to moderately altered. Together with closely spaced joints and other cross cutting structures the rock mass could be described as Fair using the GSI classification (Marinos and Hoek, 2000).

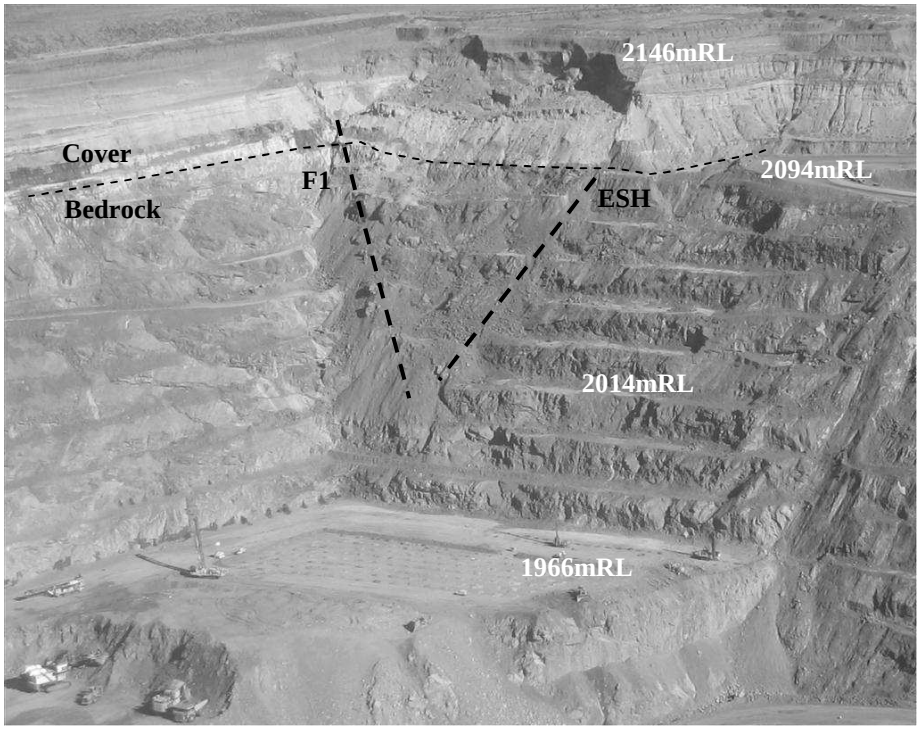


Figure 1: Photograph of the failure looking west from a helicopter.

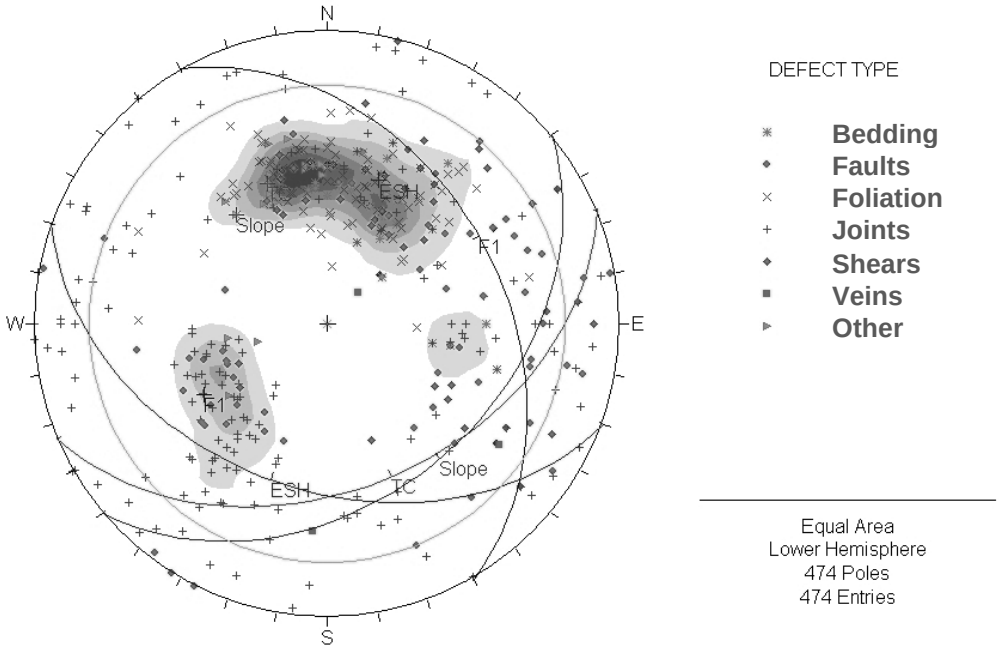


Figure 2: Stereographic projection of NW corner mapping data with wedge forming structures and 20° friction cone.

## 2.2 EASTERN SHEAR

The eastern boundary to the wedge is much less well defined than F1. Pit mapping located a potential structure and monitoring confirmed it to be the bounding structure. Although initially not considered a major shear, the structure (named Eastern Shear for the purposes of this paper) appears now to be associated with the hangingwall of a previously mapped shear package, the Komodo shear zone (Solid Geology, 2005).

The Eastern Shear (ESH) is an anastomosing ductile shear zone with a heavily foliated crushed rock fill intersecting the north wall of the Stage 6 pit. The orientation of the shear is variable and proved quite difficult to measure directly from pit exposures. Back analysis determined an orientation of 42˚/200˚ (see Section 4.3).

The ESH has been mapped on several benches and has a variable dip and azimuth. The overall shape of the structure is wavy however on a smaller scale is planar and the surface is smooth to polished. The rock mass around the shear is highly foliated and blocky and could be described as Poor using the GSI classification (Marinos and Hoek, 2000).

### 2.3 TENSION CRACK

The tension crack formed as a broad vertical arc in the sediments on the 2146mRL bench 8m below surface, Figure 1. Pit experience suggested the crack would project vertically downwards to the base of the cover sequence, Figure 3. Inspection of the wedge geometry indicated an additional structure was required to form the wedge. A review of the major structures in the area coupled with the structural pattern, Figure 2, indicated a structure called the Footwall Shear Splay (FWSS) or a combination of the FWSS and foliation formed the tension crack with an orientation of 43°/157°.

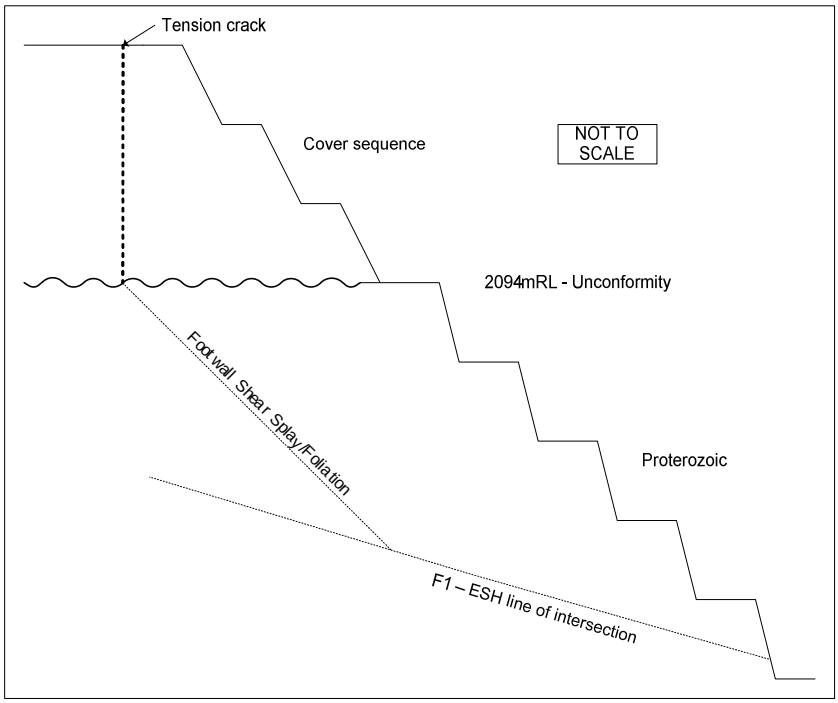


Figure 3: Schematic cross section through failure looking east.

## 3. MONITORING

### 3.1 HISTORY

The NW corner of Stage 6 has previously been identified as a potential stability concern. In a 2001 pit slope design study (PSM, 2001) the interaction between the Footwall Shear (FWS) and F1 was identified as a concern. The wedge formed was never undercut by the design wall and rock mass failure analyses showed it to be stable.

In late 2004 some work between EHM and PSM was completed to assess the potential for wedge failure on F1 and the foliation present in the footwall. The results showed the foliation was not undercut by the design pit and rock mass failure analysis indicated the wall would be stable (FoS – 1.21). This analysis was completed without the knowledge of the continuity and poor rock mass associated with the ESH (Komodo structures).

Most recently in October 2005 prism monitoring anomalies raised suspicion that the reference pillar in the NW corner, Reference 2, may be moving, Figure 4.

### 3.2 PRISM MONITORING

The movement was initially attributed to creep within the sediments as the pillar was located on an unconfined corner, Figure 4. To check this assumption, prisms were installed on the wall below Reference 2. Prism 1N169 was the first prism installed within the failure zone on October 28th, 2005. It was read from Pillar 1 in the south, situated almost perpendicular to the wedge. The prism provided data up until the December failure when it was destroyed by falling rocks, and will be the basis of discussions prior to the December failure. Prism 1N175, which was installed near 1N169 on November 18th, survived the failure and will be used to describe movement after the December failure.

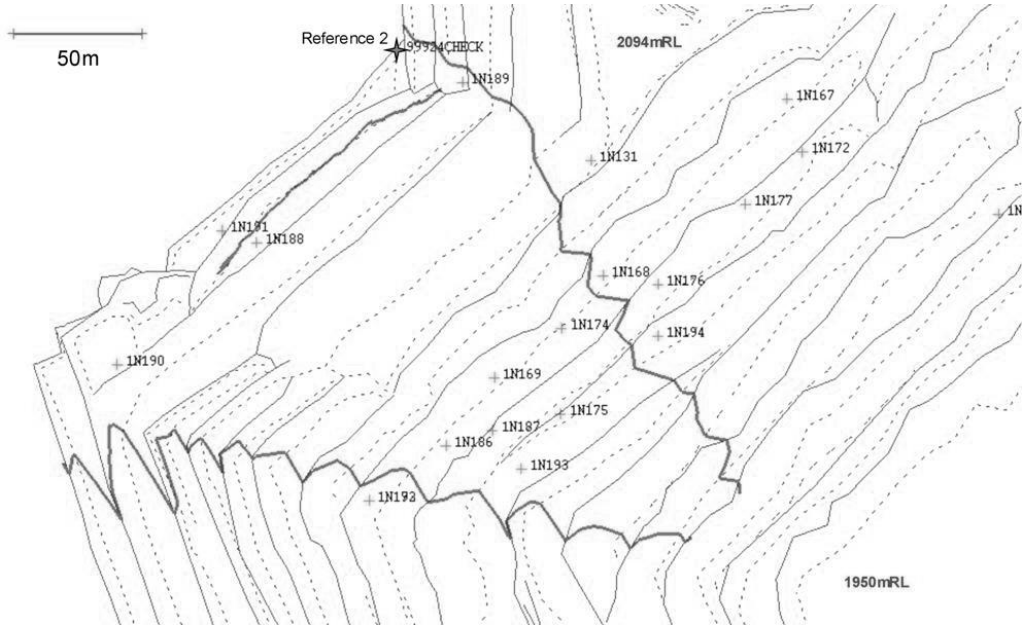


Figure 4: Plan of the NW corner showing prism locations and structures.

Movement on 1N169 occurred within days of installation and together with over 14 other prisms later installed in the area, provided evidence the movement was not only creep in the sediments but larger scale movement of the basement rock. The data can be separated into periods of pre and post initiation of failure, Figure 5 inset. Prior to the initiation of failure movement occurred as separate stick slip movement events triggered by blasting.

Post failure initiation data shows what could be described as a "textbook" failure curve. The displacement shows an exponential increase as the material gains momentum and progresses towards failure. The accelerating movement allowed failure predictions to be made and an exclusion area to be put in place. The data also provided an idea of the magnitude and type of movement possible in the material, and a curve with which to compare future movements.

Possibly the most important information gained was the time taken for failure to develop (approximately two weeks from initiation). Although not considered *'the time required'* for failure, it provided confidence that the monitoring system would pick up movement that had potential to progress to failure before personnel were at any increased risk.

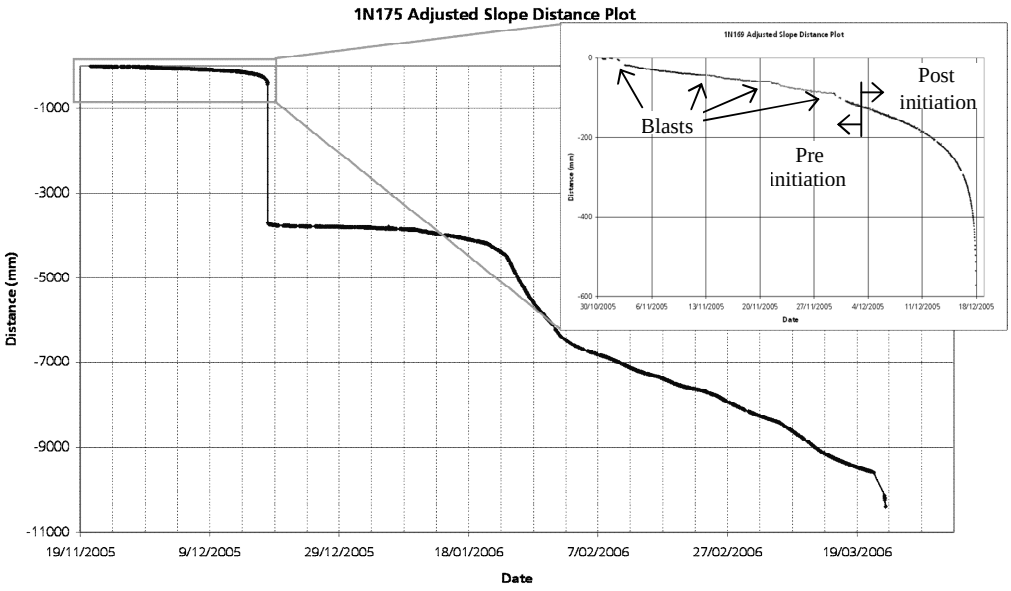


Figure 5: Adjusted distance plot for prism 1N175 with 1N169 inset

The movement was sensitive to external influences such as rain, blasting and mining at the toe of the wedge. Rain had a greater impact on wedge movement post December failure that either mining or blasting. From the first significant rain event (>25mm) for the year in mid January, until the time of the second failure in March, several spikes in velocity data

occurred. The March failure occurred following 177mm of rain in three days. From observation, rain events of larger than 25mm/day caused a significant increase in velocity.

Blasting commonly caused minor steps in movement before and after the December failure, and appears to have played a part in triggering the failure. Mining of blasted material directly in front of the toe of the wedge initiated some movement within the wedge. Mining in the general area (not directly at the toe) did not have any impact.

### 3.3 PRISM MONITORING SOFTWARE

The monitoring software used at EHM is a paired software package of Autoslope which runs the robotic monitoring instruments and provides data to a database, and Quikslope which reads from the database to display the monitoring data. This section discusses points regarding the use of both programs but will refer only to Autoslope.

Instantaneous velocity jumps or stick – slip movements are poorly represented in the velocity plots produced by Autoslope. It compares the latest reading to a reading at least 24 hours earlier, instead of comparing to the immediate reading before. This is to avoid creating large spikes in data caused by extrapolating monitoring errors characteristic between two readings over a 24 hour period.

As shown in Figure 6, the substitute for smooth velocity data is a 24 hour delay, and a loss, of true maximum rates of movement. Points 1 and 2 show how the true velocity (solid lines) and the Autoslope velocity (dashed lines) are calculated at the two points. The solid lines are tangents to the points of interest, while the dashed lines measure the overall change since at least 24hours previously (i.e. change from point 1 to point 2).

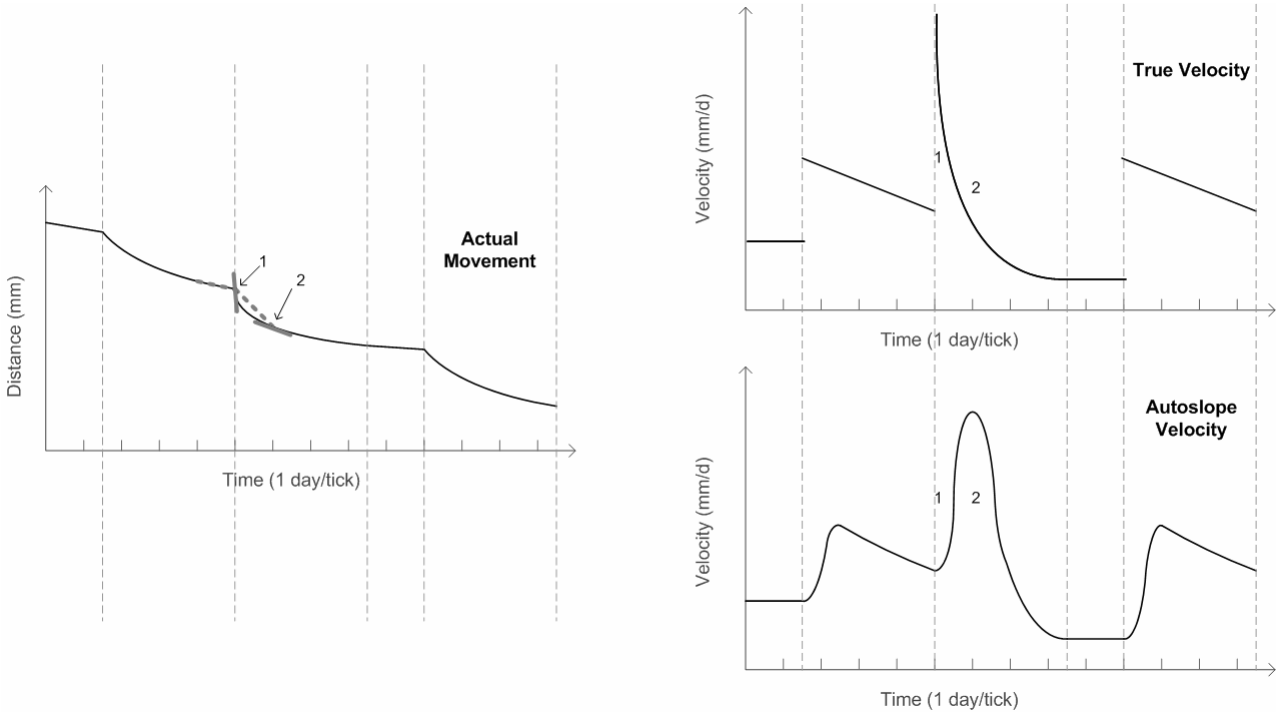


Figure 6: Sketch of true velocity and Autoslope velocity curves

While the velocity plots may not be suitable for determining the actual velocity of movement, they are useful when interpreting the sense of movement which is a helpful tool to convey to colleagues the status of the movement. It is also much easier to determine a change in rate of movement using the velocity plot than to attempt to judge the gradient changes in displacement curves.

Adjusted distance graphs must be viewed with caution and should be used in conjunction with unadjusted distance plots. Relying totally on the adjusted readings can mask movement or create false movement.

Distance measurements are more repeatable than angular measurements. This results in greater error in location plots, which use the angular measurements, than in distance plots. Scatter plots (2D prism location) and xyz movements (3D prism location) rely on angular measurements and can be very difficult to interpret unless large movement has occurred. For smaller movements, the angular error in the monitoring instrument produces extremely variable, nonsensical data. Such graphs can however be useful after significant movement has occurred.

The vector data produced by Autoslope is difficult to use due to the angular error. Movement vectors are recorded between each reading producing large variations. There is no facility within the software to use average changes in co-ordinates over time to determine movement vectors. Due to the large magnitude of movement, average movement vector calculations were able to be done using Excel and could be plotted in a 3D graphing package.

### 3.4 SLOPE STABILITY RADAR DATA

The Ground Probe Slope Stability Radar (SSR) was bought to EHM in late November 2005. The primary purpose of the visit was to ensure that the mapping and prism placement had been sufficient to define the wedge extents accurately and secondly, to act as a check on the prism data to obtain some confirmation and confidence in the data. The SSR returned results of the same magnitude as the prism system. The general agreement between the monitoring systems provided confidence to go forward using the prism system as the primary monitoring tool.

## 4. FAILURE

Following the early prism movement, the first visual sign of the instability was a large tension crack found on November 23rd on the 2146mRL bench above the moving prisms. The wedge failure model had been inferred from mapping and visual observations and confirmed and refined with monitoring data.

### 4.1 FAILURE MODEL

The failure occurred as a shallow plunging complex wedge formed on structures F1 to the west, ESH to the east, and a tension crack sub parallel to the slope, with a vertical projection through the cover sequence, Figure 3.

F1 and ESH form a very long wedge with an intersection line that plunges at approximately 17°, that was unlikely to fail without substantial water pressure. Back analysis showed that to achieve failure, water pressure was required to push the wedge and a tension crack sloping towards the toe had to exist. The tension crack had to cut off the long wedge and provide a face on which the water pressure could act upon to apply a force to oppose the weight of the wedge.

Seepage from structures and drain holes in the west wall near the failure confirm the presence of water. The recharge for the system is from a permeable sand layer in the cover sequence entering the tension crack at the sediment – Proterozoic unconformity.

Several smaller faults parallel to F1, foliation and foliation shears and cross cutting joints create a complex wedge which helps explain the large magnitude of the movement observed without complete collapse.

### 4.2 PRISM MONITORING VECTORS

Analysing the vectors of prism movement provided good control over the direction of the wedge movement, and consequently the orientation of the controlling structures. The seven prisms installed within the limits of the wedge, Figure 4, were reviewed in terms of overall three dimensional movements. Table 1 shows the prism movement vectors separated into periods of similar velocity.

Table 1: Movement vectors of prisms installed in wedge

| Date | | 1N169[1] | | 1N174[2] | | 1N175[3] | | 1N186[4] | | 1N187[5] | | 1N188[6] | | 1N193[7] | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| From | To | Dip | Dip Az | Dip | Dip Az | Dip | Dip Az | Dip | Dip Az | Dip | Dip Az | Dip | Dip Az | Dip | Dip Az |
| Installation | 17/12/2005 | -16 | 127 | -15 | 153 | -17 | 123 | -12 | 127 | -14 | 127 | -69 | 132 | | |
| 17/12/2005 | 18/12/2005 | | | -20 | 123 | -15 | 123 | -15 | 123 | -15 | 123 | | | | |
| 18/12/2005 | 24/01/2006 | | | -23 | 128 | -22 | 127 | -7 | 136 | -14 | 125 | | | -12 | 129 |
| 24/01/2006 | 5/02/2006 | | | | | -21 | 124 | | | -14 | 124 | | | -12 | 128 |
| 5/02/2006 | 21/03/2006 | | | | | -21 | 125 | | | -14 | 123 | | | -14 | 129 |
| 21/03/2006 | 23/03/2006 | | | | | -26 | 122 | | | -16 | 120 | | | -16 | 128 |

1 Installed 30/10/05. lost 17/12/05 3 Installed 20/11/05. lost 23/03/06 5 Installed 22/11/05. lost 23/03/06 7 Installed 19/12/05. lost 23/12/05
2 Installed 17/11/05. lost 20/01/06 4 Installed 22/11/05. lost 5/01/06 6 Installed 24/11/05. lost 17/12/05

The average movement across the prisms was approximately 16°/127° with the only excessive outlier, prism 1N188, situated in the slumping clay. The prism vectors confirm the failure model as sliding on the plane of intersection of the two structures.

### 4.3 DETERMINING STRUCTURAL ORIENTATIONS

As mentioned, the orientation and location of F1 are well known. The exact orientation and location of the ESH and the tension crack were less well known and were initially estimated from pit mapping and visual observation, and refined using monitoring data and back analysis.

To achieve a reasonable orientation for the ESH, the F1 orientation and the prism movement vectors were used. As the F1 – ESH line of intersection was found from prism movement (16°/127°) and F1s orientation was known, the orientation of ESH was constrained. The apparent strike of ESH on the slope face (~165°) was used to finalise the orientation.

The tension crack is assumed to be an extension of the FWSS with the same orientation. No drilling exists behind the NW corner to confirm the existence or orientation of the structure and only pit exposures over a hundred metres to the east provide the orientation data. It is noted that there is foliation with a similar orientation to the FWSS, Figure 2. A stepped failure linking foliation planes could also form (or assist in forming) the tension crack.

Back analysis using RocScience's package SWEDGE was used to determine the tension crack and water pressure requirements to achieve failure. The results indicate that with the FWSS acting as a tension crack dipping at 43°with a strike similar to that of the tension crack in the clay, a water pressure of 30% of saturated conditions was required to initiate failure.

# 5. MANAGEMENT

The primary tool used to manage safety and mining around the failure was the automated prism monitoring system. Extensive data was collected since suspicion of instability in November, 2005. Prism data has proven accurate and very precise providing adequate true displacement data and high quality relative data.

The overriding principal controlling work within the failure area was that monitoring must show the movement is either constant or decelerating. The basis behind this decision was that before a catastrophic collapse could occur, a period of acceleration and failure development is required. The potential for outside influences such as rain and blasting to shorten or bypass the acceleration phase of the failure development was recognised, and exclusion periods associated with such events were put in place. The exclusion periods usually related to the time taken to obtain sufficient prism readings to confirm decelerating or constant movement.

## 5.1 PRIOR TO DECEMBER FAILURE

When the instability was first noticed, monitoring data was insufficient to understand and manage the failure. The installation of more prisms and increased cycle frequency enabled a continually developing understanding of the movement and provided adequate data to manage the failure.

During the stick slip movement period before the December failure, a conservative view as to the possibility of failure was taken as the movement was initially difficult to understand and explain in a hard rock mine environment. Exclusion zones and exclusion periods at the base of the failure were used to manage movement.

The blast in the NW of the 2030mRL bench located at the toe of the wedge assisted in the initiation of failure. A 100 metre exclusion zone was instated one week before the failure removing personnel from the area. The failure occurred at approximately 11pm on 17th of December 2005.

## 5.2 MINING POST DECEMBER

Mining the blasted material in front of the failure began on the 19th of December under careful controls. A meeting between members of management, mining production, mine technical services and other relevant personnel was conducted to complete a Job Safety Analysis (JSA) to assess hazards, associated risks and ensure adequate control measures were in place to safely begin mining. The main control was constant, manned prism monitoring while personnel were working in the area. Fifteen prisms were included in the monitoring cycle providing new data approximately every 20 – 30 minutes, 24hours a day. Other controls related to mining procedures are mentioned below.

Mining was initially restricted to a Liebherr 996 hydraulic shovel. A later revision of the JSA based on new observations and monitoring data utilised the rope shovel because it moved material faster than the hydraulic shovel, reducing exposure time, and provided more protection from bench to sub-bench scale rock falls, which were seen as more likely than total wall failure.

A flexible mining schedule was a necessary control to reduce the risk of production delays due to mining under the failure. It was recognised and conveyed to mine planning that mining activity directly at the toe of the wedge would be undercutting the face and could trigger an increase in movement requiring personnel and equipment to be removed from the area. A backup dig face was always planned (and used on several occasions) to avoid standing a machine down.

In the event of rain, personnel and equipment were withdrawn from under the wedge. Doing so caused minimal production disruption as, at EHM, moderate rain stops truck haulage due the increased risk of tyre damage, and safety issues driving on wet clay sheeting.

A spotter was used for all activities within the failure area. The spotter was primarily in place to watch for smaller material falling from the face and the wedge boundaries.

### 5.3 PERSONNEL MANAGEMENT

Presentations were made to all mining and maintenance crews explaining what was believed to be occurring and displaying examples of the monitoring data. The main aim of the presentations was to convey the success of monitoring to date, and the confidence in the monitoring going forward. Daily updates were provided to mining personnel.

Training mining personnel to assist with monitoring of the prism system not only helped provide 24 hour coverage, but helped reduce 'black box' perceptions within the crews. By providing some personnel the tools to make decisions based on monitoring, it promoted the idea that decisions were justified by data.

## 6. DESIGN IMPLICATIONS

At the time of writing this paper, mining has been completed to the 1950mRL bench, three benches below the toe of the wedge. On this bench and the bench above (1966mRL), mining has been completed to final pit limits. Minor changes to the working pit design on some upper levels were required, such as stepping off design to allow bund walls and extra catch capacity in the event of smaller rock falls. Such minor changes were easily accommodated by the final stage design due to the low overall wall angles and wide berms of the west and north walls. The north wall has an Inter-Ramp Slope Angle of 40˚ with 17m catch berms so 5m step outs did not affect subsequent benches.

Catch capacity below the wedge increases substantially as the pit progresses. A life of mine ramp is planned under the failure area however by this time the north wall steps out some 80-90m, and the west wall steps across under the wedge toe. Because of this substantial catch capacity, the wedge is considered to pose a low risk to future mining activities below.

## 7. CONCLUSIONS

An adequate prism monitoring system can provide data to enable a pit to safely manage large scale wall movement. Adequate monitoring entails good prism coverage relative to pit structures and rock mass conditions, understanding of the monitoring system including monitoring errors and limitations and interpretation using all available monitoring data and pit experience.

Movement in the wedge was very sensitive to rain events and water pressure. This movement was directly related to the geometry of the tension crack. Water pressure enabled failure to occur on a 17 degree plunge angle.

The key to managing the failure was adequate monitoring, communication and education of mine personnel and a flexible mine scheduling.

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# PERMEATION GROUTING IN BOTANY SAND, CASE STUDY

**Claire Howell**
*Parsons Brinckerhoff*

## ABSTRACT

One of Sydney's main high voltage underground electricity cables was left damaged after consolidation settlement in Botany Sand occurred due to construction activities and dewatering of an adjacent development.

Continued settlement of the fill and Botany Sands posed a significant threat to reliable performance of the cable, as did potential heave to the cable during rectification works. Ground treatment using permeation grouting was adopted to remediate the affected area. Cement-base grouts were trialled with cone penetration tests carried out before and after, while ground levels were monitored continuously.

This paper describes the trial methods, results, the adopted stabilisation model, foundation improvement measures and challenges which arose during the ground treatment trials and rectification works. A number of restrictions were imposed on the works due to geotechnical, environmental and safety factors on and around the site, the most significant being proximity to high voltage cable and limited time envelopes for critical work. Lessons learned in this case may prove useful in similar situations.

## 1. INTRODUCTION

To the north-west of Botany Bay, Sydney, underground services in this area have been laid in Botany Sands, a unit typified by unconsolidated sand and alluvium. Acting as an aquifer, water is believed to flow through this highly permeable sediment towards Botany Bay. The ground surface in this area stands only 2 to 3 metres above sea level with groundwater levels observed to rise and fall with the tides.

## 2. HIGH VOLTAGE CABLE

TransGrid has two high voltage electricity cable circuits installed underground in the Greater Sydney area. The 330kV cable at the site, installed in 1976, provides 40% of Sydney CBD electricity supply. The cable consists of 1600mm$^2$ stranded copper core with a flexible corrugated aluminium sheath, is oil filled, wrapped with a constant oil soaked paper to act as insulant with a sealed outer sheath and encased in a polyethylene/nylon cover. The flexibility of the cable is limited by the potential for creases to form in the paper insulating layer, affecting the insulating properties. Strategically placed co-located optical fibres have recently been installed to monitor temperature along the cable. The cable lies approximately 900mm below ground level in a cable trench backfilled with a 14 to 1 sand/cement stabilized fill and topped with a concrete slab.

## 3. SITE AND GROUND CONDITIONS

The work site was located in a suburban street along a straight narrow road with old brick warehousing on one side, new multistorey residential buildings on the other. A new development was under construction at one end. The bitumen surface of the road was undulating and had been repaired on many occasions. A new sewer pipe had recently been laid down the centre of the road, parallel to the high voltage cable, at a depth of 7.5m. Due to the depth of the new service trench below groundwater levels the construction process had required dewatering. The bitumen seal over the recent work adjacent to the sewer manhole, shown on Figure 4, was observed to have cracked and settled and had been resealed twice in at least two months.

An area of subsidence, 8m in length and 4m wide, thought to be caused by inappropriate dewatering and construction techniques during the installation of the sewer main, destabilized an 8m length of 330kV cable, causing it to sag, with a maximum vertical displacement of 600mm. TransGrid was concerned that further settlement could cause damage to the high voltage cable and commissioned Parsons Brinckerhoff (PB) to undertake geotechnical investigations to assess ground conditions and provide recommendations for remedial works.

Plate 1 : Subsidence of cables shown after initial excavation

Material was carefully excavated from around the cable in the subsidence area to assess the damage (Plate 1). The cable was also supported from cross beams to prevent further movement while settlement plates were installed beneath the cable to monitor subsidence and/or heave. Either side of the cable trench the excavated material was found to be brick and rubble fill overlying sand. Groundwater filled the base of the trench at high tide.

## 4. INVESTIGATION METHODOLOGY

Existing borehole and historical information suggested shallow bedrock in the area. Dynamic Cone Penetration Tests (DCPTs) were carried out within the vicinity of the cables to gauge the level of compaction beneath the excavated surface. The results of these DCPTs revealed that the fill beneath the pavement surrounding the area was rubble/gravel, over loose silty sand with some possible voids. A number of test pits were excavated to observe the fill material and take bulk samples of the silty sand unit below.

These preliminary results, together with tight timeframe, led the team to adopt a remediation option involving ground treatment as being the most cost effective method. A number of Electric Friction Cone Penetration Tests (CPTs) were conducted around the area in order to assess the extent of ground requiring treatment and develop a better understanding of the sediment. The results of four CPTs provided a representative cross section of the destabilized area (Figure 1) revealing a palaeochannel, deeper bedrock than initially suggested and confirmed that the sediment requiring treatment was a sandy alluvium, though some clay lenses were inferred.

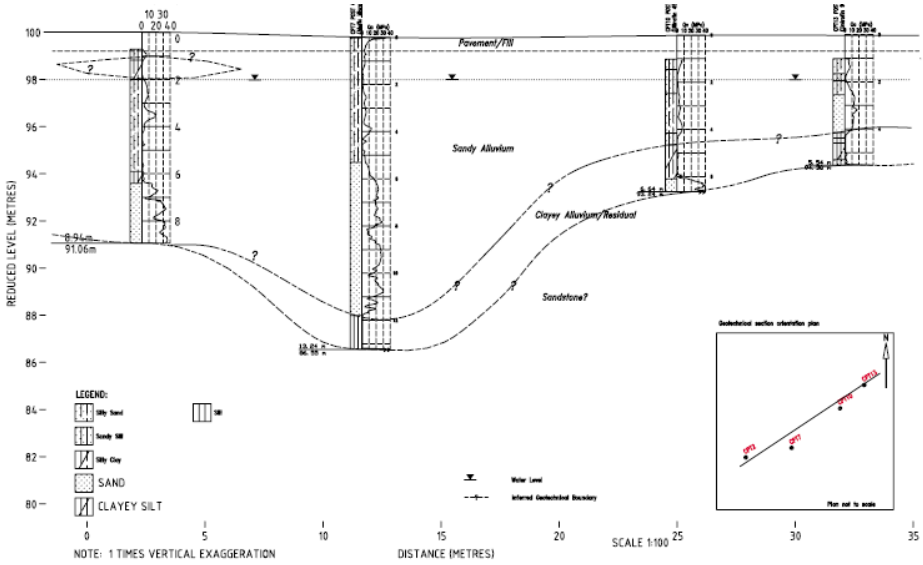


Figure 1: Cross section from CPT information.

## 5. PERMEATION GROUTING

Permeation grouting is the filling of voids in a soil or rock mass with grout fluid at a low injection pressure to strengthen and/or reduce permeability, while not destroying the original structure of the soil or rock (Bruce, 2005). The method uses 20mm diameter spears inserted into the soft sediment while flushed with water. As the spears are removed grout is pumped, at a constant pressure less than 100kPa, at a predetermined rate (litres/300mm), from a mixer, through a hose and down the centre of the spear into the sediment. Spacing of spear holes is typically set out as a grid of approximately 0.5m to 3m, with primary and secondary holes.

## 6. STABILISATION MODEL

Permeation grouting of a micro/ultra-fine grain size cement grout was assessed as the most suitable stabiliser and method for the site due to the following 4 factors; (1) the loose nature of the sediment, (2) the majority of the sediment volume to be stabilized being submerged, (3) the salinity of the groundwater and (4) the stipulation that minimal (5mm maximum) movement of the cables occurs.

Jet grouting was not chosen for this site as the high pressure of grout installation was likely to have exacerbated the already fluid and loose nature of the existing sediment. Chemical grouting was ruled out due to the large volumes of gas, created as a result of the chemical reactions, potentially causing heave of the cable.

The purpose of grouting was to increase the sediment relative density to 35% or above, along the length of the subsidence affected area, effectively creating a bridge of denser sediment on which the cables could remain stable. The proposed grouting plan was not intended to impede ground water flow so tests were required to assess how far from the grouting spear site the grout permeated. Calculations using correlations of DCPT, CPT and soil type determined that post grouting CPT testing required a cone tip resistance, $Qc \geq 5MPa$, for a relative density of 35%.

A preliminary grouting model was developed to form parallel lines of denser material transecting the cable at 1 metre intervals. A cross section of the proposed ground treatment zone is shown in Figure 2. This model increases density along the length of the cable, forming support, while also retaining an overall high permeability sediment which does not create a dam effect. In order to grout the transects directly below the cable the model uses vertical and inclined spear insertion. A trial was carried out to ascertain whether this theoretical model would meet the requirements set out above.

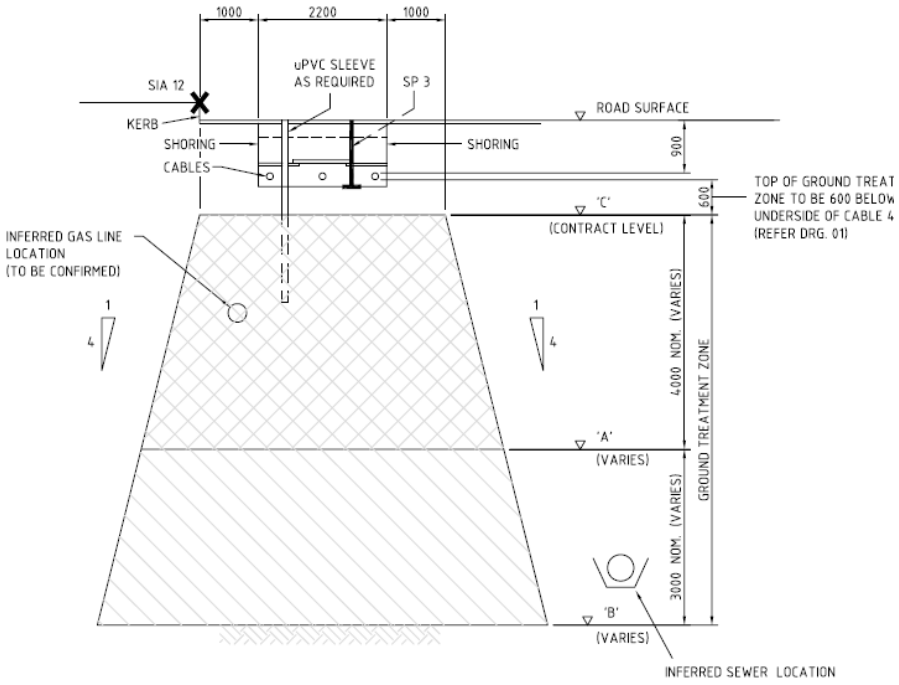


Figure 2 : Cross section of proposed ground treatment zone.

## 7. PERMEATION GROUTING TRIAL

A trial of two microfine cement base grouts was carried out to test the permeation extent of each at the site. The spears were pushed by hand down into the sediment. The trial locations were chosen adjacent to DCPT and CPT sites in the area of maximum settlement and to the north of the site, in order that post trial CPTs could provide the most information. The trial was conducted by the grouting contractor, GFW Australia Pty Ltd (GFWA). A summary of the trial is shown in Table 1.

| **Trial Name** | **Trial 1** | **Trial 2** | **Trial 3** |
|---|---|---|---|
| Grout | Alofix MC | Ultrafin 12 | Ultrafin 12 |
| Additives | MC Helper | GroutAid & SP-40 | GroutAid & SP-40 |
| Litres per 300mm | 45 | 45 | 90 |
| Maximum depth of spear per trial (m) | 4.0 | 4.5 | 3.0 |
| No. of holes trialled | 4 | 4 | 1 |

Table 1 : Summary of grout trial

Four spears were inserted into injection points between 500mm and 1000mm apart in Trial 1 and Trial 2. The extent of grout penetration was identified through a number of post trial CPTs. An example of analysis carried out at each trial location is shown in Figure 3. Results from CPT-9, CPT-10 and CPT-11 were recorded at a distance of 300mm, 1000mm and 500mm respectively, from the closest trial spear location (Figure 4).

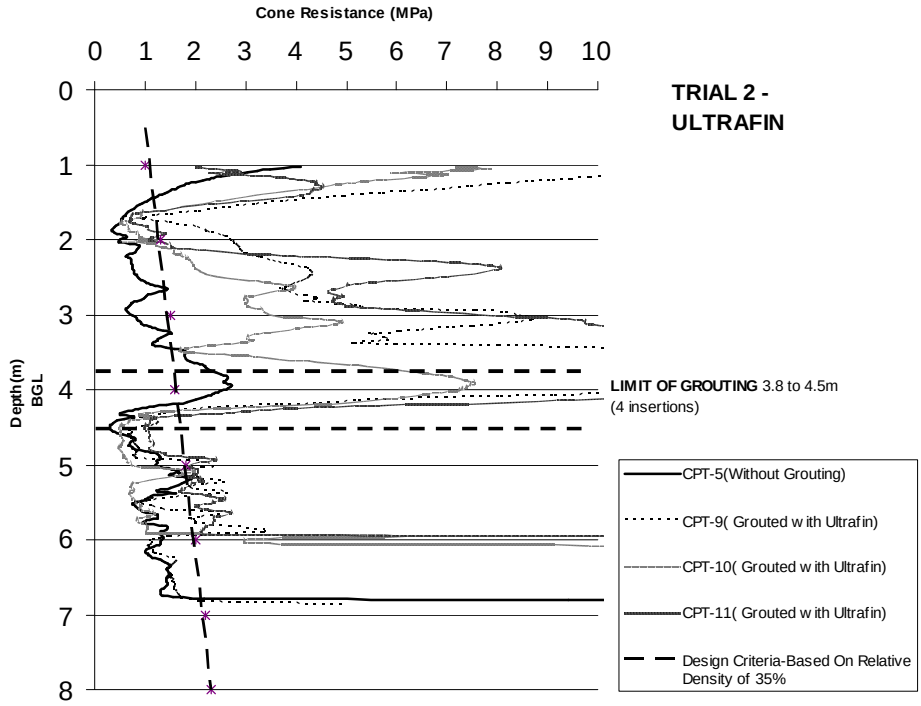


Figure 3 : Comparison of pre- and post-improvement CPT profiles

Cone resistance was shown to have increased in the grouted zone and immediately below the maximum depth of the grouting spear in each trial location.

## 8. RECTIFICATION WORK

The results of the trial led PB to recommend Ultrafin grout for the rectification work. A pattern of 19 rows (transects) of spear holes spaced 1.5m apart was adopted (Figure 4). The grouting method was reassessed by GFWA following the trial due to difficulty inserting the spears through fill. Rectification work utilised a track-air rig to insert the spear (a modified drill bit) without rotation to introduce grout. The methodology used primary and secondary holes with at least 24 hours curing time between installations, to avoid interconnectivity. The proposed maximum depth of spear insertion was defined by the known depth of sewer, borehole logs and CPTs which inferred the sandstone bedrock a minimum of 5m below ground level at the eastern end of the site.

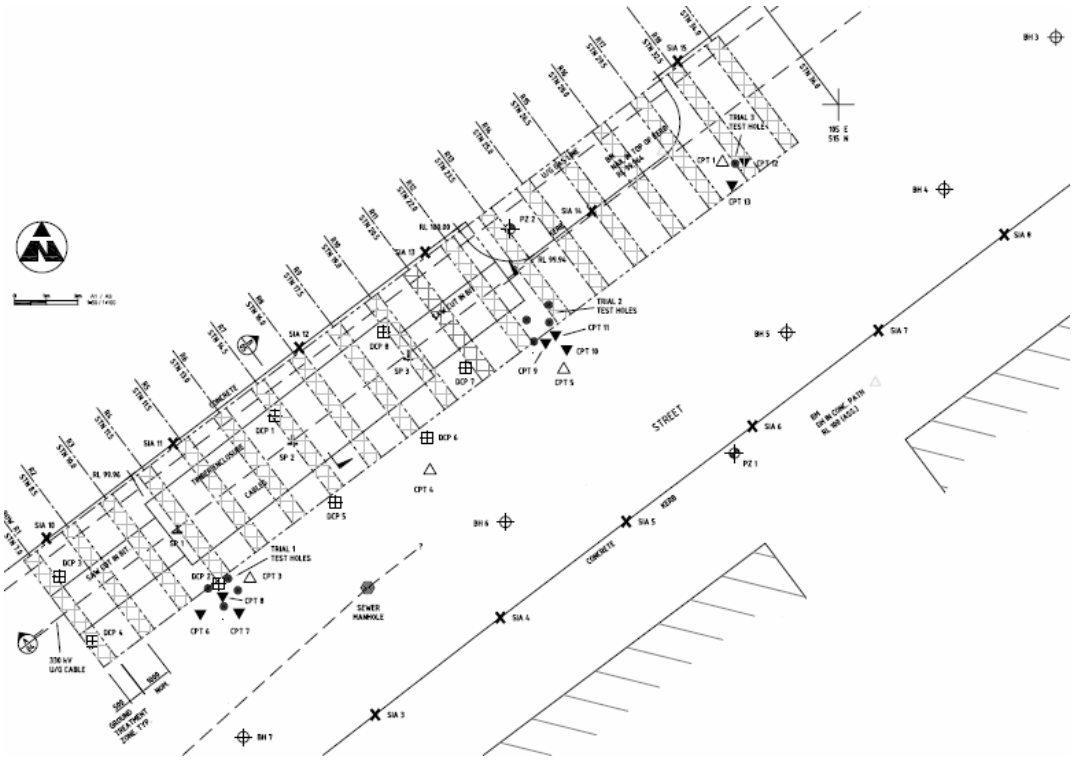


Figure 4 : Plan of ground treatment transects showing test locations

## 9. GROUNDWATER MONITORING

The aquifer at this site is understood to be an unconfined sand aquifer. An investigation was undertaken to assess whether the grouting process would cause a dam effect. Two vibrating wire piezometers were installed to measure fluctuations in water level throughout the grouting process and for the following three weeks. The water level traces were found to oscillate in sympathy with each other over an amplitude of 0.85m and 0.99m, indicating the aquifers at these locations are hydraulically linked. The groundwater flow direction was anticipated to be easterly to south easterly, towards the Cooks River and Botany Bay.

## 10. POST IMPROVEMENT VERIFICATION TESTING

At the completion of the rectification work, the following testing was undertaken:

*CPT* – Further CPTs conducted in areas treated with grout showed the cone tip resistance to have consistently increased to Qc $\geq$ 5MPa.

*Groundwater* – Monitoring of the two vibrating wire piezometers found that water level traces continued to show the two locations were hydraulically linked, providing evidence that the ground treatment had not caused a measurable dam effect.

*Settlement* - Settlement plates were installed at various positions across the site, including beneath the cables, and were monitored continuously during trial and remediation grouting procedures. Some settlement and heave was recorded during the grouting process.

The results of the above testing confirmed that the permeation grouting carried out was appropriate for the site and achieved the required stabilisation.

## 11. MANAGEMENT OF SITE ISSUES

*During Trials*

- Safety – Traffic was diverted, leaving one lane to open to one-way traffic. Traffic management halted traffic temporarily when required, with prior planning. All trial sites were located outside the cable trench due to the cable being live at the time. The cable trench was kept covered during the trial period with the exception of uncovering sections for observational purposes.

- Supply constraints - There were serious consequences to Sydney's continuity of power supply if the cables were damaged sufficient to take them out of service, or requiring additional repairs (eg cable sheath damage) resulting from this operation
- Timeliness - The cable damage incident involved not only distortion of the cables but also the risk of further distortion due to unstable soils at the site. It was therefore critical that prompt trials were taken to allow a proper and timely consideration of the trial results thus enabling a prompt determination of the treatment of the problem and to then allow the works to proceed as soon as was practicable.
- CPT rig access - The exact locations of the tests were limited by the size of available testing rigs, as in order to keep the road open during testing the rig had to be parallel to the trench and the road.
- Inserting spears – Problems with the spears terminating on rubble fill 100 to 200mm below the road surface was overcome by excavating shallow test pits to the top surface of natural sediment.
- Interconnectivity – At some spear locations grout was observed to bubble up out of adjacent holes at certain levels during the grouting process. This indicated that the high permeability of some sediment layers connected holes and, in one incident, connected with a sediment layer intersected by the cable trench, which resulted in grout pooling in the base of the trench. Following this, adjacent trial spear holes were grouted on alternate days and no grouting was performed above the level of the base of the trench.
- Permeation - Varying sediments highlighted in the CPTs posed a challenge with regards to how far the grout would penetrate. Permeation of the few clayey / organic layers noted was expected to be limited and inconsistent, while permeation of the sand layers during the trials was more likely to be reflected across the site during the rectification grouting.
- Excess grout water – During the process of grouting excess water is produced from inserting spears, flushing grout from spears and moving from hole to hole. The water is dirty, may contain grout particles and cannot be allowed to enter the storm water system. Sediment controls were set up to divert water to an existing pit which was later pumped out and removed from site.

*During Rectification work*

- Safety – The remedial grouting required spears to be inserted adjacent to the high voltage cables. This work could only be carried out during an electrical outage which was pre-arranged. Equipment and personnel were required to be clear of the cables when the cable is made live. TransGrid have a safety system in place for any work near their cables. As during the trials, traffic is managed to protect those on site.
- Supply constraints - There were serious consequences to Sydney supply continuity if the cables were damaged sufficient to take them out of service or requiring additional repairs (eg cable sheath damage) during this operation. - If the cables were damaged such that they would need major repairs (which would probably involve long lead times) this would cause serious issues for security of supply and high risk and consequences from failures in other parts of the network. This was unacceptable to TransGrid.
- Timeframe of outage – Return to service requirement of 4 hours was imposed during these works this was necessary in case another part of the Sydney power supply network would fail during this period. Careful team planning of the number and locations of spear holes in this area was required.
- Movement of cable – A limit of movement was set at +/-5mm. Further subsidence was limited by temporary works while heave was unlikely after no heave of ground surface had been monitored during trials. Despite this continuous settlement plate monitoring was carried out during the work.
- Permeability – Groundwater monitoring across the site was required to confirm that a dam effect had not occurred during or after the rectification grouting. Piezometer measurements revealed that the groundwater levels either side of the site were not affected by the grouting process. In the event that a dam effect had occurred the recovery method would have been to increase the permeability of the grouted mass by drilling a series of inclined holes.

## 12. ACKNOWLEDGEMENTS

The remediation site described in this paper posed many (non-geotechnical) challenges. The author would like to thank TransGrid for permitting publication.

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# SILVERSTREAM SEDIMENTATION POND: LESSONS LEARNT IN THE USE OF GREYWACKE FILL

**Andrew Kennedy**
*Tonkin and Taylor*
*Wellington*

## ABSTRACT

The construction of the Silverstream sedimentation pond involved the use of greywacke siltstone, sandstone and mudstone derived soil and rock as fill. The properties of this greywacke soil and rock material are variable which means that controlling compaction is challenging. As part of the continuing expansion of the larger Silverstream landfill site there will be considerable future earthworks using the materials encountered as part of the Sedimentation Pond construction. This paper recommends methods for sourcing and controlling the compaction of greywacke fill in subsequent contracts.

The geology of the area is discussed, specifically its implications for the construction of the sedimentation pond. A review is carried out of the selection and management of the borrow areas that supplied the greywacke fill. This includes the variation in the material's properties and moisture content conditioning that was required. The compaction control procedure for the sedimentation pond construction is examined in detail. The paper goes on to discuss the lessons learnt from the project and suggests the most appropriate ways in which to deal with the supply and compaction of greywacke fill.

## 1. INTRODUCTION

Silverstream landfill is located to the north of Wellington in a side valley to the east of the main Hutt Valley. The landfill has been in operation since 1972 and in that time has accepted approximately 3M $m^3$ of waste. The existing landfill site covers an area of around 15 ha. The footpint is being extended in stages to fill the valley downstream of the current landfill. This will increase the capacity by approximately 5M $m^3$ and will serve the local community for the next 30 to 50 years.

Stormwater from the perimeter and internal drains of the existing landfill was originally directed into two sedimentation ponds. These were located at the toe of the existing landfill. This area will be developed as part of the expansion of the landfill. These ponds were also undersized by current standards for effective silt retention. For these reasons it was necessary to construct a new, larger sedimentation pond further down the valley. The site chosen was approximately 1km downstream from the landfill in a narrow steep sided section of the valley.

## 2. GEOLOGY OF THE AREA

### 2.1 TYPICAL SOIL PROFILE

The basement rock at the landfill site and throughout the general Wellington region comprises interbedded sandstone, siltstone and mudstone sequences of the Rakaia Terrain, which is 200-230 million years old. This rock is generally known as "greywacke" and shall be referred to as such in this paper. The greywacke at the Silverstream landfill site is overlain by a combination of loess, colluvium, residual soil, alluvium and some man made fills (Tonkin and Taylor, 2003). Steep side slopes form the abutments to the sedimentation dam. The typical soil profile on these side slopes comprises a thin layer of topsoil, overlying surficial soil in turn overlying greywacke. This surficial soil is made up of residual soils up to 1m thick (greywacke that is weathered in place) and colluvial soils up to 2m thick (soils that have migrated downslope by erosion/ mass movement). On the valley floor greywacke is covered by a 3-6m thick layer of alluvial silts and gravels (Tonkin and Taylor, 2004a).

### 2.2 VARIABILITY OF GREYWACKE PROPERTIES

The engineering properties of greywacke rock depend on rock type and the degree of weathering.

### 2.2.1 Rock type

Greywacke rock on site is composed of interbedded claystone, mudstone, siltstone and sandstone. In unweathered rock the sandstone beds normally have the highest strength, followed by the siltstones, mudstones and then claystones. As the material becomes increasingly weathered there will still be distinct differences in the properties associated with the grain size and minerology of the parent rock.

### 2.2.2 Weathering Grade

The depth of the weathered zone depends primarily on the topography. The weathered layers are typically parallel to the ground surface. The depth to unweathered greywacke is greater on the flat hill crests and is less on the valley sides where loose highly weathered material will migrate downslope before a deep weathered profile can develop.

There are a number of other factors which can alter this pattern, some of which can make predicting the degree and depth of weathering difficult from the surface exposures. One of these factors is the erosion by surface water. In watercourses this is more rapid than the weathering process and so the weathered zone along the valley floor and side streams is thin or absent. The rock type also affects the weathering process - the finer grained rock types are more easily weathered so the claystone will break down more readily. The presence of faulting and associated fracturing of the rock increases the depth of weathering. These factors lead to a variable and unpredictable weathering profile.

## 3. THE SEDIMENTATION POND AND EMBANKMENT DAM

Work on the new sedimentation pond involved clearing the main reservoir area and constructing a 12m high embankment dam, Figure 3.1 (Tonkin and Taylor, 2004b). The primary purpose of the dam was to safely retain stormwater for silt control and flood detention. The dam was constructed without a liner so a low permeability structure was essential. Seepage control was achieved through the use of a low permeability core keyed into the side abutments and the bedrock. The material required for these shoulders needed to be high strength to ensure stability of the embankment. The upstream face of the dam was protected from wave action by placing a more rubbly rock fill armour. The quantities of fill required were 3,000m$^3$ for the low permeability core and 11,000 m$^3$ for the shoulder fill material. The grading envelopes for the two materials are shown on Figure 3.2.

**Figure 3.1**: Silverstream Sedimentation Pond and Embankment Dam

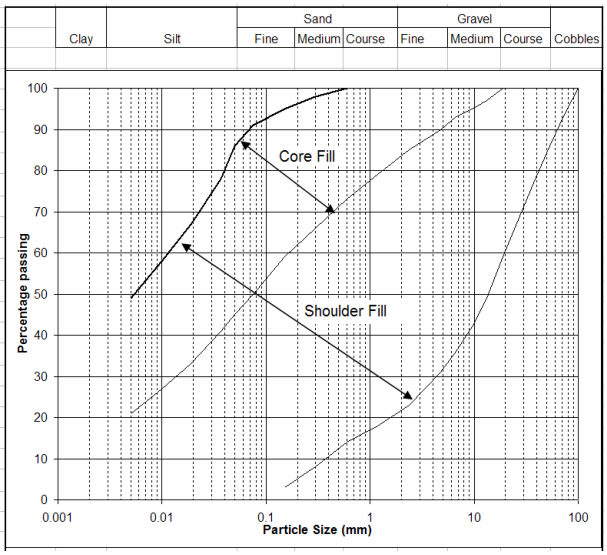


**Figure 3.2**: Particle Size Envelopes for Core and Shoulder Fill

## 4. SELECTION AND MANAGEMENT OF BORROW AREAS

Geotechnical investigations were undertaken by Tonkin and Taylor Ltd in 2004. Seventeen test pits and trenches were dug to assess the foundation integrity for construction of the pond and embankment, and suitable sources of fill material for construction of the dam. The fill types identified were from sources that were part of the bulk excavation. These were the cuts required for construction of the eastern spillway/ access road (Figure 4.1) and the clearing from within the reservoir storage area (Figure 4.2). The principal borrow area selected to supply the core fill was the cuts on the steep eastern side slopes of the valley.

## 4.1 BORROW AREA FOR CORE FILL

### 4.1.1 Material from bulk excavation work

The material excavated from the cut slopes on the eastern side of the valley (Figure 4.1) was a mixture of topsoil, colluvium, residual soils and various grades of the weathered greywacke. It was originally intended that the low permeability core fill would be made up of colluvium, residual soils and Extremely Weathered (EW) greywacke. There were two main reasons why this was unsuccessful:

- **Time of year.** The contractor was behind programme during this contract and the bulk of the excavation was carried out in winter. At this time of year working the steep cut face to separate the different material types became more difficult.
- **Depth of suitable material.** Because of the steepness of the slope, geotechnical investigations were limited to scala penetrometer sounding. These showed a thin layer of surficial material approximately 1.0m deep. When the benches were cut, the depth of this layer was found to be more variable than expected.

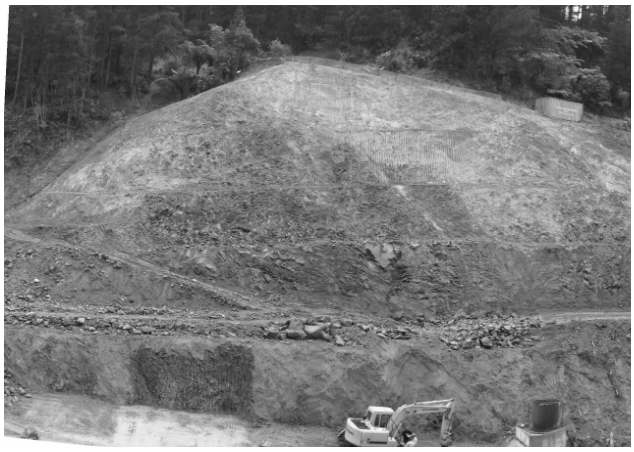

**Figure 4.1**: Eastern cut slopes for construction of the spillway and access track

**Figure 4.2:** Excavation of the alluvial fan material at the upstream toe of the embankment

A second source for the low permeability core fill were the alluvial/colluvial fans that were within the pond footprint, Figure 4.2. This material had to be stripped to increase the storage capacity of the pond. It contained sufficient fines for use as a low permeability fill and its moisture content was approx 4-5% above Optimum Water Content (OWC) when it was excavated. Earlier testing on this material had indicated a similar insitu moisture content and if the earthworks had been carried out in the summer months it would have been possible to sufficiently dry this material. At the time of stripping this was not possible, especially with the limited drying and space available at the base of the valley.

### 4.1.2 Material sourced from alternative borrow area

A geotechnical investigation was undertaken to find an alternative borrow area for low permeability fill. The alternative borrow area was a relatively flat hill crest, approximately 1km to the south east of the sedimentation pond site. Part of the area had been previously used as a source of material for the daily landfill cover and so benches had already been cut. The majority of the residual soils and the EW greywacke had already been removed from these benches, but there were some small areas of low permeabilty material on the edge of the borrow. There was also a more extensive source of residual soils and EW greywacke on the hill crest adjacent to the landfill borrow area.

The material was a mix of residual soil with some EW greywacke. The residual soil was clayey silt, moist-wet with moderate plasticity, and the EW greywacke was sandy gravelly silt, dry-moist. The gravel clasts broke down under

compaction to a sandy silt. Both these materials had suitably low permeability for use as core fill. However, the variation in material properties within the borrow areas made it difficult to generate a homogeneous fill.
There was a large range in the insitu moisture content relative to the OWC. The residual soil was typically around 31% moisture content with an OWC of around 24-25%. The EW greywacke was usually drier than the OWC of 22-23%. This meant that conditioning of the material was essential. This was done at the borrow platform which had the advantages of space and improved drying conditions. Conditioning involved mixing the two materials, drying them at the borrow platform and regularly checking the moisture content. The fill was carted down to the works site when it reached OWC.

### 4.2 SHOULDER FILL BORROW AREA

The shoulder fill material was excavated from the eastern cut batters for the spillway and access road, Figure 4.1. This material was primarily required to be a high strength fill and the permeability was not as much of a controlling factor. This meant that the range in the particle size distribution was much broader than the core fill. The shoulder fill material ranged from surface soils to EW and Moderately Weathered (MW) greywacke. In general this material could be compacted to a suitable strength and air void content straight from the cut face. Conditioning was only required in the latter stages of the project when the material was below OWC and there was a greater proportion of more blocky MW greywacke rock. This fill material required wetting before a suitably low air void content could be achieved.

## 5. COMPACTION TESTING

### 5.1 GENERAL

The moisture content and wet density of the compacted fill were tested using the Nuclear Densometer (NDM) and oven moisture contents, and with sand replacement tests to calibrate the NDM. The strength of the compacted fill was determined through use of a Scala penetrometer probe for the granular fill and a shear vane for the more cohesive core fill. The strength was also monitored by observing the weave of the fill under the compactor. The specification for compaction testing is as set out in Table 5.1 below:

| Material Type | Min. dry density | Max. air void content | Minimum Strength |
|---|---|---|---|
| Core Fill | 1600kg/m$^3$ for 10 test average. 1550kg/m$^3$ for single test. | 5% | 100kPa (measured by shear vane) |
| Shoulder Fill | | 10% | 3blows/50mm (measured by Scala) |

Table 5.1 Compaction Criteria

Insitu permeability testing was also carried out using a standpipe permeameter test. The maximum permeability permitted for a field test in the core was $1x10^{-7}$ m/s. Testing showed that the core fill had a permeability ranging from $4.8x10^{-8}$ m/s to $3.0x10^{-9}$ m/s. The permeability of the upstream shoulder was also tested. Despite the relaxed air void content and the broad range in the grading of this material the hydraulic conductivity was only slightly higher, with tests ranging from $1.6x10^{-7}$ m/s to $1.5x10^{-8}$ m/s.

The broad grading range permitted for the shoulder material as well as the greater permitted air void content meant there was generally little difficulty in supplying this material or in achieving the specified compaction. The following discussion focuses on the low permeability core fill in which it was more difficult to achieve the compaction criteria.

### 5.2 CORE FILL

Testing results from the geotechnical investigation suggested that the core and shoulder fill materials would have a Maximum Dry Density (MDD) typically ranging from 1550kg/m$^3$ to 1650kg/m$^3$. The Proctor curve for a clayey silt material with MDD of 1620kg/m$^3$ and an OWC of 22% is shown on Figure 5.1. The minimum dry density criterion reflected this expected range. The low permeability fill material used in this contract did not have a consistent MDD so controlling compaction using a dry density criterion was not appropriate.

Figure 5.1 shows the NDM test results for the core fill. A pattern in the distribution of dry density and moisture content can be seen over the duration of the works as the grading of the material changes. In July and August the relatively high dry density values and lower moisture contents represent the residual soil/EW mix excavated from the existing cut benches at the borrow area (sandy silts with fines content of approx 50%). In September, October and November the material was excavated from the crest of the hill which was typically residual soil (clayey silt with fines content of up to 95%). This material had a much lower MDD (1450-1550kg/m3) and higher OWC (25-26%).

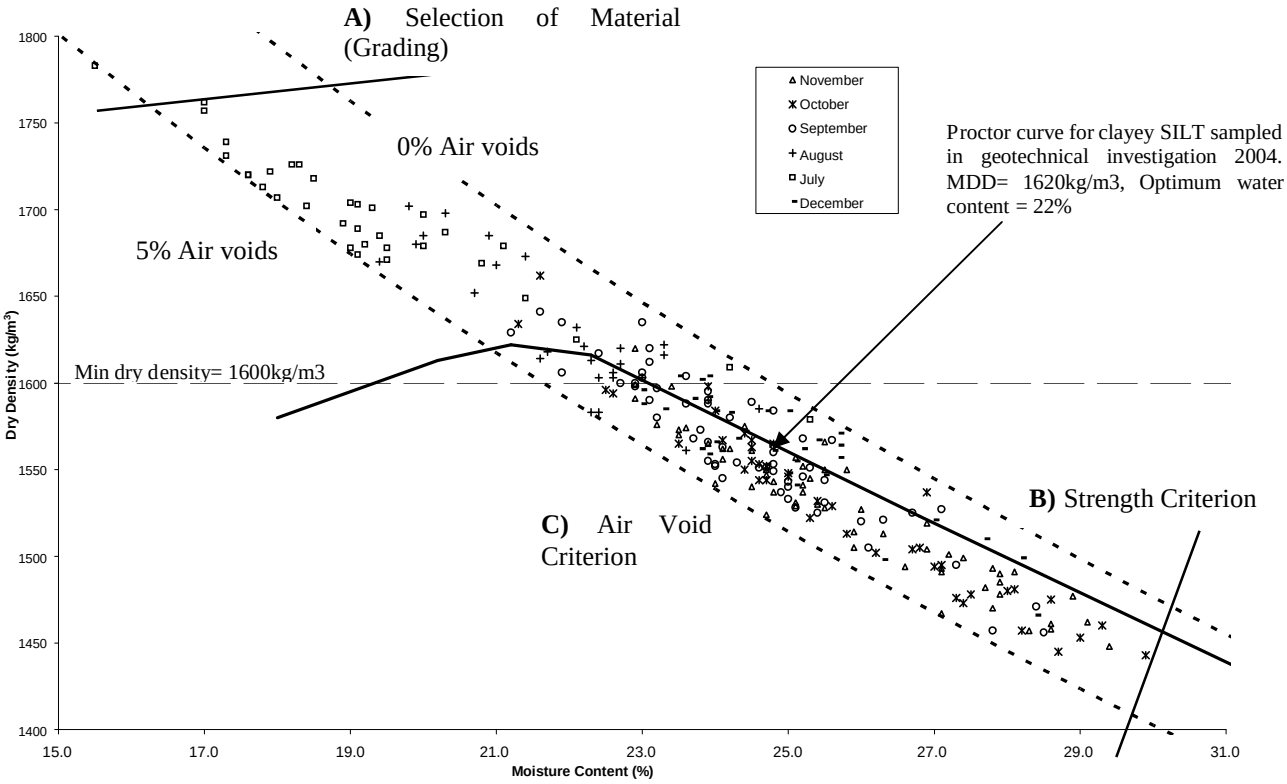


**Figure 5.1**: Nuclear Densometer Testing- Core Material

The testing requirements for the fill were amended during the works because of the variation in the grading of the material within the selected borrow area. The minimum dry density criterion was removed from the testing specification to account for this. Despite the variability in the material properties there was still a large (1.0-1.5m) layer that could be used as core fill. The following methods were used to ensure that compaction requirements were met:

- **Selection of Material (Grading)**. Through a visual inspection and by calibration using grading curves it was possible to select and monitor the material types used for core fill. The grading curves showed if the material was becoming too coarse or if the fines content was too low. The instu permeability test can then be used to confirm the material has been adequately compacted
- **Strength Criterion.** Testing of the strength of the material became of greater importance to ensure that the moisture content of a material was not too far above OWC. This was done through shear vane testing and by monitoring the weave of the fill.
- **Air Voids Criterion**. It was also important to keep a close check on the air voids content because a high strength could be achieved at a moisture content lower than the OWC. Whilst this would provide a high strength material at the time of compaction it could lead to high permeabilty values and there is a risk the material could lose strength on saturation.

For each of the above criteria it is possible to illustrate the bounds they place on the dry density and moisture content of the compacted fill material. These limits are shown on Figure 5.1. The grading criterion (A) ensures that there is no oversize material and that the fines content is sufficient to keep the permeability low. The strength criterion (B) ensures

that the material's moisture content does not greatly exceed the OWC for that particular grading. The air voids criterion (C) ensures that the material is densely compacted at, or wet of, OWC so that the permeability remains low.

# 6. LESSONS LEARNT IN THE USE OF GREYWACKE DERIVED FILLS

## 6.1 SELECTION OF A SUITABLE BORROW AREA

Using the bulk excavation works to supply the fill quota is a conventional and potentially cost efficient option. It was intended that the bulk fill for the Silverstream dam would be won from benches cut to enable construction of the eastern spillway and access road supplemented by colluvial and alluvial fan deposits. In this particular project a cut to fill operation was not suitable and an alternative borrow area was used.

The savings that can be made in having a separate borrow area that has a plentiful supply of suitable and easily extracted material, space for conditioning and good access can outweigh the savings that come from trying to utilise the cut material in difficult conditions.

If there is no reasonable alternative and a cut to fill approach is adopted then it is important to ensure that a detailed methodology is agreed with the contractor prior to accepting a tender. This methodology should state how the benches or other excavations will be made and how the material will be identified, separated, stockpiled and conditioned for use as fill.

## 6.2 COMPACTION CONTROL

The most common method for the control of fill compaction is by dry density and a narrow range of moisture content either side of optimum. Trying to control compaction by using absolute (rather than relative) limits on dry density and moisture content is not appropriate unless a consistently homogeneous material type can be supplied. The maximum dry density (MDD) and optimum water content (OWC) for greywacke derived soils and weathered greywacke material are variable even within a small borrow area.

A more appropriate and flexible method of controlling compaction is through an air voids criterion, a strength criterion and a careful selection of fill material. This allows a much wider range of material to be used and still achieve a well compacted, low permeability fill. The targets for the percentage air voids and strength need to be chosen based on the specific requirements of each earthworks project so that they are robust and flexible enough to control the fill compaction.

# 7. ACKNOWLEDGEMENTS

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# THE LAKE COLERIDGE ROCK AVALANCHE DEPOSITS, CANTERBURY

**Jenny Lee**
*Tonkin & Taylor, Wellington, New Zealand*

## ABSTRACT

Three separate rock avalanche events have been identified from a single source area at the summit of Carriage Drive, near Lake Coleridge in Canterbury, New Zealand. They have been dated between 9,700 years and 600 years B.P. A previous regional study of rock avalanches dated a single rock avalanche deposit at 150 ± 40 years B.P. at this site. This paper provides details of both the age and the sequence of events at the site, associating the deposits with postglacial tectonic activity. Each of the three events blocked the Ryton River, and anomalously steep downstream outwash terraces record dambreak floods from the last two events. A flat upstream terrace suggests that a lake existed there for some time after the second event. These events imply significant hazards for a small campground on the Ryton River delta on the lakeside downstream of the site.

## 1. INTRODUCTION

The Lake Coleridge Rock Avalanche Deposits (LCRADs) are located on Ryton Station in the middle Rakaia Valley, 80km west of Christchurch (Fig. 1). Since the Late Pleistocene glaciations, extensive tectonic, seismic, erosional and paraglacial processes have reshaped the landscape through of the middle Rakaia Valley through fluvial action both during and after ice retreat, fan development, and slope failures of all sizes.

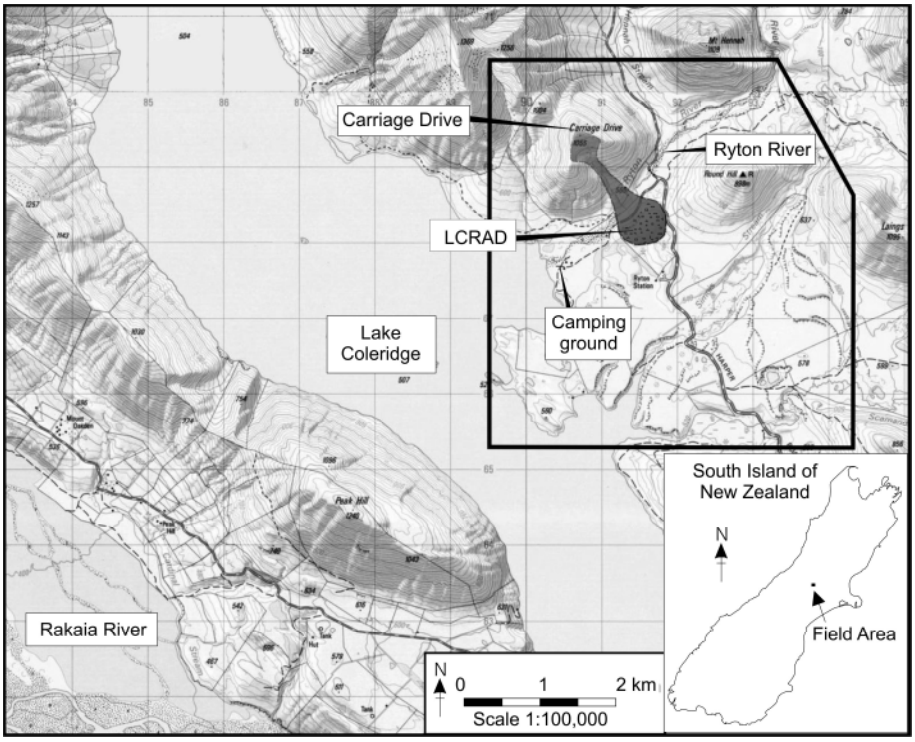


Figure 1: Location of the Lake Coleridge Rock Avalanche Deposits

Rock avalanches represent the large end member of a continuum of slope failures. They are often coseismic and deep-seated and fail catastrophically. Traveling at extremely high velocities (~50m/sec) they tend to break-up and fragment during motion. Numerous studies (e.g. Whitehouse, 1983; Whitehouse and Griffiths, 1983; Prebble, 1995) show that rock avalanches are a significant geomorphic process in the Southern Alps of New Zealand.

Eisbacher and Clague (1984) and Whitehouse and Griffiths (1983) suggest a minimum volume (~ $10^5$ - $10^6$ $m^3$) at which a rockfall changes from independently falling clasts into a flowing stream of disintegrating rocks which constitute a rock avalanche with deposits overlying valley floor topography and commonly extending up opposing valley slopes. A common consequence of rock avalanche deposit emplacement is the instantaneous delivery of large volumes of sediment to drainage networks (Hovius *et al*, 1997; Korup *et al*, 2004). Geomorphic impacts from rock avalanches include valley floor burial, river channel alteration, large-scale fanhead avulsions (Korup et al., 2004), and the creation of natural dams (e.g. Costa and Schuster, 1988).

## 2. REGIONAL TECTONIC SETTING AND GEOLOGY

Basement rock in Canterbury is Torlesse Supergroup greywacke, the most easterly of the Permian and Mesozoic terranes in New Zealand, and was accreted to the Pacific edge of the Gondwana margin approximately 250 million years ago (Bradshaw *et al*, 1981). Within the Canterbury area the basement material is the Rakaia sub-terrane, the rocks of which are well indurated, weakly metamorphosed bedded sequences of moderately to poorly-sorted coarse to fine sandstone and siltstone, interpreted as submarine fan type sequences (MacKinnon, 1983). Inland Canterbury is characterized by a tectonic transition from subduction to collision: the upper crust is dominated by thrust faults associated with growing folds (Chamberlain, 1996). The Lake Coleridge area straddles the active boundary of the Australian and Pacific Plates (Norris *et al*, 1990). To the north, major dextral northeast trending predominantly strike-slip faults of the Marlborough Fault Zone transfer oblique plate motion from the Hikurangi Margin through to the Alpine Fault (Chamberlain, 1996). The southwest termination of the active right-lateral strike-slip Porter's Pass-Amberley Fault Zone (PPAFZ) is located approximately 10km to the southeast of the LCRAD (Smith, 2003).

## 3. MECHANISMS AND MORPHOLOGY OF ROCK AVALANCHES

Rock avalanches are normally derived from large (>1 x $10^6 m^3$) and relatively intact rock masses (Smith, 2003). Initial failure predominantly occurs along pre-existing defects, a process described as "collapse" by Davies and McSaveney (2002). A rock avalanche transforms from a semi-coherent rock mass to a fragmenting mass starting just after detachment, allowing debris to "flow", and to reach extremely high velocities with long runout distances if allowed by topography. They may pose a serious threat to human lives and structures.

Triggering mechanisms for rock avalanches include; earthquakes, over-steepening (including both human excavation and glacial and fluvial erosion), dilation of the rock mass, rock weathering and storm events. Seismic shaking from earthquakes is commonly perceived as the most important mechanism for rock avalanche activation (e.g. Keefer, 1984), but Smith (2003) noted that only two of the six historic rock avalanches in New Zealand were triggered by earthquakes.

Earthquakes however do provide one of the main requirements to initiate a rock avalanche, which is the instantaneous detachment of a rock mass from its parent edifice. The effects of seismic shaking on rock masses include comminution, dilation of fractures, joints and defects, reducing the shear strength along failure surfaces (Smith, 2003). Based on the estimated age of the deposits, it is considered likely that the LCRA events were triggered coseismically.

The geometry of the LCRAD are described (Nicoletti and Sorriso-Valvo, 1991; Govi et al., 2002) using the following factors: the maximum runout distance *(L)* is measured from the top of the source scar to the distal extent of the deposit; the elevation difference *(H)* is measured between the two points that define *L; D* is the length of the rock avalanche. The ratio *H/L* is known as the "Fahrböschung"; it has been used to quantify excessive travel distance, but this logic has recently been questioned by Davies and McSaveney (1999), who showed that the relationship between *H/L* and internal friction was complex. However, excessive travel distance *($L_e$)* of rock avalanche deposits is still a widely used concept, and the Fahrböschung is an empirical measure of this.

## 4. THE LAKE COLERIDGE ROCK AVALANCHE DEPOSITS

The LCRAD site has previously been studied by Whitehouse (1983) and Whitehouse and Griffiths (1983). They identified six rock avalanches in close proximity to the LCRAD site, but included the LCRAD as a single rock

avalanche event. Whitehouse (1983) dated the LCRAD at 120 years old using weathering rind depth; Whitehouse and Griffiths (1983) redated it, again using weathering rinds, at 150 ± 40 years B.P. The source area for the deposits is 1055 m above sea level known as Carriage Drive, a dislocated peak of the Cottons Sheep Range (Fig. 1).

The present day total volume of the LCRADs is estimated at 5.5 x $10^6$m$^3$ and a further 7.5 x $10^6$m$^3$ is estimated to have been eroded by the Ryton River since emplacement. The LCRAD debris is highly fragmented, compacted and chaotic; there are internal flow structures visible in eroded cliff sections within the Ryton riverbed; and the debris "flowed" over the local topography. Rock avalanche deposits often have a rough or hummocky character, and some have lobe-shaped fronts. Rock avalanche deposits can be chaotic and/or inversely stratified (Cruden and Varnes, 1996); there is field evidence for both of these at the LCRAD. This study identified three separate rock avalanche deposits (Fig. 2) from the same source area. An older and larger rock avalanche deposit (LCRA1) overlies glacial till and associated outwash surfaces. Two younger and much smaller deposits (LCRA2 and LCRA3) are located within the eroded Ryton River section of the older deposit. The deposit volumes decrease from LCRA1 (10 x $10^6$m$^3$; this study), to LCRA3 (0.1 x $10^6$m$^3$) which is of very limited extent.

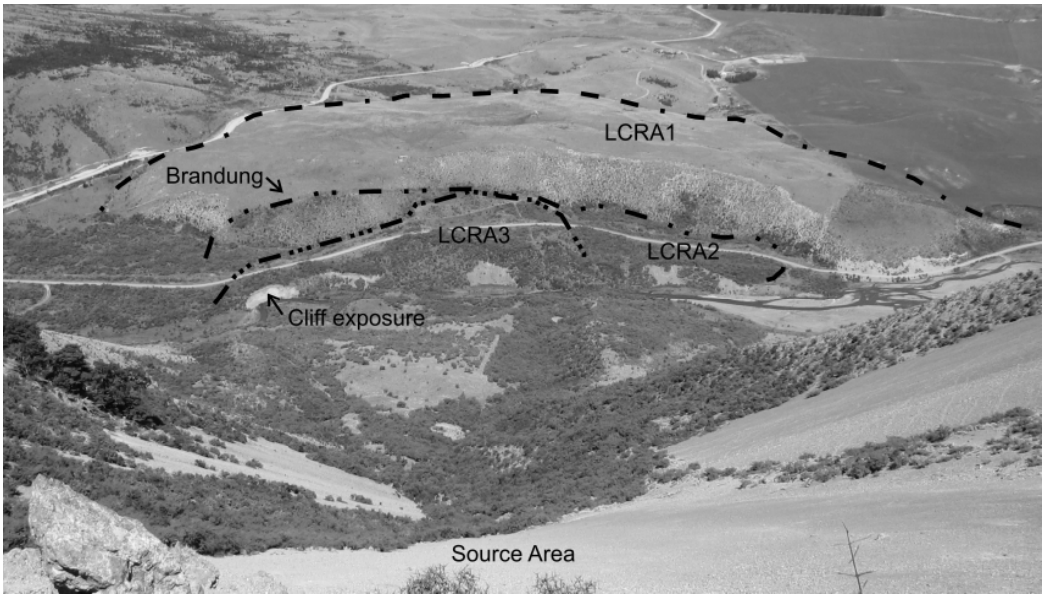


Figure 2: Three separate Lake Coleridge Rock Avalanche Deposits viewed form the source area. Ryton River runs from left to right across the centre of the picture.

### 4.1 LCRA1

LCRA1 is located at the southeastern base of Carriage Drive. Debris of the event buried the Ryton riverbed and traveled up and over the glacial terraces on the opposite side of the river. The deposit displays a classic lobate structure (e.g. Mitchell *et al.*, 2001), and has a runout of 1780m (Table 1). The distal extent is ~350 m north of the Ryton Homestead, and ~800m upstream of the Ryton campground. This is close to human infrastructure, given the sparseness in population along the Southern Alps. The volume of LCRA1 from this study is 13 x $10^6$ m$^3$, covering an area of 0.55 km$^2$. Whitehouse (1983) gave a significantly smaller volume of 4 x $10^6$ m$^3$. A possible explanation for this difference in volume calculations is the calculation of volume eroded by the Ryton River - this study estimated 7.4 x $10^6$ m$^3$ of material has been removed, whereas Whitehouse (1983) calculated 0.7 x $10^6$ m$^3$ had been eroded.

The internal structure of the rock avalanche deposit varies with distance from the base of Carriage Drive. The medial deposit comprises highly compacted, shattered, granular to small gravel sandstone clasts, with a-axes aligned perpendicular to the emplacement direction. These are surrounded by highly fractured, and in some places sheared argillite (mudstone) units. This resembles an emplacement bedding structure, with alternating beds of sandstones and mudstones, similar to the intact rock mass characteristics of the Carriage Drive source area. Such preservation of source structure is common in rock avalanche deposits. Towards the periphery of the deposit, clast size increases and a chaotic fabric exists with no apparent bedding. Trenching was carried out around the periphery of the deposit, exposing both the rock avalanche debris and the underlying glacial till surface. A 150 mm horizon of silty loess was present between the two deposits. From this horizon an optically stimulated luminescence (OSL) sample was collected and dated at 9,720 ± 750 years B.P. This provides an upper limit for the age of LCRA1.

| | |
|---|---|
| Total planimetric surface *(S)* | 0.701 km² |
| Runout *(L)* | 1780 m |
| Length *(D)* | 1725 m |
| Elevation difference *(H)* | 495 m |
| Excessive travel distance ($L_e = L - H/tan\ 32°$) | 1030 m |
| H/L; Fahrböschung = arc tangent *(H/L)* | 0.28 (15.5°) |
| $L_e/L$ | 0.58 |
| Length of accumulation | 450 m |

Table 1: Runout Data for LCRA1

The triggering mechanism for the rock avalanche event is likely to have been one or a combination of; steepening of the rock slope by glacier erosion; rock mass dilation due to post-deglaciation rebound of the rockmass after the last glaciation; and a seismic event.

### 4.2 LCRA2

LCRA2 clearly came from the same source area as LCRA1; it is much smaller in areal extent, volume and geomorphic expression. It is unclear whether this rock avalanche deposit was incorporated into the Whitehouse (1983) and Whitehouse and Griffiths (1983) data set. It was possible that it was not, as it failed to meet their minimum volume threshold of $10^6$ $m^3$. LCRA2 has an estimated loose volume (accounting for "bulking") of 523,000 m³, with an average thickness approximately 6m. It buried the area between the base of Carriage Drive and LCRA1, where the Ryton River had incised into the earlier deposit. The boundary between the LCRA1 and LCRA2 is easily distinguished by a prominent "brandung" (backwards facing scarp: Hewitt, 1998). The runout *(L)* was 1350 m from the source scar to the distal edge of the deposit. The elevation difference *(H)* between the two points that define *L* was 470 m, giving a Fahrböschung of 19° similar to that of larger rock avalanches elsewhere.

Internally the LCRA2 deposit is chaotic. It is matrix-supported with a large percentage of fines (pebbles to fine sands), with larger angular clasts spaced randomly throughout. Very little structure is visible within the lower portion of the section, but towards the top, faint alternating pseudo bedding or lenses of sandstones and mudstones are present. This type of layering ("inverse stratification") is common in rock avalanche deposits. The Ryton River has incised through the deposit, recently exposing a long cliff section, from which a piece of *Leptospermum scoparium* (mānuka) was radiocarbon dated at 668 ± 36 years B.P. It is likely that the mānuka tree was incorporated into the debris by the rock avalanche event, thereby dating it. Continued dilation of the rock mass on Carriage Drive after the first rock avalanche is likely to have been the precursor of the second event. However LCRA2 occurred during a period of high seismicity between 500 and 700 years B.P. This activity occurred both along the western segment of the PPAFZ, with rupture dates of 1,152 ± 51 years and 600 ± 100 years B.P. (Smith, 2003), and along the Alpine Fault whose most recent ruptures are well constrained at 850 years B.P., 500 years B.P., 330 years B.P. and 253 years B.P. (Yetton *et al*., 1998). Hence a coseismic trigger is likely for LCRA2.

### 4.3 LCRA3

The source of LCRA3 is again Carriage Drive. The deposit is far smaller in extent, volume and geomorphic expression than that of LCRA2. This rock avalanche deposit was first identified by this study. The deposit directly overlies LCRA2 and is visible in an eroded cliff section within the Ryton riverbed (Fig. 3). Its thickness is about 2 m. The morphology of the deposit is poorly defined and the contact with the underlying LCRA2 deposit is mostly unclear. The small size of the deposit made statistical analysis impractical. The contact between the LCRA2 and LCRA3 deposits is poorly defined away from the cliff section. The lack of a soil profile on the LCRA2 deposit indicates that LCRA3 may have occurred shortly after the LCRA2 event – unless the soil was stripped during emplacement.

Figure 3: Cliff in riverbed exposing contact between LCRA2 & LCRA3 deposits

## 5. EFFECTS ON THE RYTON RIVER

A major impact of rock avalanches is the rapid delivery to the valley floor of large volumes of fragmented rock debris. Due to the close proximity of mountains and valleys to rivers and streams, the impacts on fluvial systems are usually significant. Rock avalanches also create temporary or semi-permanent natural barriers impeding the flow of water and sediment (Costa and Schuster, 1988; Korup *et al*, 2004). This impact of rock avalanches on fluvial systems depends upon whether they impound water and sediment; lake volume; dam duration; and how the dam breaches. The main factors controlling rock avalanche dam stability are the size and shape of barrier and the composition and compaction of the deposit (Hewitt, 1998). Valley geometry also plays a critical role in determining if the valley is narrow enough to be blocked with sediment, or if the barrier can simply be bypassed by the river.

A lake bench upstream of the rock avalanche deposit suggests a significant period of damming of the Ryton River by LCRA1. The lake covered an area of approximately 1.1 km² with an estimated volume of $1.7 \times 10^7$ m$^3$, assuming present day valley topography. Overtopping failure of the dam is calculated to have occurred within 100 days using the formula of Davies and Scott (1997), assuming that there was no significant seepage of water through the dam, and mean annual flow conditions.

The effects of LCRA2 and LCRA3 are discussed together because they are believed to have occurred close together in time. The debris crossed the Ryton River and was confined within the low-lying topography formed by erosion of the Ryton River into LCRA1. Topographic constraint of the LCRA2 runout is clearly indicated by the well developed "brandung" feature (Fig. 2). LCRA2 is believed to have temporarily dammed the Ryton River. The maximum dam height (7m) was substantially less than that of LCRA1. The surface area of the LCRA2 lake using the present-day valley topography was 0.46 km², with a volume of ~ $2.3 \times 10^6$ m$^3$. The dam is estimated to have overtopped after approximately two weeks, with a consequent dam-break flood preserved by three dam-break outwash terraces directly downstream of the LCRA2 deposit. These terraces formed by deposition of debris from the flood flow, which carried very high bedload sediment concentrations. The terraces were identified by their steepness of gradient and the angularity of the clasts which indicate a hyper-concentrated flow deposit; significantly different from the local river deposits. The small size of the LCRA3 deposit, the lack of a soil horizon, and its location within the same runout area as the LCRA2 deposit suggest that it occurred very soon after LCRA2. The LCRA3 deposit may well represent a lag deposit from the second rock avalanche event, and the Ryton River may already have been dammed by LCRA2 at the time of LCRA3.

## 6. FUTURE HAZARDS

Continued dilation by gravitational toppling-style failure occurs along the sub-vertical argillite beds on the two main ridgelines of Carriage Drive, and has produced a 35,000 ± 5,000 m³ wedge-block of material displaced from the in situ rock mass. An earthquake may initiate detachment of in situ bedrock from the source area, therefore the resulting rock avalanche could potentially be larger than the current estimated displaced block of material, and would pose a risk to tourist or farming operations adjoining the Ryton River.

The campground at the mouth of the Ryton River (Fig. 1) is at risk of rainstorm or snowmelt-induced flooding, especially since the delta is prograding into the lake and therefore aggrading in the long-term. Any future mass movement from Carriage Drive (most likely during an earthquake or major rainstorm) poses an additional risk of landslide damming followed by sudden dambreak flooding, and river bed aggradation as the dam material is reworked by the river. The size of the rock avalanche event will determine both the consequences to the occupants at the campground and the land owners and the required emergency response. Education of the land owners about options to mitigate the hazards and an emergency response plan should be implemented. Signage should be erected for the general public outlining the hazards and the correct emergency response procedures.

## 7. ACKNOWLEGEMENTS

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# GEOLOGICAL MODELLING IN THE NORTHLAND ALLOCHTHON, NORTH AUCKLAND, NEW ZEALAND

**Kori Lentfer** BSc (Tech), PG Dip (Earth Science)
*Engineering Geologist – Foundation Engineering Limited*
*Orewa, New Zealand*

## ABSTRACT

The Northland Allochthon (referred to by practitioners as Onerahi Chaos Breccia) is recognised as being geotechnically challenging to engineer, on account of its sheared and shattered nature, low strength and expansive characteristics.

The development of detailed geological models is critical to ensure adequate and appropriate measures are taken to stabilise slopes for building and infrastructure development. In order to develop these models a combination of geological and geomorphic mapping, subsoil investigations, groundwater analyses and laboratory testing is commonly utilised. Once the model has been developed an evaluation of various measures for effective mitigation of slope instability can then be undertaken with more confidence.

## 1. INTRODUCTION

Slope instability in Northland Allochthon soils poses a significant risk to building and infrastructure development. As growth and development continues in the greater northern Auckland region, steeper and more unstable land is being developed requiring comprehensive site investigations to formulate accurate geological models. Once this has been undertaken, appropriate and cost effective slope stabilisation measures can be designed.

The Northland Allochthon underlies much of Northland, New Zealand extending as far south as Silverdale on the northern outskirts of Auckland city as shown in Figure 1. The scope of this paper is limited to the geological units that are commonly encountered in the greater Silverdale area and focuses on the importance of developing detailed geological models for slope stability analyses specifically within the Northland Allochthon.

## 2. GEOLOGICAL SETTING

### 2.1 Allochthon Formation and Emplacement

The Northland Allochthon has different lithologies which vary greatly in nature and strength. How the Allochthon was formed has been the subject of great debate over the years. It is now commonly accepted that both sedimentary rocks and submarine basalts originally deposited in a sedimentary marine basin off modern day northeastern Northland were transported by gravity sliding and/or tectonic thrusting as nappes in a generally southwest direction towards present day Silverdale resulting in tectonic superposition of these materials (Hayward *et al*., 1989). The deformed, sheared, faulted and folded lithologies provide good evidence of tectonic deformation being involved in transportation and emplacement of the Northland Allochthon (Rafferty, 2001). Distal parts (including the Silverdale area) were transported at least 350km with the maximum transportation rate estimated at 350mm/year following up to us much as 14km of uplift of the depositional basin (Balance & Spörli,1979). In these distal areas the base of the allochthon was emplaced into the early Miocene aged Waitemata Basin.

In general, the non-calcareous or siliceous mudstones and siltstones that make up the majority of the allochthon are almost always identified as being highly shattered and consisting of tightly interlocking plate like or rhombic fragments. Calcareous lithologies tend to be more resistant to weathering, suffer less frittering and are often more stable with broader fracture patterns and larger particle sizes than siliceous materials. Remobilisation of calcite during weathering may recrystallise in voids and on defect surfaces to bind the rockmass together.

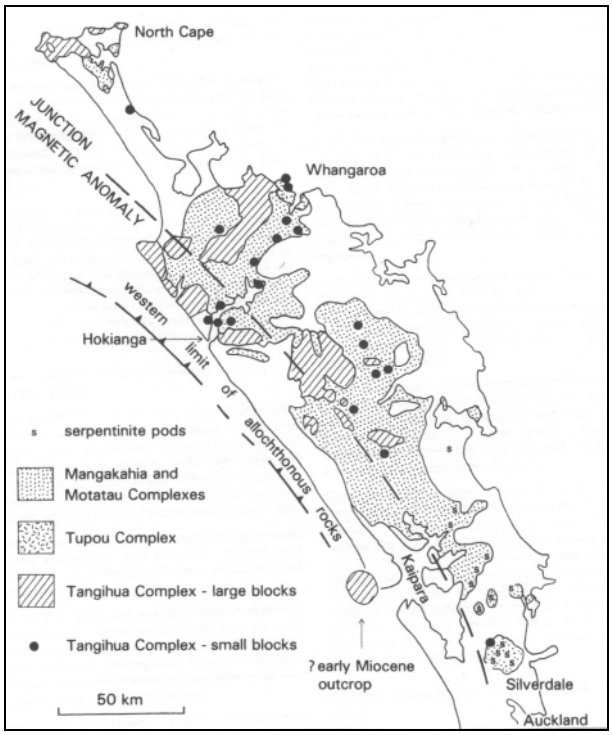


Figure 1: Spatial Distribution of the Northland Allochthon (Hayward et al., 1989).

2.2 Lithological Units

In the Silverdale area, the Mangakahia Complex is the most abundant geology comprising the Whangai Formation and Hukerenui Mudstone. Allochthonous Akarana Supergroup/ Waitemata Group, Mahurangi Limestone and isolated Puriri Mudstone of the Motatau Complex can also be found. The Tupou and Tangihua Complexes as shown in Figure 1 are not encountered in Silverdale. The main lithologies encountered here are listed in detail below in the general order of increasing strength and stability:

| Geological Unit | Rock Type | Structure | Strength & Cementation | Defects | Comments/ Stability Characteristics |
|---|---|---|---|---|---|
| Hukerenui Mudstone (Mangakahia Complex) | Mudstone/ siltstone, cream, green, red/brown, light to dark grey, high clay content | Highly sheared and deformed, lensoid to rhombic tightly packed rock fragments, with fragments or blocks of other lithologies, locally massive but generally highly heterogenous | Extremely weak to weak, weakly to moderately cemented | Typically not continuous defects within rockmass, rock fragments polished & slickensided, thinly laminated to thin silt/clay, limonite, manganese infill | Generally least stable lithology, also known as melange (mixed) zone, transition zone grades into less weathered rockmass, serpentinite pods along fault traces, shear planes typically develop at top of transition zone |
| Whangai Formation (Mangakahia Complex) | Siltstone, minor calcareous siltstone & muddy limestone, light brown, light to dark grey | Highly shattered, locally sheared and contorted, tightly packed rock fragments, often appears as a chaotic agglomeration but also contains continuous shears/faults | Extremely weak to weak, weakly to strongly cemented | Rock fragments polished & slickensided, thinly laminated to thin silt/clay, limonite, manganese, calcite infill, shear zones/faults with polished & slickensided silt/clay infill | Low permeability residual soils and highly fractured, sheared and permeable transition zone allows high pore water pressures to develop and planar creep type slope failures to occur |

| | | | | | |
|---|---|---|---|---|---|
| Mahurangi Limestone (Motatau Complex) | Calcareous siltstone & sandstone, argillaceo-us limestone, muddy limestone, light grey, white | Highly shattered with tightly interlocking rock fragments | Extremely weak to weak, weakly to strongly cemented | Undulating slickensided to smooth joints, minor dark grey silt and clay infilling | Transition between residual soils (often highly plastic clay) and bedrock is typically abrupt with slope failure often occurring on this plane |
| Allochthonous Akarana Supergroup/ Waitemata Group | Grey silty fine sandstone & minor calcareous fine sandstone, alternating sandstone & siltstone flysch | Massive to poorly bedded, and locally sub-parallel thinly interbedded with continuous shears/faults | Extremely weak to weak, weakly to moderately cemented | Widely spaced steeply inclined shear zones/faults with polished & slickensided silt/clay infilling and sandstone/siltstone rip-up clasts | Septarian concretionary boulders up to approximately 2 to 3 metres diameter Considered to be more stable than other allochthonous units |

## 3. SLOPE STRATIGRAPHY

Slope stratigraphy is typically divided into three general groups: residual soils, transition zone and bedrock as shown in Figure 2. Each of these units have distinctive physical and engineering properties that influence their behaviour.

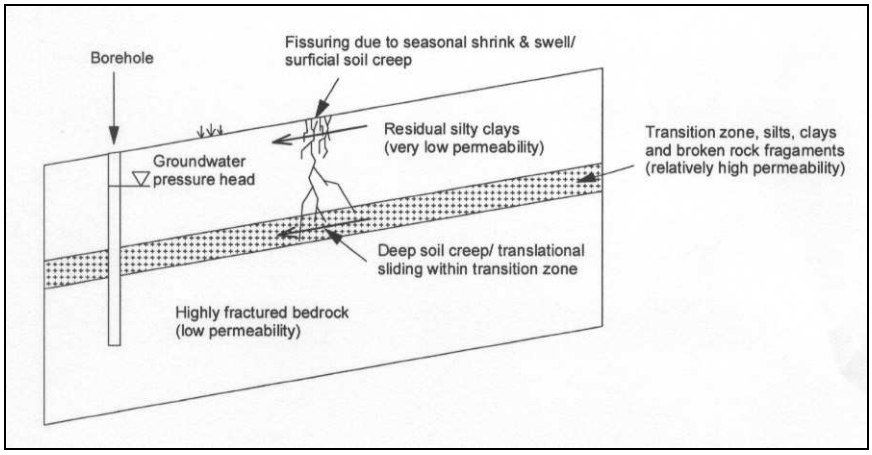


Figure 2. Schematic of a typical slope profile within the Northland Allochthon.

Residual soils typically comprise clayey silts and silty clays with high smectite (swelling) clay contents and low permeabilities. These soils are often highly fissured due to seasonal shrinkage and swelling within the upper metre of the soil profile, often with a very thin coating of soft, highly plastic clay with polished/ slickensided surfaces. Fissuring can also extend to depths greater than would normally be attributed to this surficial effect. In this case mass movement is considered to be the cause. Soils are often sensitive to disturbance with significant reductions in shear strength.

Transition zone soils between residual soils above and shattered bedrock below are most commonly responsible for land instability. For a pervasively sheared transition zone the defects typically have polished surfaces with remoulded clay infill that provides the dominant control on shear strength. In some lithologies (especially the Hukerenui Mudstone) it is rare to find a slope that has not previously failed forming a planar to undulating, slickensided shear plane with thinly laminated to thin very soft, highly plastic smectic rich clay infilling.

The transition zone between the residual soils and underlying bedrock may be gradational or there may be an abrupt boundary. A thin transition zone may be especially common where slope instability has occurred although relatively thick layers of intensely sheared soils/ highly weathered rock mass can also be found.

The underlying bedrock typically comprises a highly sheared mass of tightly interlocking fragments often with high clay contents. Weathering decreases with depth although the rockmass strength can be affected if exposed to the atmosphere. When confined the rockmass typically displays high penetrative strength properties although shear strengths may be low. Sampling difficulties make this difficult to determine.

# 4. GEOLOGICAL MODELLING

## 4.1 General

Developing a comprehensive geological model involves integration of a number of important elements. The typical methodology begins with confirming the geological setting, followed by mapping the large scale features (regional geomorphology) and investigating the small scale features (outcrops or exposures). Subsoil investigation is then undertaken including groundwater monitoring and sampling. When developing a geological model there will always be areas of uncertainty and incomplete information so experience can also play a big part. Often two minds are better than one in solving problems and putting together the pieces. Geological models will change and evolve as investigation work progresses and also during construction as excavations expose more subsurface features.

## 4.2 Geological Setting

The first step in any site investigation should be to establish the geological setting, the most important factors are:

- review of published technical papers and theses
- assess the depositional and tectonic history
- estimate the current tectonic stress regime (although not an easy task to accomplish)
- identify structural geological features
- inspection of any outcrops and observing lithology changes in particular
- confirm the underlying geology and identify the likely lithologies to be encountered

## 4.3 Geomorphology

Slopes often have a hummocky appearance or benched profiles which develop from successive land movements and are often easy to recognise by the presence of long and undulating slopes and relatively subdued topography. Usual landslide features like slip scarps and debris mounds are not common except on the steeper slopes. Reeds normally found growing in swampy areas are widespread on the slopes due to the low permeability of residual soils.

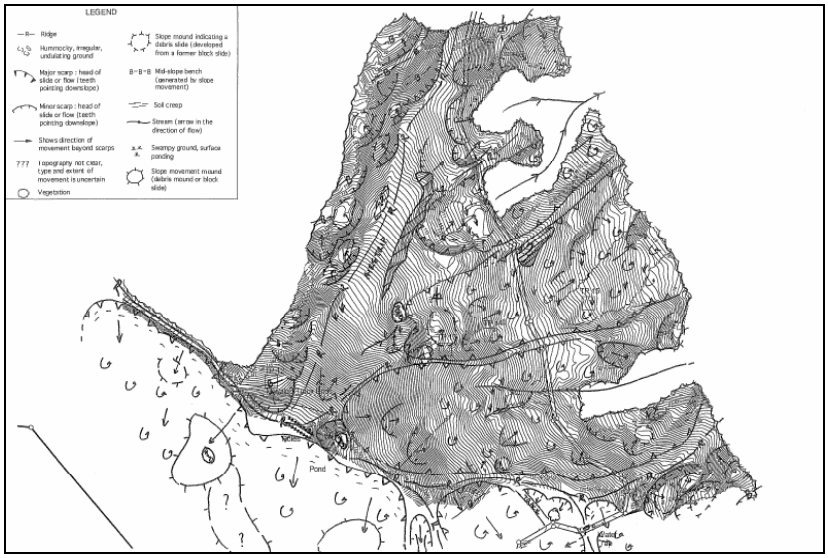


Figure 3. Geomorphology Plan of a site in Northland Allochthon terrain showing typical large translational landslides.

Initially the large scale (regional) geomorphic features of a site should be analysed during a desk study using aerial photographs, published maps and reviewing other investigation work in the area. Following this a detailed walk over study will be required involving mapping of the site noting all relevant small-scale (local) surface features. Any identifiable structural controls such as local and regional faults should be mapped, although this is not always possible as continuous faults are often difficult to identify in the shattered and deformed rock mass.

Geomorphological characterisation can then be undertaken using diagrams and plans to display and analyse typical slope instability features in these materials as shown in Figure 3. This is one of the most important aspects of geological modelling in Northland Allochthon terrain.

4.4 Slope Failure Mechanisms

Land instability types that are commonly encountered include surficial seasonal soil creep to relatively deep seated slow, progressive movement of the entire soil profile with shearing occurring within the transition zone layer as planar/ translational land sliding. The transition zone may be fractured and broken but relatively tight where only minor movement has occurred. However in areas of major instability this layer may be highly deformed or contain continuous planar to undulating, slickensided shear planes with very thin soft and wet clay infilling. Laboratory testing and back analysis of failed slopes has shown Hukerenui Mudstone residual friction angles to be generally between 10º and 13ºand as low as 8º (Winkler, 2003).

Slopes as gentle as 8 to 10 degrees have been found to be unstable within the Hukerenui Mudstone with natural slope angles related to the underlying lithology. Slopes underlain by calcareous or siliceous rocks of the Whangai Formation may have gradients in excess of 20º.

Slope failures are almost inevitable when cuts are formed at angles steeper than those found in the natural state, especially if transition zone materials are daylighting or the mass balance of the slope is compromised by removal of toe support.

Deep-seated landsliding involving displacement within the rock mass is rare and difficult to recognise. Large cuts on earthworks sites have exposed continuous shear zones within the rock mass that may be attributed to landsliding. In many cases these shears may have tectonic origins.

4.5 Subsoil Investigation

Detailed material descriptions will typically involve on site logging of hand auger boreholes, trial pits or machine boreholes. There are limitations to each of these methods that should be considered prior to undertaking any subsoil investigation, but this topic is not covered here. Undisturbed and disturbed soil samples should be obtained at this time for laboratory analysis.

Inclinometer monitoring is useful to identify and analyse slope movement especially in areas of complex landsliding where the depth of movement is not easily determined or where multiple planes of failure are possible.

4.6 Defects Analysis

Bedrock outcrops are rare even in stream beds, as there is often significant accumulation of alluvial sediments and colluvium from the slopes above. Road cuts and earthworked areas are the only other places where analysis of defects is possible without the excavation of large trial pits.

The determination of rock mass strength (RMS) analysis using the Hoek-Brown failure criterion may be undertaken, but caution must be emphasised. This is due to significant uncertainty and anisotropy in rock mass properties making selection of intact rock element $\sigma_{ci}$, the material constant $m_i$ and the Geological Strength Index (GSI) very difficult. The shear strength of defects is not controlled by substance strength but rather by the properties of the smectite rich clay infilling.

Stereonet analyses are important where continuous defects sets, shear zones and faults are found, often within Whangai Formation bedrock and Allochthonous Akarana Supergroup.

4.7 Laboratory Testing

Commonly performed laboratory testing within Northland Allochthon soils and rock mass include:

- atterberg limits tests to determine expansivity of residual soils (allochthon soils are typically expansive)
- drained triaxial testing of undisturbed soil samples to determine peak and fully softened
- unconfined compressive strength (UCS) testing, but there are sampling difficulties
- triaxial testing of rock has proven to be problematic due to its highly shattered nature and sampling difficulties
- ring shear testing of remoulded soil samples on clay seam/ shear plane samples
- soil permeability and rock porosity
- slake durability testing
- soil density testing, usually involved with compaction control during earthworks operations.

4.8 Groundwater and Hydrology

Determining groundwater behaviour and developing groundwater models is one of the most important aspects in geological modelling. Within the Northland Allochthon the schematic slope profile shown in figure 2 is often relevant as a groundwater model. The highly fractured and crushed transition zone between the clay rich residual soils above and tightly packed fragments of the bedrock below acts as a confined aquifer.

When drilled through or exposed in an excavation only minor seepages are usually found although when piezometers are installed, the pressure head within this unit is at a much higher level than where the seepage is encountered.

Porosity variations in juxtaposed lithological units may be related to deep-seated instability within the rock mass. A sandstone unit 'sandwiched' between low permeability mudstones or melange units can develop high pore water pressures instigating mass movement (Winkler, 2003).

## 5. CONCLUSIONS

Slopes underlain by Northland Allochthon lithologies within the greater Silverdale area require detailed geological modelling to ensure that suitable and cost effective land stabilisation measures are implemented to protect buildings and infrastructure. There are a number of critical aspects involved in developing a comprehensive geological model and the geotechnical engineer/ engineering geologist should have a thorough understanding these prior to undertaking any design work.

There will always be uncertainties in any geological model but it is important to strive and fill in any 'gaps' wherever possible to reduce the risk of slope failure during or after land, building or infrastructure development works.

## 6. ACKNOWLEDGMENTS

I would like to acknowledge Universal Homes Limited and Hospice Hibiscus Coast Trust for use of site investigation information and Foundation Engineering Consultants Limited for use of their equipment and expertise.

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# AN OVERVIEW OF MULTI-STOREY DEVELOPMENT AND MINE SUBSIDENCE RISK IN NEWCASTLE, AUSTRALIA

**S.J. Mackenzie**
*Coffey Geotechnics Pty Ltd*
*Newcastle, NSW*

## ABSTRACT

Much of Newcastle is undermined by abandoned coal mine workings dating from the early 1800's to early 1900's. As residential and commercial building heights in the Newcastle central business district increase, the issue of mine subsidence and the associated risk needs to be reconsidered by engineers, government authorities and the community in general. A critical aspect is the reliability or certainty with which mine subsidence risk is known. Also critical, but more difficult to formulate, is what level of risk is acceptable. Mine grouting methods can be used to remove or reduce risk, but is the cost justified? This paper aims to introduce the mine subsidence issues facing Newcastle to a wider audience. It is hoped that an open discussion on the topic of acceptable risk will be instigated and that comparisons to other low probability high consequence risks will eventuate.

## 1. INTRODUCTION

Newcastle is the sixth most populous city in Australia and the largest coal export centre in the world. In 2005, approximately 78 million tonnes of coal was exported from the Hunter Valley, typically by open cut or long-wall operations situated in rural or bush land settings. Newcastle is undermined by abandoned mines in up to three coal seams between approximately 10m and 80m depth. Residents of Newcastle are generally unaware of the risk of subsidence, despite a history of events resulting in parts of Newcastle central business district (CBD) subsiding by as much as 0.8m.

Figure 1: Approximate Extent of Undermining

The risk of mine subsidence continues to affect development in Newcastle in a very real but poorly appreciated way. Land use zoning and building heights nominated in planning instruments do not generally reflect mine subsidence risk and often subsidence constraints are not realised until after development concept stage.

Remediation by way of grouting can reduce or remove the risk of subsidence and allow development on almost any site, albeit at considerable and often unexpected cost to the developer. As a subsidence event hasn't occurred in the Newcastle CBD for 98 years and the coal pillars at risk of failure have factors of safety typically greater than 2, the need for remediation is often a source of contention. At the heart of this issue are the following:

- What is the risk of mine subsidence and how reliable are our estimates?
- Who is liable if subsidence were to occur and what risk are they willing to accept?
- How much does remediation cost and who should pay?

## 2. BRIEF HISTORY OF MINING IN NEWCASTLE

In Australia, coal mining by Europeans began in 1797 in the cliffs surrounding Newcastle. Mining came under government control in 1804 but methods and equipment remained basic (Comerford, 1997). In 1828, Governor Darling granted the Australian Agricultural Company 2000 acres of land which had been reserved for future expansion of Newcastle and the company increased coal output to 41000 tonnes for the year 1847. Other companies were soon mining in and around Newcastle and in 1900, 4 million tonnes of coal was extracted (Turner, 1982). The majority of the coal was won from the 6m to 8m thick Borehole Seam that lies at a depth of 70m to 80m. By 1916, after approximately 119 years, the economically mined coal beneath Newcastle had been exhausted and mining operations ceased.

Figure 2 : Hand Mining of Borehole Seam

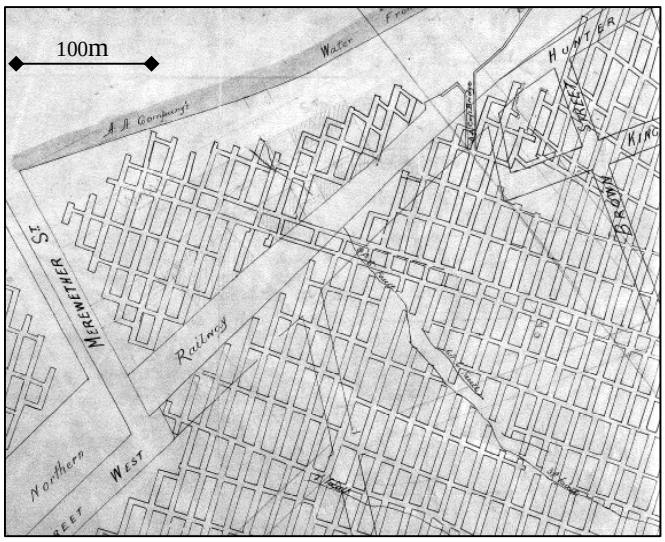



Figure 3 : Typical Bord and Pillar Workings of Borehole Seam

Trough subsidence events or 'creeps' occurred periodically during mining and most frequently in the period from 1890 to 1908. The cause of such subsidence is the failure or crushing of a number of adjacent coal pillars. As shown in Figure 4, typical features of this type of subsidence are a bowl shape with a compression zone in the middle and a tension zone around the perimeter.

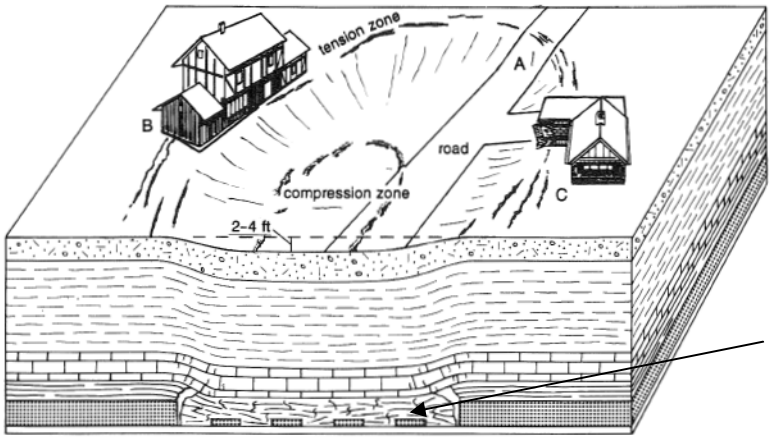
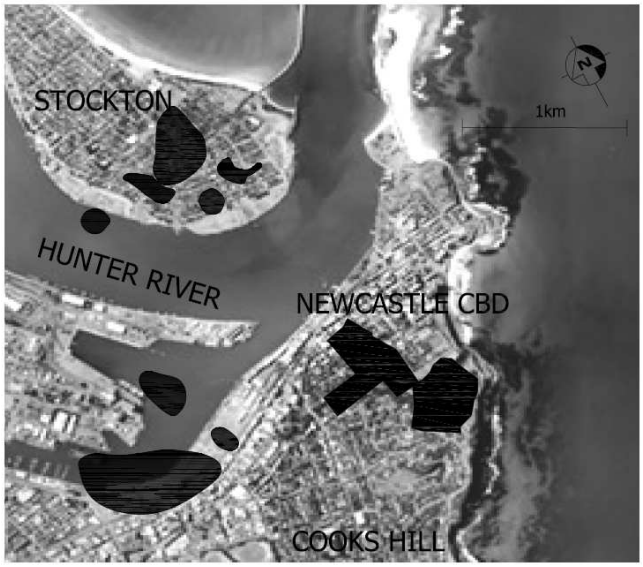



Figure 4 :

Typical Trough Subsidence Features. (Bauer et.al., 1993).

Approximate locations of recorded creeps are shown in Figure 5. The most recent of these events occurred beneath the Newcastle CBD and surrounding hill in three stages from 1906 to 1908 as summarized in To (1988). Subsidence of up to 0.8m was reported and significant damage to buildings and underground utilities resulted, Subsidence of up to approximately 1.2m has been reported by Atkins (1902) for other creeps beneath Newcastle Harbor.




The extent of mining shown is indicative only.

Figure 5 : Approximate Locations of Creeps

## 3. ROLE OF THE MINE SUBSIDENCE BOARD

Underground coal mining typically results in surface subsidence. Mining companies must submit a Subsidence Management Plan to the NSW Department of Primary Industries. If damage occurs, the Mine Subsidence Compensation Act 1961 requires the Mine Subsidence Board (MSB) to compensate the land owner, usually by way of repairing the damage. Money is raised by the NSW State Government through a levy on each tonne of coal mined; currently approximately 8c per tonne. The current price of coal is approximately $65 AUD per tonne. To limit its risk exposure, the MSB has the authority to reject development applications or to place restrictions on proposed development. This authority only extends to proclaimed mine subsidence districts. However, the MSB must compensate for damage, regardless of whether the damage is within a proclaimed district or not.

In broad terms, the risk of subsidence is informally defined as being the product of the likelihood of a subsidence event occurring and the consequences of that event. From the MSB's perspective, the relevant consequence is measured by the expected cost of repair. The taller the building or greater the development density; the greater the risk of subsidence, regardless of the likelihood. At present, buildings in excess of three to five stories are not expected to be approved by the MSB in the Newcastle CBD unless the risk of subsidence can be reduced to acceptable levels. This reduction can be achieved by any one or a combination of the following:

- Grouting of the mine workings to reduce likelihood of subsidence and/or magnitude;
- Reducing the proposed development size and therefore also the repair cost;
- Designing the proposed development to accommodate the predicted subsidence.

An alternative approach is to conduct more detailed investigations, research and analysis to give increased reliability to estimates of subsidence risk. Assuming that a better understanding will reveal current risk estimates to be conservative, higher and more reliable estimates of pillar factor of safety will result.

Further discussion on the history and function of the MSB is presented in Wilson (1973).

## 4. PILLAR STABILITY AND PROBABILITY OF FAILURE

The likelihood and magnitude of subsidence is a function of the depth of mining, thickness of coal extracted, geological conditions and method of mining. Assessment of subsidence likelihood and magnitude is typically made by geotechnical consultants drawing on information from mining records, borehole data as well as a number of empirical and analytical methods. Some examples specific to Newcastle are described in Pells et.al. (1988), Fennell (1997) and Hawkins (2005).

In order to quantitatively assess the stability of a coal pillar, a factor of safety is determined. This is the ratio of pillar strength to the load or stress on that pillar. There are many methods in practice throughout the world to estimate pillar strength. The University of New South Wales (UNSW) Pillar Design Methodology presented in Galvin et.al. (1998) is commonly used in Australia. This approach is based on semi-empirical relationships derived from a data set of 63 failed and 114 un-failed pillars from South African and Australian mines and was developed to aid pillar design rather than the assessment of old abandoned pillars. This is reflected in the distribution of pillar ages with 96% of pillars being less than 18 years old at the time of their inclusion into the data set. Furthermore, the 4% of pillars that are older than 18 years are all un-failed. In comparison, the pillars beneath Newcastle CBD are now between 90 to 200 years old. As the data set does not adequately represent pillars older than approximately 50 years, the use of the UNSW Pillar Design Methodology to assess the pillars beneath Newcastle should be done with caution.

The UNSW Pillar Design Methodology also assumes that: mine roof and floor conditions are good, loading on pillars is relatively uniform, and other conditions such as geological faults or full pillar yield (crushing) do not exist. The affects of inundation of the workings with water are not incorporated; however, some assessment of this load reduction has been made using numerical modelling as described in Mackenzie and Clark (2005).

Pillar factor of safety is usually expressed as a probability of failure according to the UNSW power law model of rectangular pillar strength as shown in Figure 6. As previously discussed, the method does not adequately incorporate uncertainties related to pillars older than approximately 50 years such as: pillar dimensions, pillar deterioration, extraction thickness, and the affect of water inundation on pillar load and strength. The assessment of such uncertainties lends itself to reliability analysis as summarised in Christian (2004). Both Stewart and Love (2005), and Cole et.al. (1994) give examples of the application of basic reliability theory to abandoned coal mines in urban environments.

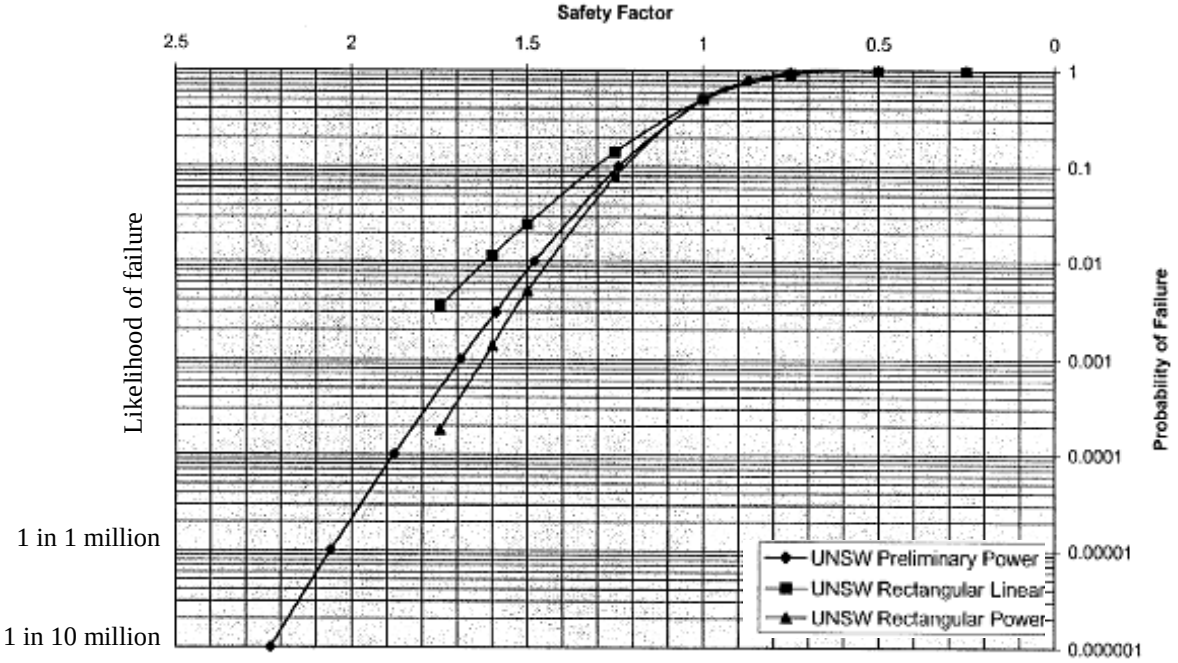



Figure 6 : Factor of Safety versus Probability of Failure. (Galvin et.al., 1998)

In the Newcastle CBD, a typical estimate of the likelihood of wide spread subsidence resulting in approximately 0.8m surface subsidence is 1 in 1 million to 1 in 10 million. As noted in Stewart and Love (2005), this probability of failure is over an undefined time period and an assumed period of 10 years seems appropriate based on the UNSW data set. The annual probability of failure is therefore the probability obtained from Figure 6 divided by 10. This annualizing of probability is necessary to allow it to be compared to other risks as presented in the following section.

## 5. ACCEPTABLE RISK

In engineering terms, risk is usually defined as the product of probability of failure and consequence with the latter often being expressed as monetary loss or number of lives lost. What constitutes acceptable risk varies between individuals and is dependent on many factors such as personal experience, awareness, risk reduction cost and the benefits of accepting a particular risk. A detailed discussion of what is considered acceptable risk for various circumstances and organisations is presented in Baecher and Christian (2003). Two charts published in this text are reproduced below. Regions representing approximate mine subsidence risk in Newcastle have been added.

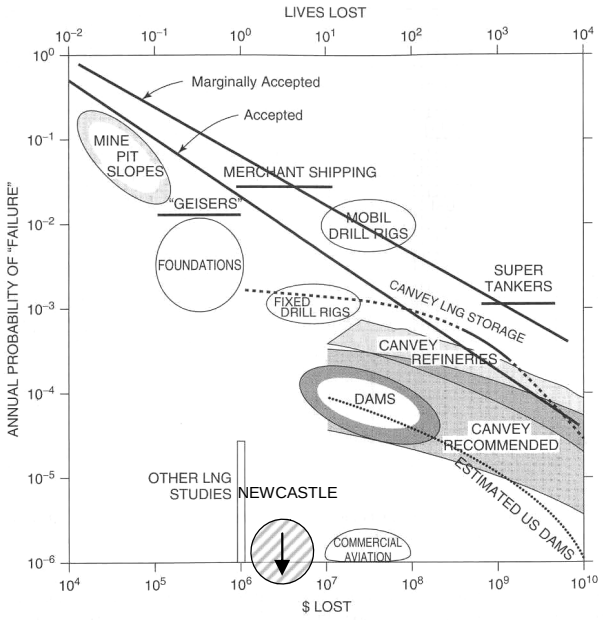



Figure 7 : F-N chart showing average annual risks (Baecher 1982b).

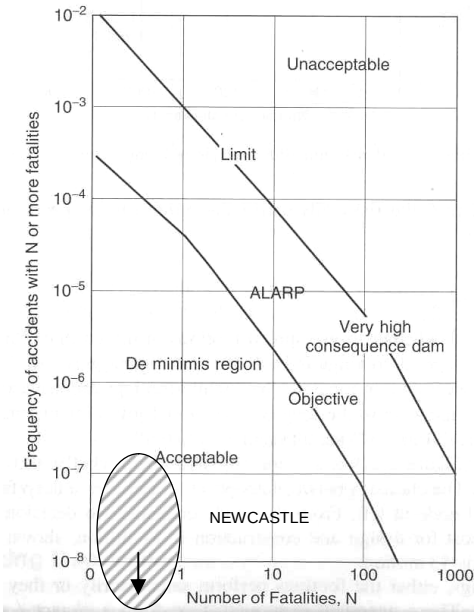



Figure 8 : Risk guidelines from ANCOLD (1994)

For a three storey building, a 1 in 1 million likelihood of 0.8m subsidence would normally be acceptable to the MSB and current Mine Subsidence Development Guidelines for Newcastle CBD reflect this. In comparison, a 1 in 100 million likelihood of 0.8m subsidence would not be acceptable for a building of 15 to 20 stories. The cost of replacement of such a building would be in the order of $30 million to $50 million, whereas the total assets of the MSB are approximately $42 million with approximately $8.5 million per year being expended on claims and operating costs (MSB 2005). Hence, a single claim from a ten storey building could exceed the available funds.

The monetary cost of building repairs is very significant, however loss of human life is considered extremely unlikely as the rate of subsidence and thus damage is expected to be sufficiently slow to provide adequate warning.

The primary reasons for the MSB's unwillingness to accept these risks are a lack of confidence in the assessed likelihood of failure (stemming from uncertainties in pillar dimensions and the affects of water and age), limited funds and an inability to reinsure and thus offset their risk exposure.

## 6. RISK REDUCTION

The risk of subsidence can be reduced or removed by grouting of the mine workings. Typically flyash or sand based grout is pumped into the workings via boreholes. Grouting need not fill the entire mine void and "Strategic Grouting" can sometimes be used to place grout where it will be most effective in either increasing critical pillar stability or reducing subsidence, or both. Grouting of the shallower (20m) Yard Seam has been undertaken in the Newcastle CBD for at least four developments. However, no grouting of the Borehole Seam beneath the Newcastle CBD has been undertaken to date with the exception of a trial at Lee Wharf, Newcastle (Coffey Geosciences 2004). To the authors knowledge, grouting of flooded mine workings at greater than 50m depth in an urban environment has not been undertaken anywhere in the world. The first such grouting project is scheduled to commence in November 2006 at Lee Wharf where approximately 3500 tonnes of flyash based grout will be pumped into flooded Borehole Seam workings at approximately 70m depth.

The estimated cost of grouting to allow construction of a multi-storey building is estimated to be between $500,000 to $1,000,000 with the greatest cost items being raw grout material and borehole drilling. The author is aware of nine proposed buildings in the Newcastle CBD that currently require such grouting to reduce risk to an acceptable level.

## 7. CONCLUSIONS

The risk of mine subsidence in the Newcastle CBD is generally believed to be very low. However, significant uncertainties in the assessment method remain. Grouting as a method of risk reduction exists, but is costly and may not be warranted. Further work is required to formally establish a risk assessment framework for mine subsidence assessments and to develop guidelines for acceptable risk. This process should be done in consultation with local and state government as well as representatives from the general community and property developers. The findings of Newcastle-wide risk assessments should be used to target further research and to refine current development guidelines. If grouting is required, an integrated scheme will be more efficient than the current ad hoc approach of grouting individual sites as required.

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# AVOIDING EMBANKMENT FAILURE ON ADELAIDE HILLS ROADS

**Nelson Mendoza**
*Graduate Engineer, Department for Transport, Energy and Infrastructure*

## ABSTRACT

The Department for Transport, Energy and Infrastructure has concerns with shallow slip failures in the embankments of roads in the Adelaide Hills as these failures, and their repair, create safety issues and cause delays to traffic.

These shallow slip failures can be attributed to a combination of poor drainage and the geometry of the road embankment. Analysis has been performed to determine the geometry of embankments that would be more susceptible to failure under poor drainage conditions. It was found that the batter angle of the embankment was the most significant factor and that embankments with batter angles greater than 35° degrees, above the horizontal were likely to undergo shallow slip failure in saturated conditions. This finding was supported by measurements of embankments that had suffered shallow slip failure.

Aerial photography was used to produce a digital terrain map of a section of Lower North East Road at Anstey Hill. From this the batter angle of the embankment has been determined along the length of the road and hence the risks of shallow slip failure. A map has been produced showing how risk of embankment failure varies with location. This map also shows variation of the natural hillside slope with location, as the cost of repair of the slip increases rapidly with hillside slope. This information can be used to prioritise drainage improvement or maintenance on Adelaide Hills roads.

## 1 INTRODUCTION

The Department for Transport, Energy and Infrastructure (DETI) has concerns with shallow slip road embankment failures in the Adelaide Hills. An example of such failure is shown in Photograph 1 and illustrated in Figure 1. These shallow slip failures can be attributed to a combination of blocked culverts and/or drains and the geometry of the road embankment, in particular, batter angle and fill height.

**Photograph 1 – Typical shallow slip failure**

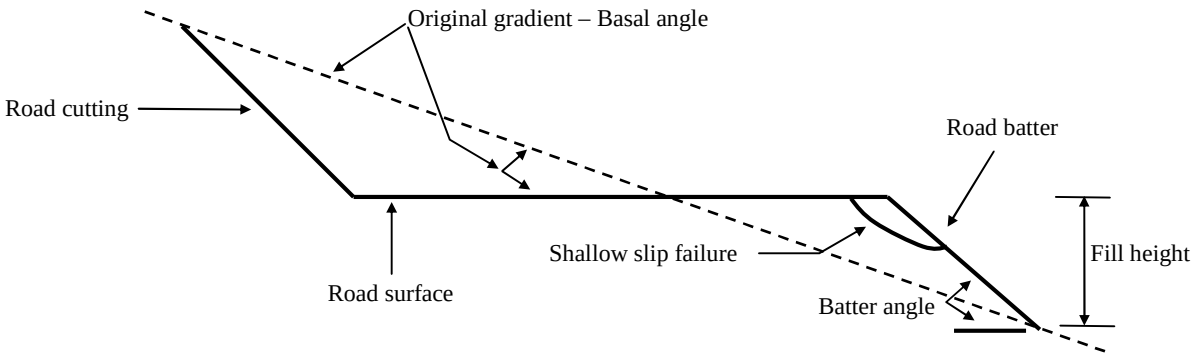


**Figure 1 – Typical shallow slip failure**

DETI's current understanding is that many of the shallow slip failures in the batters of road embankments occur when water runs over the road and down the batter. The soil on the batter becomes saturated, loses strength and slips.

One aim of this project was to determine the critical road embankment geometry, that is, values of batter angle, basal angle, fill height, etc, which, if exceeded, would result in a shallow slip occurring when water flowed over the road and saturated the soil on the batter. Another aim was to determine an efficient method for identifying those sections of road with this geometry in order to prioritise the maintenance of drainage infrastructure to minimise the risk of shallow slip failure.

## 2 IDENTIFYING CRITICAL ROAD EMBANKMENT GEOMETRY

During most winters there are one or two shallow slip failures in Adelaide Hills roads. A record of those failures has been kept over the past few years (Hood and Engelaar, 2005). Table 1 shows a summary of information about the geometry of these sites. It appears that no failed embankment had a batter angle less than 36 degrees, regardless of basal angle or fill height.

| Site Number | Batter Angle ($^o$) | Basal Angle ($^o$) | Fill Height (m) |
|---|---|---|---|
| 1 | 36 | 23 | 8 |
| 2 | 38 | 10 | 3.5 |
| 3 | 38 | 10 | 12 |
| 4 | 38 | 14 | 9 |
| 5 | 40 | 20 | 10 |
| 6 | 45 | 25 | 15 |
| 7 | 46 | 18 | 2 |
| 8 | 52 | 33 | 12 |
| 9 | 80 | 35 | 1.2 |

**Table 1 – Geometry of Failed Sites**

In addition to measuring the batter angle, basal angle and the fill height the following data was also recorded as it was deemed to be relevant to the investigation:

- dimensions of culvert and/or drain cross section, either diameter or breath and width,
- condition of culvert and/or drain (blocked, partially blocked, free),
- location of culvert and/or drain with respect to the shallow slip failure,
- road grade and width,
- presence of free water, and
- evidence of slope movement.

# 3 ANALYSIS

Slope stability modelling of road embankments with various geometries was performed to determine which aspects of geometry affected the factor of safety for shallow slip failure most strongly.

## 3.1 SOIL STRENGTH PARAMETERS

Most embankments for Adelaide Hills roads were constructed by a cut and fill process, the material cut from the hillside being used to build the road embankment. Site investigations and the SA Department of Mines and Energy Map - 1986 suggest that most of the soil found on hillsides in the Adelaide Hills areas is clayey quartz sand. This would overlie weathered rock. In view of this it is likely that the Adelaide Hills road embankments are largely comprised of granular material in the form of sand and weathered rock, with some clay fines. A friction angle of 35° degrees was adopted for this material (see, for example, *Figure 6-3-1 "Typical values of the shearing resistance of cohesionless soils"* from M Carter, "Geotechnical Engineering Handbook", Pentech Press)

The batter angle for the failed sites was greater than 35° degrees, which suggested that there was some cohesion in the fill material prior to failing. This would be expected in view of the clay content of the fill. A cohesion of 5kPa was adopted for dry fill. It was further assumed that the cohesion would become negligible on saturation of the fill.

The cohesion for the natural soil was assumed to be 10kPa. A cohesion of 100kPa was adopted for the weathered rock. These soil strength parameters are summarised in Table 2 below.

| Layer | Cohesion (kPa) | Friction Angle ($^o$) |
|---|---|---|
| Fill | 5 | 35 |
| Soil | 10 | 35 |
| Bedrock | 100 | 35 |

**Table 2 - Soil strength parameters used in XSLOPE model**

The software program XSLOPE (Balaam, 1994) was used for the stability modelling. Figure 2 shows a typical XSLOPE model used in this analysis. The three main layers, fill, natural soil and bedrock are evident.

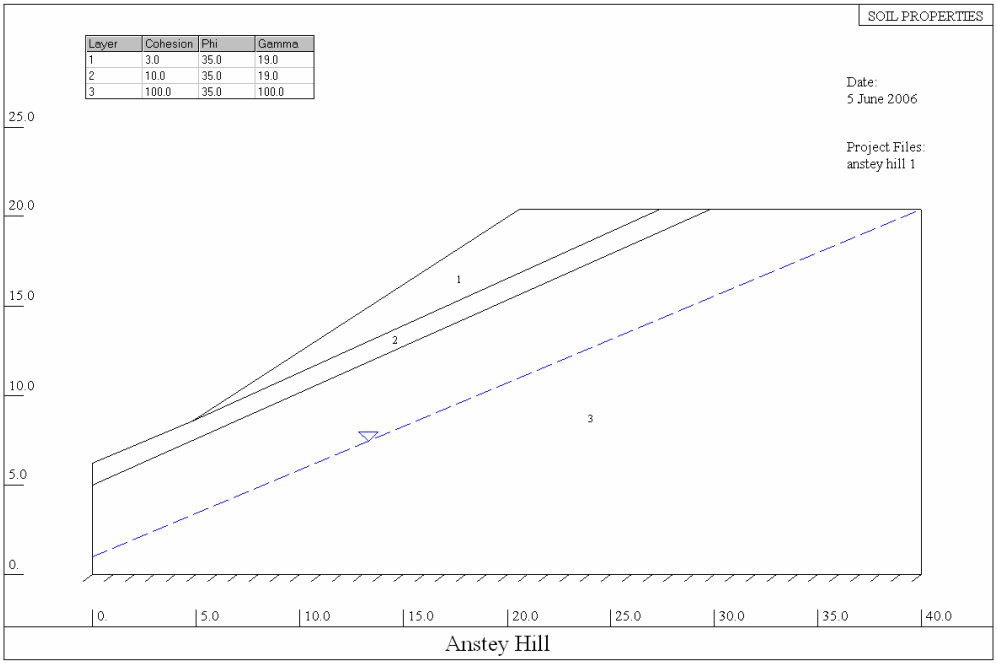


**Figure 2 – XSLOPE model of road embankment**

### 3.2 FACTOR OF SAFETY – RECORDED FAILED SITES

XSLOPE software was used to determine the factor of safety of each recorded failed site under both dry (c=5kPa) and saturated (c=0kPa) conditions. The factors of safety determined in the analysis are summarised in Table 3 below.

| Site Number | Batter Angle (°) | Basal Angle (°) | Fill Height (m) | FoS (c=5 kPa) | FoS (c=0 kPa) |
|---|---|---|---|---|---|
| 1 | 36 | 23 | 8 | 1.69 | 0.99 |
| 2 | 38 | 10 | 3.5 | 2.05 | 0.95 |
| 3 | 38 | 10 | 12 | 1.32 | 0.94 |
| 4 | 38 | 14 | 9 | 1.45 | 0.94 |
| 5 | 40 | 20 | 10 | 1.37 | 0.88 |
| 6 | 45 | 25 | 15 | 1.07 | 0.74 |
| 7 | 46 | 18 | 2 | 2.38 | 0.75 |
| 8 | 52 | 33 | 12 | 0.96 | 0.6 |
| 9 | 80 | 35 | 1.2 | 1.67 | 0.66 |

**Table 3 – Factor of safety (failed sites)**

In all but one case the factor of safety for dry conditions for the sites exceeds unity. This supports the assumption of the fill having some cohesion in order to be able to stand with a batter angle greater than 35° degrees. The factor of safety is less than unity for all the failed sites when they are saturated (cohesion =0 kPa). The foregoing indicates the choice of soil strength parameters and XSLOPE modelling to be valid.

### 3.3 VARIATION OF FACTOR OF SAFETY WITH GEOMETRY

XSLOPE models were created with various combinations fill height, basal angle and batter angle to investigate their impact on the factor of safety under dry and saturated conditions.

#### 3.3.1 Effect of Batter Angle

Figure 3 below shows the variation of factor of safety with batter angle for embankments of varying height. The factor of safety was calculated for both dry (c=5kPa) and saturated (c=5kPa) conditions.

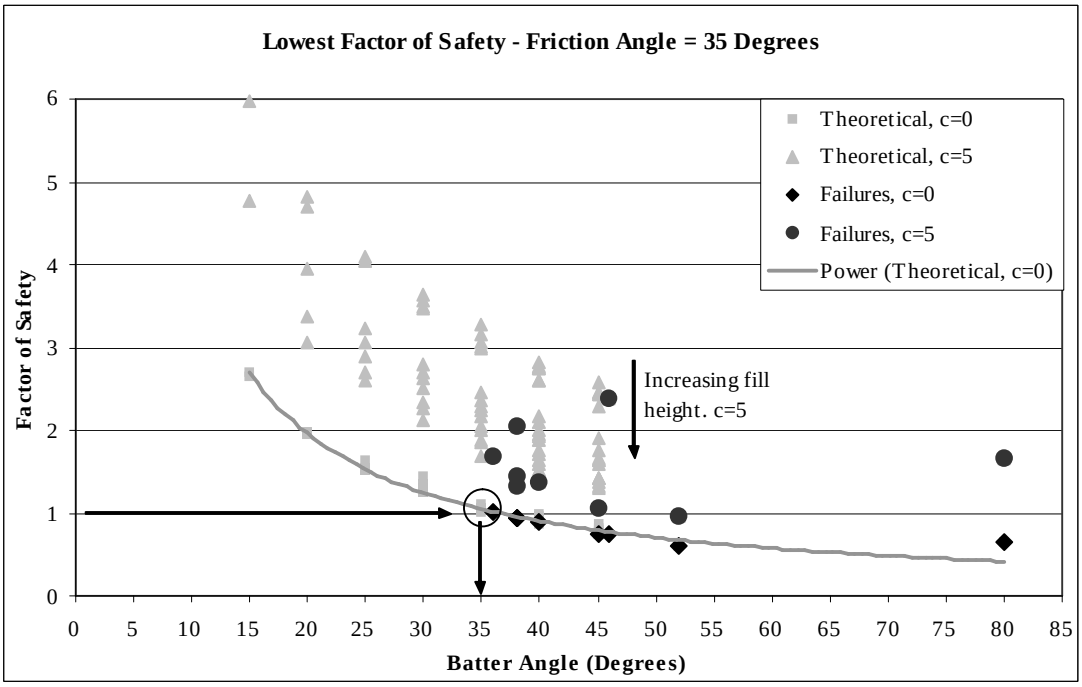


**Figure 3 – Factor of safety versus Batter angle (degrees)**

Figure 3 indicates that road embankments with batter angles greater than 35 degrees will be stable under dry conditions. However it was found that the critical angle at which the embankment batter becomes unstable (factor of safety less

than 1) under saturated conditions is 35° degrees, this result is supported by measurement of the geometry of the recorded failed sites.

### 3.3.2 Effect of Fill Height

The factors of safety for shallow slip failure obtained from XSLOPE for varying fill heights, batter angles greater than 35°, and a constant basal angle of 25° degrees, and c=0kPa, are summarized in Table 4. It can be seen that, in saturated conditions, the factor of safety does not change significantly as the fill height increases. This result is also supported by the recorded failures as they have varying fill heights ranging from 1.2 to 12 m with similar modelled factors of safety under saturated conditions.

| Factor of safety | | | | |
|---|---|---|---|---|
| **Basal Angle (°)** | **Batter Angle (°)** | | | **Fill Height (m)** |
| | **35** | **40** | **45** | |
| **25** | 1.07 | 0.88 | 0.78 | **2** |
| | 1.07 | 0.88 | 0.74 | **4** |
| | 1.07 | 0.88 | 0.74 | **6** |
| | 1.07 | 0.88 | 0.74 | **8** |

**Table 4 – Variation of Factor of safety with fill height**

It is concluded that the embankment of an Adelaide Hills road under saturated conditions is at significant risk of shallow slip failure if the batter angle exceeds 35 degrees.

## 4 IMPLEMENTATION OF FINDINGS USING AERIAL PHOTOGRAPHY AND DIGITAL TERRAIN MODELLING

The relationship between factor of safety and road batter angle under saturated conditions found by the XSLOPE modelling is shown in Table 5. Three categories of the factor of safety are used because it is customary in stability analysis to consider slopes with a factor of safety exceeding 1.3 to be stable and those with a factor of safety less than unity to be unstable.

| Factor of Safety (FoS) | Batter Angle |
|---|---|
| > 1.3 | < 30° |
| 1.0< FoS < 1.3 | 30° to 35° |
| < 1 | > 35° |

**Table 5 – Relationship of factor of safety with batter angle**

Aerial photography has been used to produce a Digital Terrain Model (DTM) of a section of Lower North East Road at Anstey Hill (Figure 4). From this digital map the batter angle has been determined at 10 metre intervals along the road and a colour-coded map of this road has been produced showing how risk of embankment failure varies with location. The colour-coding system for the road batter angle is described in Table 6 and the map produced is shown in Figure 5. In general the greater the basal angle (i.e. the hillside slope) the more costly the repair of a shallow slip failure. Hence it is useful to incorporate colour coding to indicate how the basal angle varies with location. The colour-coded system for the basal angle is described in Tables 6 and 7 and is also shown in Figure 5.

| Batter Angle | Colour |
|---|---|
| < 30° | White |
| 30° to 35° | Grey |
| > 35° | Black |

**Table 6 – Colour coding for Batter angle**

| Basal Angle | Colour |
|---|---|
| < 30° | White |
| 30° to 45° | Grey |
| > 45° | Black |

**Table 7– Colour coding for Basal angle**

The presence of large trees within the road reserve made it difficult to locate the toe of the batter at all locations. At such locations it was assumed for the purpose of this project that the bottom of the batter would range to a maximum of 5m away (horizontal distance) from the top of the batter. At such locations, onsite inspection is recommended to verify the bottom of the batter.

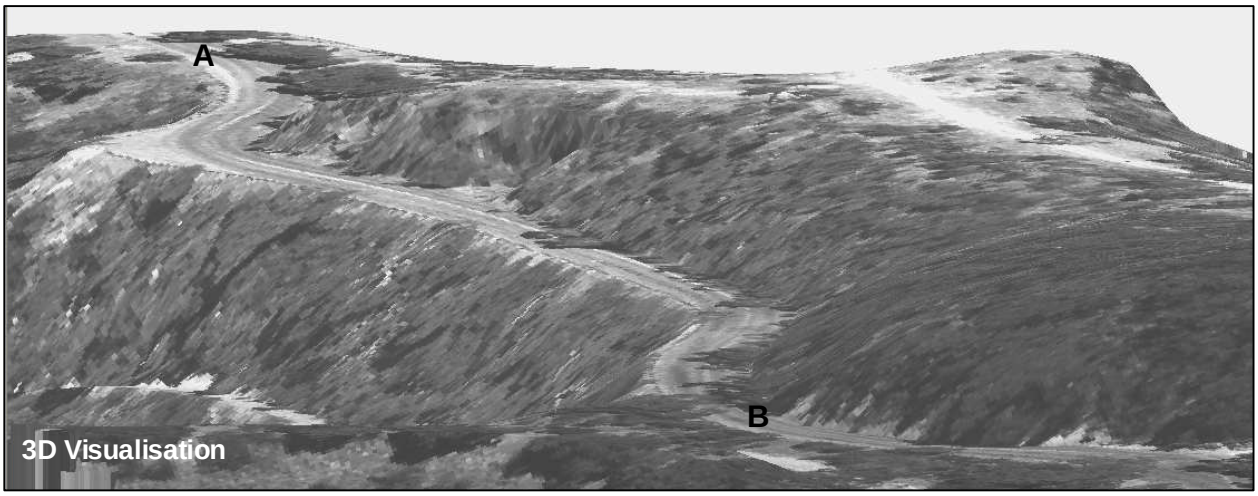


**Figure 4 – Section of Lower North-East Road at Anstey Hill**

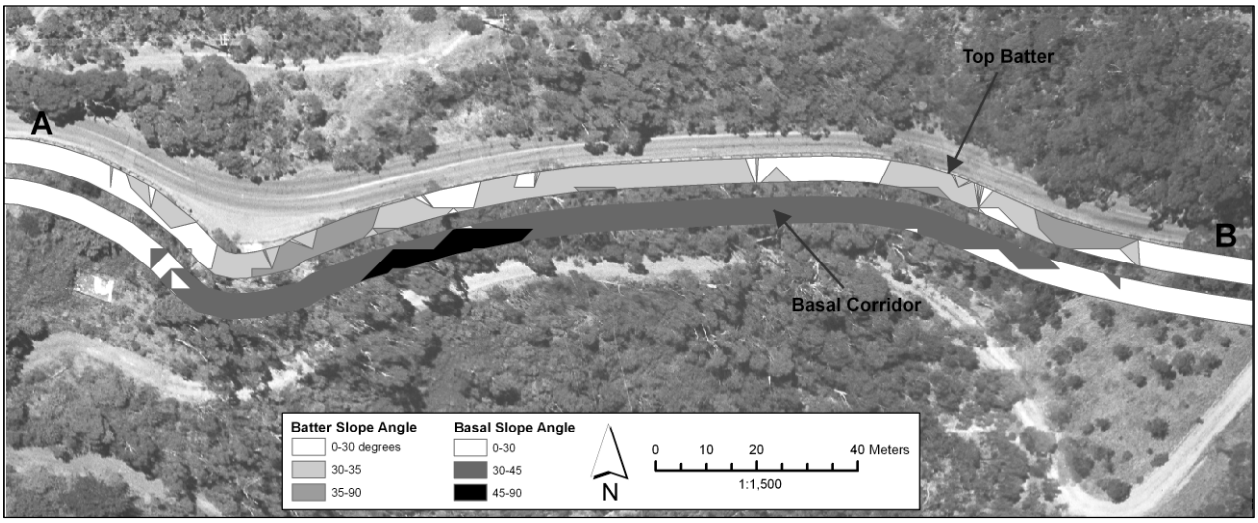


**Figure 5 – Implementation of findings using colour-coding**

## 5 REFERENCES


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- SA Department of Mines and Energy Map - 1986
- N Balaam 1994. XSLOPE, slope stability analysis using Bishop's simplified method, Centre for Geotechnical Research, University of Sydney.
- M Carter "Geotechnical Engineering Handbook" -*Typical values of the shearing resistance of cohesionless soils Figure 6-3-1*, Pentech Press


## 6 ACKNOWLEDGMENTS

I would like to thank Mr Richard Herraman, Geotechnical Group Manager with the Department for Transport, Energy and Infrastructure for giving me the opportunity to undertake such an interesting project. The experiences gained during the course of this project were invaluable, and have given me an excellent insight to slope stability. I would also like to thank Mr David Byrne, Mapping Specialist – Aerometrex, for providing me with the aerial photography and the digital terrain model, and Mr Gary Gordon and Mr Mark Whitehouse, GIS officers - Department for Transport, Energy, and Infrastructure for producing the colour-coded maps.

# AN ASSESSMENT OF THE LIQUEFACTION POTENTIAL OF LOOSE SANDS FOR THE PURPOSE OF PILE DESIGN

**Carly Moore**
*URS Australia Pty Ltd*
*Adelaide, South Australia*

## ABSTRACT

This paper discusses the design of piled footings for a transmission line installed on Torrens Island in South Australia. A geotechnical investigation was carried out by URS Australia Pty Ltd which revealed a soil profile consisting of loose Semaphore Sands, overlying silty sands of the St. Kilda Formation, overlying silty and shelly sands of the Glanville Formation, underlain by the sands and hard clay of the Hindmarsh Formation. Due to the presence of loose saturated sands in the upper layers, significant design loads and the requirement for the transmission line to be operational post-earthquake, it was considered necessary to assess the liquefaction potential of the surficial layers, as may result under earthquake conditions. The method used for assessment of the liquefaction potential has been adopted from equations presented by Youd et. al. (2001). A ratio of Cyclic Stress Ratio to Cyclic Resistance Ratio was calculated as a function of length along the pile, based on the geotechnical model derived from the site investigation, for a magnitude 6.0 earthquake. It was found that the highest potential for liquefaction was in the loose sands, as expected, but that the factor of safety against liquefaction was greater than unity for the length of the pile. The design geotechnical model was verified by load-displacement predictions which were consistent with the response of one working pile to a static uplift test.

## 1. INTRODUCTION

A geotechnical investigation and footing design was carried out for a new 66 kV transmission line, consisting of eight power poles, to be installed at the northern end of the existing Torrens Island Power Station in Adelaide, South Australia. A desktop study of published geological data and reports of previous investigations known to have been undertaken in the Torrens Island area was initially undertaken. An intrusive site investigation involved drilling a borehole at each power pole location, with Standard Penetration Tests (SPTs) (to AS1289.6.3.1-1993 "Methods of testing soils for engineering purposes") at 1.5 metre intervals in each borehole. Geotester readings were taken in all fine-grained soils to ascertain their undrained shear strength. The continuously recovered cores were logged using the Visual Tactile method (to AS1726-1993 "Geotechnical site investigations") and a geotechnical model was derived, from which the footings for the power poles were designed. A discussion of the design constraints, the geological profile and its implications on the design of the piled footing system is presented in this paper. The potential for liquefaction of surficial sands was a primary consideration in the design of the footing system and was assessed utilising a method presented by Youd et. al. (2001). The liquefaction assessment is discussed in Section 5 of this paper.

## 2. DESIGN CONSTRAINTS

The design loads for the power poles were mainly a result of wind loads and self weight of the conductor lines. The footings for all power poles were conservatively designed for the maximum design load, which would occur at a power pole located at an angle in the transmission line. The resultant lateral load at the top of a 20 metre high power pole translated to a high bending moment at the base of the tower.

The proposed transmission line was part of an upgrade to the feeder to the western suburbs of Adelaide. The transmission line was therefore considered as a 'general structure type III' – that is, essential to having post-earthquake function – in accordance with AS1170.4-1993 "Minimum design loads on structures, Part 4: Earthquake loads".

These design constraints were significant as the local geology meant that Torrens Island would be a problematic site, as discussed below.

## 3. GEOLOGICAL PROFILE

The Soil Association Map (SA Dept. Mines & Energy 1989) shows that Torrens Island consists of quaternary Estuarine deposits, with coastal sand dunes on the western shore of the island. From published data by Sheard & Bowman (1996), the geology of the Torrens Island area consists of the St Kilda formation, with Semaphore Sand in the upper layer (shown in Figure 1), overlying the Glanville formation, underlain by the Hindmarsh formation. The upper Semaphore Sand layer is recent dune sand, deposited by littoral drift of coastal sand northward along the shoreline. As the site had

been previously undeveloped, the Semaphore Sand was expected to be of very loose to loose density, in its unconsolidated form.

According to Belperio (1993), after the Holocene sea reached its present level approximately 6600 years ago, the LeFevre Peninsula progressively migrated northward, as shown in Figure 2. "Subtidal, intertidal and back-barrier sedimentation within this (Port Adelaide estuary) has generated a thick and complex but coherent facies architecture dominated by saturated and unconsolidated mud, sand and organic-rich sediments." (Belperio 1993). The Holocene strata occurs below Torrens Island as the St Kilda formation and was expected to contain sandy soils, some layers of soft clay and silt and potentially some highly organic bands, a result of the depositional environment of reed beds and marine grasses. The overall surficial soil at the investigation site was expected to be normally consolidated. The underlying Hindmarsh Clay is typically of very stiff to hard consistency and of high plasticity.

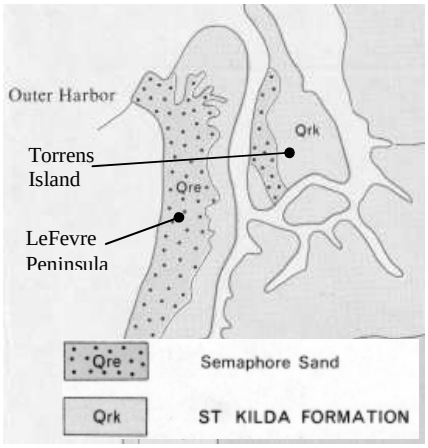


Figure 1: Geological Map of Torrens Island and vicinity (SA Department of Mines & Energy 1989)

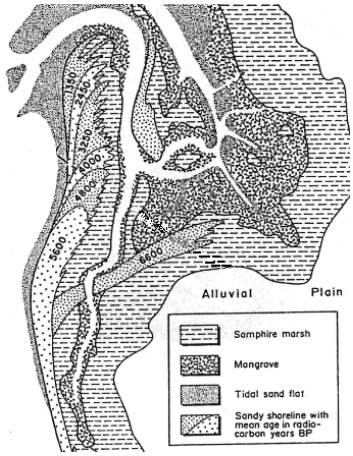


Figure 2: Formation of Outer Harbour and LeFevre Peninsula (Belperio 1993)

The site investigation undertaken involved the drilling and sampling to target depths of 15 metres using the method of continuous flight hollow stem augers with wireline retrieval of split spoon samplers. The presence of a high water table and loose surficial sands resulted in the selection of this drilling method, as it allows the borehole to be continuously supported during sampling and testing. Figure 3 depicts the site investigation being undertaken.

Figure 3: Site Investigation – Hollow Stem Drilling with Continuous Split Spoon Sampling and SPT

The site investigation indicated a relatively uniform soil profile consistent with that expected from the desktop study. Standing groundwater levels were recorded between 1.55 and 2.05 m depth. A summary of the soil profile that was encountered at the site is summarised in Table 1.

The loose nature of the normally consolidated surficial Semaphore Sand layer was notable during the site investigation, as mobilisation of the drill rig to a number of borehole locations proved difficult, with the rig becoming bogged at one location. Dynamic Cone Penetrometer (DCP) tests (to AS1289.6.3.2-1997 "Methods of testing soils for engineering purposes") were carried out near each borehole location to obtain a continuous record of the shear strength in the upper layers of the surficial soils. DCP test results were used to calculate the California Bearing Ratio (CBR) in this layer,

which was then used to prove that the bearing capacity of the surficial soil was insufficient to support shallow footings for the power poles. Estimated settlements for a 3.0 m x 3.0 m x 0.4 m footing were calculated to be in excess of 250 mm which was unacceptable.

SPT N values and Geotester readings indicated that the high plasticity, hard consistency clay of the Hindmarsh Formation would be a suitable founding layer for deep footings, especially as the desktop study indicated the thickness of the Hindmarsh Clay to be in the order of 80 metres.

| Formation | Depth to Layer (m) | Thickness of Layer (m) | Major Soil Type (USC Symbol) | SPT N Range | Comments |
|---|---|---|---|---|---|
| Fill (Gravelly Sand) | 0 | 0 – 0.8 | (SP) | - | Ignore Fill layer for design |
| Semaphore Sand | 0 – 0.8 | 1.6 – 3.6 | SP | 1 - 6 | Fine to medium sand, light brown and grey |
| St Kilda Formation | 1.9 – 3.75 | 5.6 – 7.25 | SM/ML | 0 - 7 | Silty sand over low plasticity silt, seagrass fibres, seaweed and organic matter, organic odour, shell matter |
| Glanville Formation | 8.9 – 9.7 | 0.55 – 1.6 | SM CL/SC/ML | 10 - 27 | Silty sand over low to medium plasticity clay (firm to stiff consistency), shelly zones, cemented sand and shells |
| Hindmarsh Formation | 9.9 – 11.3 | > 3.7 | CH/SP | 13 - 35 | Silty and sandy clay layers with sand layers over high plasticity clay (hard consistency), mottled grey and reddish brown |

Table 1: Geological units and their properties, based on the geotechnical investigation

Due to the relatively uniform soil profile, it was appropriate to derive a single geotechnical model of the soil profile to use for the design of the power pole footings. The model that was derived is presented in Table 2. The groundwater level has been taken to be at 1.5 m depth below surface level. Values for each of the soil properties were calculated at each SPT location and the design values assigned to each layer were selected from the range of values calculated.

| Stratum | Material | Depth to Stratum (m) | $\phi$ (deg.) | $c_u$ (kPa) | E (MPa) | $\nu$ |
|---|---|---|---|---|---|---|
| Semaphore Sand | Sand | 0 | 33 | 0 | 10 | 0.3 |
| St. Kilda Formation | Silty Sand | 3.5 | 30 | 0 | 5 | 0.3 |
| Glanville Formation | Silty Sand | 9 | 34 | 0 | 10 | 0.2 |
| | Clay | 11 | 0 | 50 | 20 | 0.2 |
| Hindmarsh Formation | Clay | 12.5 | 0 | 250 | 75 | 0.2 |

*$\phi$ = friction angle* *E = modulus of elasticity*
*$c_u$ = undrained cohesion* *$\nu$ = Poisson's ratio*

Table 2: Geotechnical Model for the Torrens Island Power Station Site

## 4. DESIGN OF THE PILED FOOTING SYSTEM

A 4-pile footing system was selected for this project due to the high design overturning moments determined for the structures and the geotechnical profile. Specifically, reinforced concrete bulbous-based driven cast in-situ piles were selected for their capacity to resist both tension and compression loads, which would accommodate the overturning load by 'push-pull' action. Installation of these piles is largely unaffected by the high groundwater and soft soils, as they are formed by driving temporary steel casing into the ground and casting the pile in-situ while withdrawing the casing.

Reinforced concrete piles were recommended over steel piles for their ability to resist an aggressive environment, such as that expected at Torrens Island. The ultimate design loads for the piles are presented in Table 3.

| Load Case | Applied Load (kN) | Bending Moment (kNm) |
|---|---|---|
| Axial Compression | $S^*_C = 1000$ (1) | $M^*_C = 75$ (2) |
| Axial Tension | $S^*_T = 527$ (1) | $M^*_T = 24$ (2) |
| Lateral Load | H* = 30 | M* = 122 (3) |

*(1) Axial loads take into account pile self-weight. (2) Bending moment due to 75 mm eccentricity, in accordance with AS2159-1995. (3) From program PULL2B (Sugar 1989) lateral analysis output*

Table 3: Ultimate Design Loads per Pile

The computer program TAPILE (Sugar 1989) was used to predict the axial load – displacement response of the piles to uplift and compression loading. A static tension test was carried out on one working pile to verify the adopted geotechnical parameters used in the pile design. The predicted and measured load-displacement responses are presented in Figure 4 below.

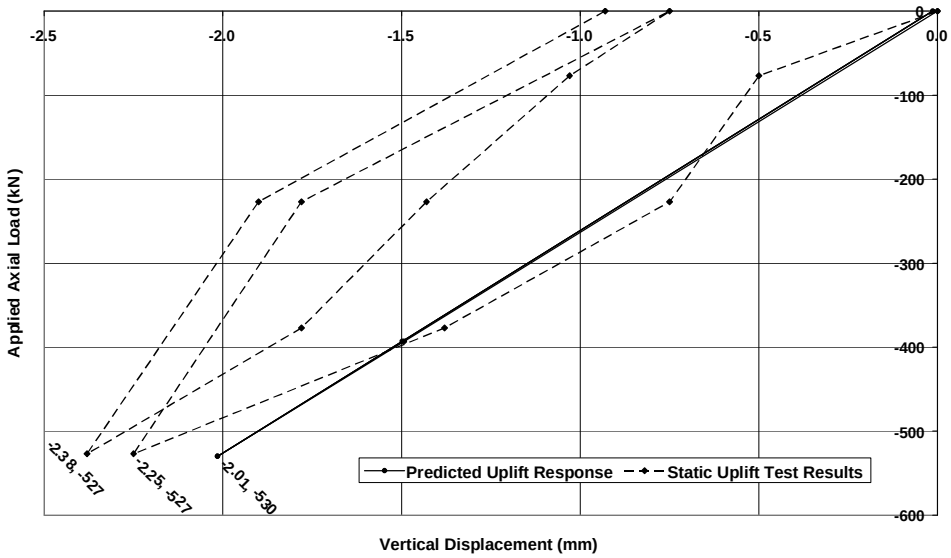


Figure 4: Load-displacement Response to Uplift Loading

As shown in Figure 4, the maximum uplift displacement for an ultimate tensile test load of 527 kN was predicted to be 2.0 mm, with a negligible permanent displacement. The static tension test showed that a maximum average displacement of 2.4 mm was measured for an applied load of 600 kN (plotted in Figure 4 as 527 kN, taking self-weight into account), with a final displacement of 0.9 mm. The plastic deformation measured was slightly greater than that predicted, however the magnitude of deformation was less than 1 mm and relatively insignificant. It is expected that the difference may be attributed to the use of a linear elastic model for a real soil, which could have undergone plastic displacements at some point along the pile length. The slope of the response predicted correlates well with the slope of the response measured during testing, indicating that the values of elastic moduli E used in the model are suitable for the site. The correlation between predicted and measured displacements for uplift loads provided confidence in the geotechnical model adopted for the pile design.

As discussed above, the proposed transmission line would require post-earthquake function, as it would service the western suburbs of the Adelaide metropolitan region. Due to the loose and unconsolidated nature of the St. Kilda formation and the Semaphore Sand, it was considered necessary to assess the potential for liquefaction of these upper sand layers, as liquefaction results in the movement of soil particles into voids otherwise present in loose granular soils. As defined by Youd et. al., "liquefaction refers to the phenomena of seismic generation of large pore-water pressures and consequent softening of granular soils" (2001). Liquefaction would likely result in a loss of support to the upper length of pile, requiring a greater structural capacity of the pile to resist the loading. It would also result in the loss of some shaft resistance to uplift and compression loading.

The piles were initially designed based on an assumption that there would be no support to the upper 3.5 metres of the pile, resulting from liquefaction of the Semaphore Sand layer. A 'free head' pile case was considered in determining the

lateral capacity of the piles and it was found that the structural capacity of the piles governed the design. Shaft resistance of the upper 3.5 metres was also ignored in determining the resistance to uplift and compression loading.

Software PULL2B (Sugar 1989) was used to analyse the response of the pile to the applied ultimate lateral load. A conservative estimate of lateral deflection was calculated to be 28 mm, assuming the top 3.5 m of soil was lost due to liquefaction; however the induced bending moment was less than that required to yield the pile and the pile was expected to remain structurally sound under this design earthquake condition. However, an assessment of the resistance of the founding soil to liquefaction was carried out, to gain an appreciation of the theoretical risk of liquefaction occurring. It should be noted that it is conservative to consider ultimate wind loads under these design earthquake conditions.

## 5. ASSESSMENT OF LIQUEFACTION POTENTIAL

The Youd et. al. (2001) method was used for the assessment of the liquefaction potential. This method has been termed a 'simplified procedure' for use in engineering practice to evaluate the liquefaction resistance of soils. The procedure is based on empirical evaluations of field observations and both field and laboratory test data. The method involves a comparison between the seismic demand on a soil, in terms of a Cyclic Stress Ratio (CSR), and the liquefaction resistance capacity of the soil, in terms of a Cyclic Resistance Ratio (CRR).

The cyclic stress ratio is defined as follows:

$$CSR = 0.65\,(a_{\max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \tag{1}$$

where $a_{max}$ = peak horizontal acceleration at the ground surface, equal to 0.1g for the site (AS1170.4-1993 "Earthquake loads" for a 500 year return period); g = acceleration of gravity; $\sigma_{vo}$ and $\sigma'_{vo}$ are total and effective vertical over-burden stresses, respectively; and $r_d$ = stress reduction coefficient, as defined in Equation (2) below. Clearly, the seismic loading on the soil is dependent on the magnitude of earthquake and the depth (z) of the layer being considered.

$$r_d = 1.0 - 0.00765\,z, \quad \text{for } z \le 9.15 \text{ m} \tag{2a}$$

$$r_d = 1.174 - 0.0267\,z, \quad \text{for } 9.15 \text{ m} < z \le 23 \text{ m} \tag{2b}$$

The liquefaction resistance of a soil (CRR) can be determined from the results of either an SPT, a cone penetration test (CPT), shear-wave velocity measurements ($V_s$) or the Becker penetration test (BPT). Since SPTs were carried out in each borehole for this investigation, these results were used in the liquefaction assessment. A benefit of using SPTs to calculate CRR is that there has been abundant test data recorded at liquefaction sites used to correlate SPT N values with CRR. The CRR for magnitude $M_w = 7.5$ is calculated by the following equation:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10\cdot(N_1)_{60} + 45]^2} - \frac{1}{200}, \quad \text{(valid for } (N_1)_{60} < 30) \tag{3}$$

where $(N_1)_{60}$ is the SPT blow count normalised to an over-burden pressure of 100 kPa and 60 % hammer efficiency. The correction factors to determine $(N_1)_{60}$ are presented in the original source. Account is also taken of the fines content of the soil, as reported by Youd et. al. (2001), an increase in CRR occurs in soils with increased fines content. $(N_1)_{60}$ is converted to a clean-sand value $(N_1)_{60cs}$, dependent on the fines content. For the assessment undertaken for the pile design, a nominal fines content of 20 % was assigned to the silty sands of the St. Kilda and Glanville formations and 0 % for the Semaphore Sand, based on the recovered core samples.

A Magnitude Scaling Factor (MSF) is applied to the $CRR_{7.5}$ value, to convert the resistance ratio to smaller or larger magnitude earthquakes than $M_w = 7.5$. The 'Revised Idriss Scaling Factors' presented by Youd et. al. (2001) were used for the assessment of the Torrens Island piles. The MSF is calculated by Equation (4).

$$MSF = 10^{2.24}/M_w^{2.56} \tag{4}$$

In South Australia only one earthquake has been recorded to have a magnitude exceeding 6.0. In Adelaide, the largest recorded earthquake was magnitude 5.6 in 1954. Love (1996) considered that the expected return period for a magnitude 6.0 earthquake in the Adelaide region was approximately 500 years, which corresponds to the return period for peak acceleration for Adelaide. A magnitude 6.0 earthquake was therefore taken as the design earthquake.

A Factor of Safety (FS) is then calculated as a ratio of $CRR_{7.5}$·MSF to CSR. A value of FS greater than unity indicates the soil layer of concern is safe against liquefaction. For the Torrens Island pile design, a factor of safety against liquefaction was calculated along the length of the upper 11 metres of the design pile, based on the geotechnical model. Only the saturated coarse-grained soil layers were considered, as consolidated fine-grained soils such as those of the

Glanville and Hindmarsh formations are too clayey to liquefy. The CSR, the $CRR_{7.5}$·MSF and the FS values were plotted as a function of depth as presented in Figure 5.

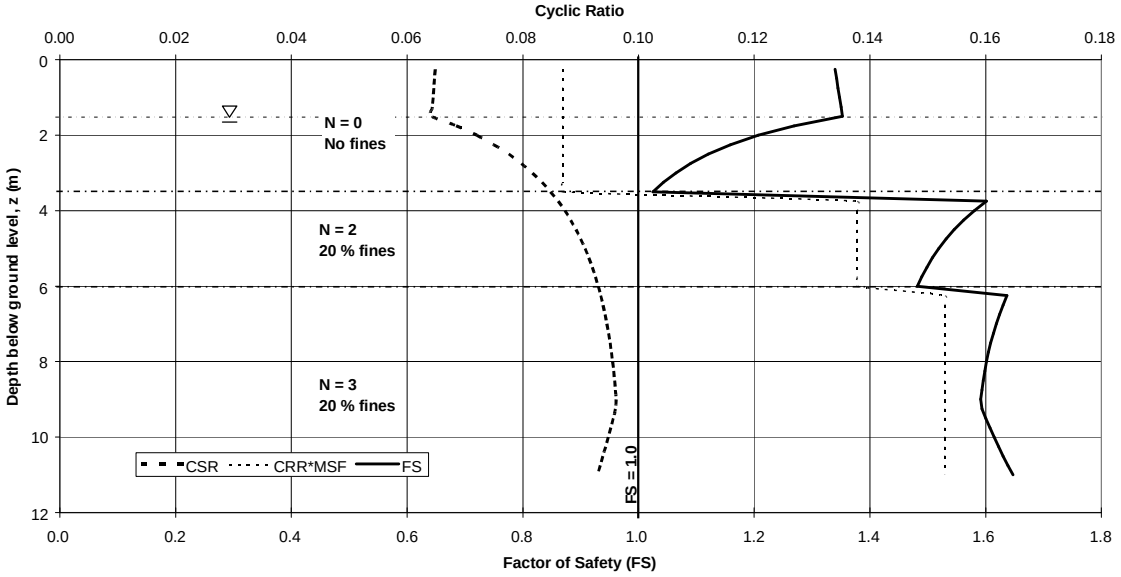


Figure 5: Liquefaction Potential versus Pile Length

It was found that the highest potential for liquefaction was in the upper 3.5 metres, as expected. The value of FS was lowest below the depth of the water table in the Semaphore Sand layer. The factor of safety against liquefaction was greater than unity for the length of the pile, for a magnitude 6.0 earthquake, based on the design geotechnical model. The pile as designed was therefore adequate for the design earthquake loading.

## 6. CONCLUSION

Within this paper the results of a geotechnical investigation and a design for piled footings for a 66 kV transmission line are discussed. Design constraints included high overturning loads and a requirement for post-earthquake function. The geological profile at the Torrens Island site presented some important design considerations, due to loose surficial sands, the presence of a high water table and highly organic matter. A geotechnical model was derived for the design, based on the findings of a desktop study and a site investigation. The model was verified by good correlation between a predicted load-displacement response and in-situ static uplift testing undertaken on one working pile. A liquefaction assessment showed the upper Semaphore Sand layer to have the highest potential for liquefaction, but calculations suggested the soil profile has adequate resistance against liquefaction due to a magnitude 6.0 earthquake. Despite a relatively problematic site, a footing design and construction was successfully undertaken for the transmission line.

## 7. ACKNOWLEDGEMENTS

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# PREDICTION OF STRESS STATES IN THE COOPER BASIN, AUSTRALIA

**C.P. Morelli & S.P. Hunt**
*Australian School of Petroleum*
*The University of Adelaide*

## ABSTRACT

Previously determined borehole breakout and drilling-induced tensile fracture (DITF) data from wells in the Cooper Basin have indicated substantial variability about a mean maximum principal stress direction of 101°N. Using these data, along with over 100 seismically interpreted fault planes that intersect the top of the basement structure, a 2D distinct element (DE) model of the basement of the province was produced. The stress rotations produced by the model were then compared with the borehole breakout and DITF data previously mentioned and the errors compared with those from a previous model of the same area. Knowing the stress state of the basement provides information regarding potential problems that may be encountered in future drilling operations. However the most significant result from predicting the stress state would be for future petroleum production, due to the identification of low volumetric strain regions surrounded by higher volumetric strains, which may indicate regions of hydrocarbon pooling due to pressure gradient potentials. This method would also be applicable to other petroleum producing basins.

## 1 INTRODUCTION

Previously, work has been carried out on the Cooper Basin comparing basin scale DE model predictions with field observed rotational stress data in an attempt to validate the model (Morelli et al., 2006). This work was carried out in order to provide information on the relative strength of faults, so as to determine the risk of fault reactivation. The work was also used to predict the stress variability across the region for future drilling operations due to the fact that stress rotation data is only available from image logs of individual wells once they are drilled and has no bearing on stress rotational data away from the well bore. However, due to some unexpectedly high error results, it was concluded that there were possibly some basement faults 'missing' from the 'original' model and upon further inspection of previous data (Boult, 1996; Warburton Basin GIS Data Atlas, 2002) this was deemed to be the case.

In this paper, the effect of including these 'missing' faults into the DE model (creating a 'new' model) is assessed under an extreme strike slip stress state based on the previously analysed Cooper-Eromanga Basin regional stress state (Reynolds et al., 2006). This was conducted by comparing the field observed borehole breakout and DITF $\sigma_{Hmax}$ rotation data (Reynolds et al., 2005) with those predicted by the model. The model output was then analysed to identify regions of low mean stress surrounded by higher mean stresses to indicate possible regions of hydrocarbon pooling.

## 2 BACKGROUND

### 2.1 GEOLOGY

The Cooper Basin is a NE-SW trending, 2.5 km thick, Late Carboniferous to Middle Triassic, intracratonic, non-marine depositional basin located in central Australia (PIRSA – P&G, 2005), with the study area being the South Australian and adjacent Queensland sectors of the basin (Figure 1). It unconformably overlies the Cambrian to Ordovician Warburton Basin and underlies the Early Jurassic to Late Cretaceous central Eromanga Basin, also unconformably (Figure 2). With depth to the unconformity varying from 970–2800 m and the basin reaching a maximum depth of approximately 4400 m in its deepest trough (Laws & Gravestock, 1998). It also forms the most significant element of the largest onshore oil and gas province in Australia (Laws & Gravestock, 1998) and extensive drilling has occurred since the first natural gas discovery in the Cooper Basin in 1963 (O'Neil, 1998).

Rejuvenations of underlying basement features inherited into the Cooper Basin have been critical for generating some structures for trapping petroleum. This influence on more recent geological events indicates the basement has significant influence on the overlying sedimentary sequence as can be seen in Figure 2, with major 'structures' and 'structural' closures within the Cooper and Eromanga Basins inherited from Warburton or older features (Meixner et al., 1999). The structures are also known to have been episodically reactivated since the Warburton Basin time (Gravestock & Jensen-Schmidt, 1998).

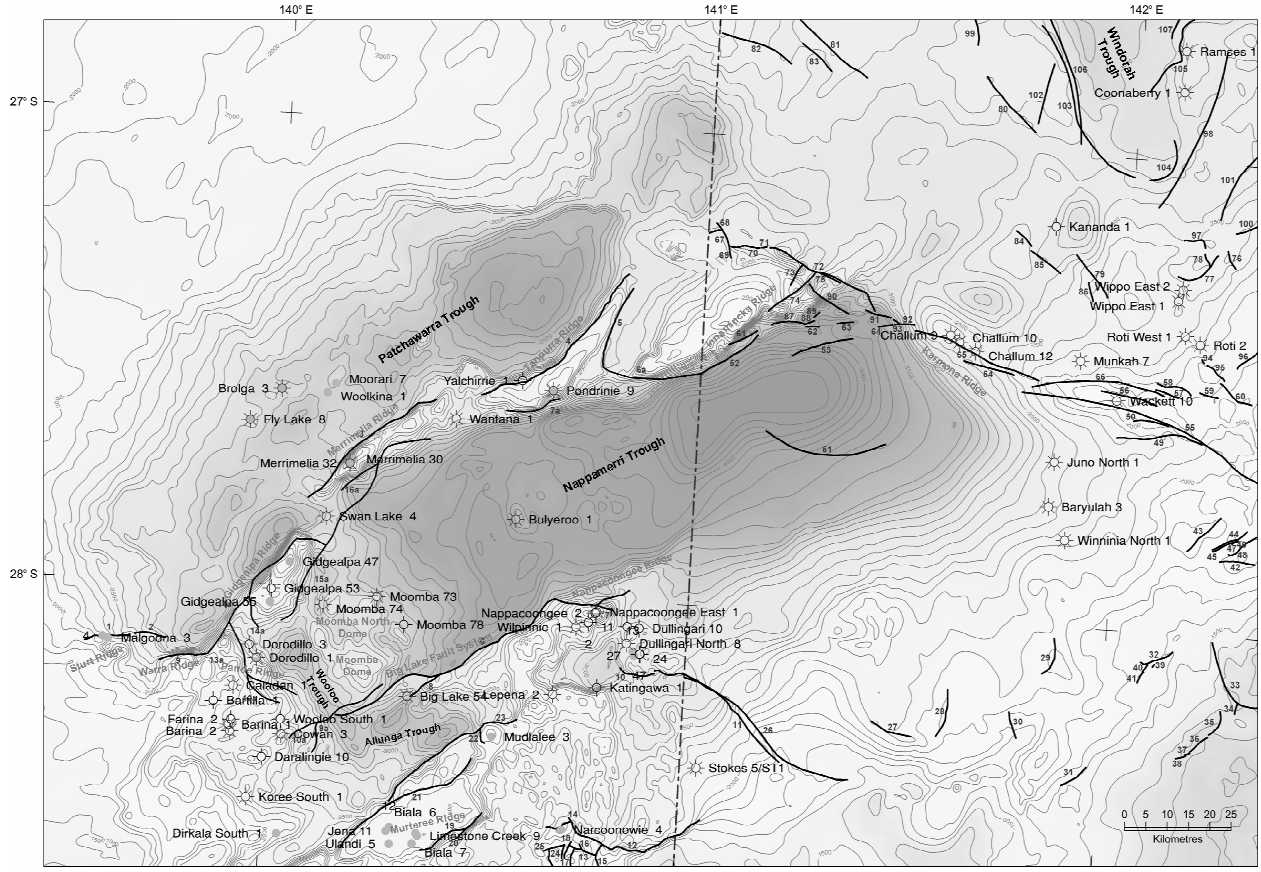


**Figure 1:** Study area, base of Cooper Basin or top of Warburton Basin (Z seismic horizon) depth structure contour map, showing well locations, principal structural elements (major basement-cutting faults) and fault numbering sequence for the 'new' model.

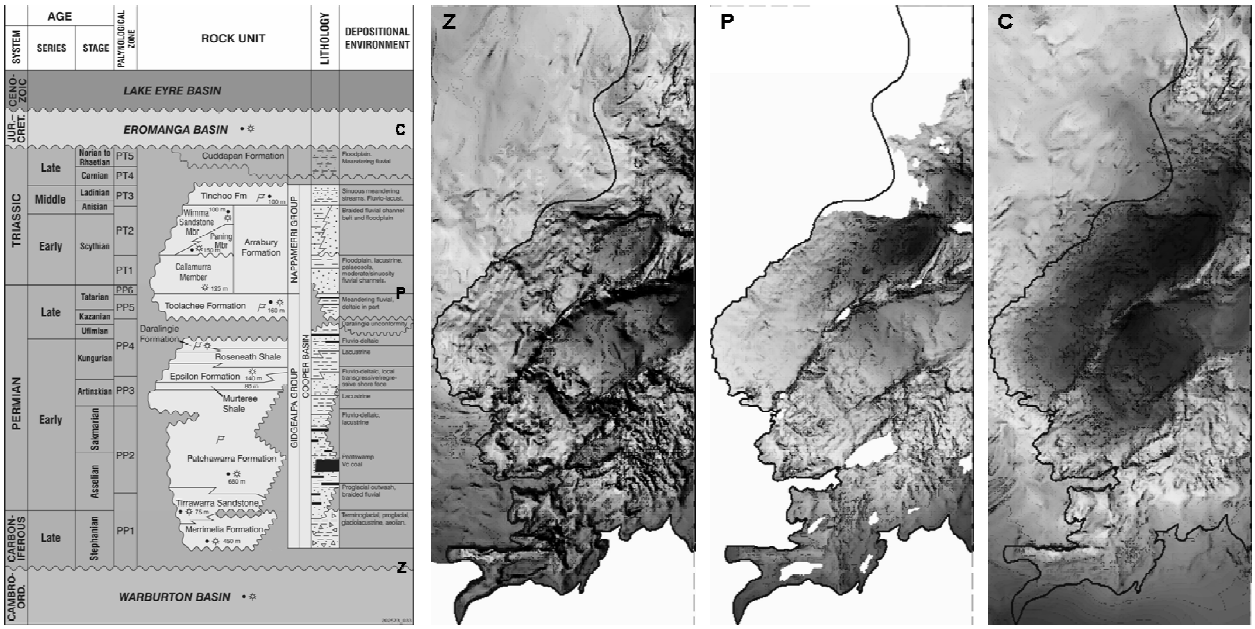


**Figure 2:** Stratigraphic summary of the Cooper Basin, Australia with seismic horizons indicated (PIRSA – P&G, 2005) with depth structure images of (Z Seismic Horizon) Base of the Cooper or top of Warburton Basin, (P Seismic Horizon) Near top of Toolachee Formation (Late Permian), (C Seismic Horizon) Near top of Cadna-owie Formation (Early Cretaceous) also shown (Warburton Basin GIS Data Atlas, 2002).

### 2.2 BASEMENT FAULT INTERPRETATION

Initially a contour depth to basement map was obtained (PIRSA – P&G, 2006), which contained a series of faults interpreted and overlain on the basements structure and was the basis for the original model (Morelli et al., 2006). After analysis of the results from the original model it became clear that a number of faults might be 'missing' from this map, particularly in the Gidgealpa Ridge region. Further investigation into previous reports provided a figure of the Pre-Permian structure of the Gidgealpa region from the South Australian Oil & Gas Corporation (cited in Boult, 1996), as well as the Warburton Basin GIS Data Atlas (2002), both of which contained additional faults. Upon closer inspection a number of major faults were identified and incorporated into the map (Figure 1) and the 'new' model.

### 2.3 MODELLING

Major discontinuities within a rock mass have the ability to disturb the applied stress field. Ramsay (1967) showed how faults might affect stress trajectories and that $\sigma_{Hmax}$ tends to be focused along faults with low cohesion or stiffness and orthogonal to faults with high cohesion or stiffness and many workers have corroborated this work since that time (Bell, 1996; Camac et al., 2004; Hardebeck & Hauksson, 2000; Zhang et al., 1994).

DE modelling simulates faulting-related deformation using the Coulomb slip model, whereby below the frictional limit only elastic deformation (strain) occurs. At a constant magnitude of applied differential stress the resultant elastic deformation (strain) is constant and governed by Young's modulus, as elastic deformation is independent of the Coulomb parameters below the frictional limit. Once the fault reaches it's frictional/shear stress limit, as defined by the strength envelope, shear displacement occurs along the length of the fault.

In order to better understand the parameter sensitivity of a basin-wide DE model, stress perturbations around single faults have been modelled and it was observed during the calibration studies that the model output is highly dependant on the following parameters (Hunt & Boult, 2005):

- ratio $k = \sigma_1 / \sigma_3$
- angle between the maximum principal stress $\sigma_1$ and fault strike, $\theta$
- effective fault friction angle, $\varphi_e$

Previously, the same study area was modelled with the 2D DE method (Morelli et al., 2006) with numerical predictions of $\sigma_{Hmax}$ rotations comparing well with field observation means from borehole breakout and DITF interpretations (Reynolds et al., 2005).

### 2.4 STRESS DATA USED FOR FAULT MECHANICAL MODELLING

The regional stress state has been analysed previously (Reynolds et al., 2006) and these data are presented in Figure 3. In fact the stress regime within the Cooper-Eromanga Basins is thought to vary, hence upper and lower limit bounds are given on Figure 3.

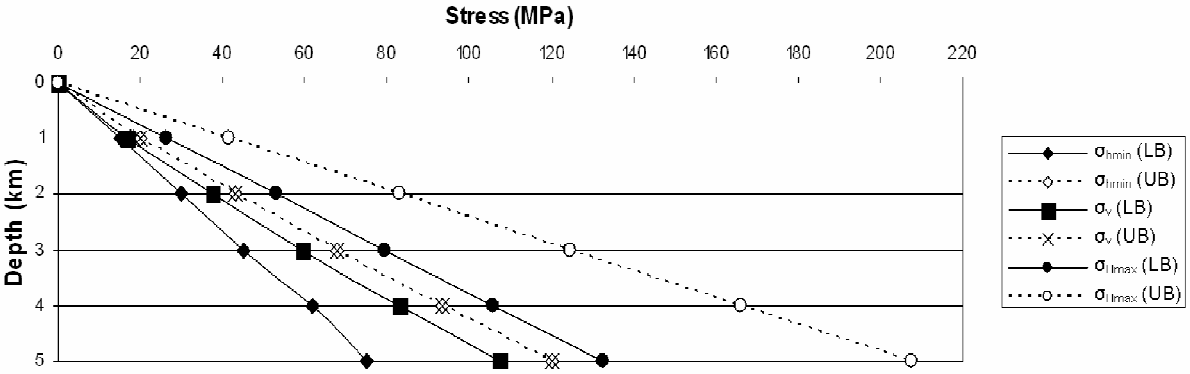


**Figure 3:** Principal stress magnitudes with depth, demonstrating the degree of uncertainty and possibility of strike-slip to reverse stress regimes (Reynolds et al. 2006).

## 3 DISTINCT ELEMENT MODEL

### 3.1 METHOD

The 'original' 2D DE model (Morelli et al., 2006), which was produced using Itasca's Universal Distinct Element Code (UDEC), was modified to contain the additional major faults as mentioned previously. This 'new' model was then subjected to the same stress state as the 'original' model so as to provide a direct comparison between the two models.

### 3.2 PARAMETERS

The properties required for description of the intact rock's mechanical behaviour were: bulk modulus (K), shear modulus (G) and density (ρ). The values used for this 2D DE model of the Cooper Basin, are given in Table 1.

**Table 1:** Rock properties applied to the model.

| Block Density, ρ (kg.m$^{-3}$) | Bulk Modulus, K (GPa) | Block Shear Modulus, G (GPa) |
|---|---|---|
| 2600 | 1.5 | 0.6 |

In this work a low friction angle was applied to the faults to produce significant displacement and to restrict any fault propagation effects, an elastic rock mass was applied. The faults were also assumed to be vertical, hence a plain strain model was applicable. A mean $\sigma_{Hmax}$ orientation of 101°N (Reynolds et al., 2005) and magnitudes as shown in Table 2 were applied to the boundaries of the model to simulate the maximum strike-slip differential stress at a depth of 4 km from Reynolds et al. (2006).

**Table 2:** Stress properties applied to the model.

| $\sigma_{Hmax}$ (MPa) | $\sigma_V$ (MPa) | $\sigma_{hmin}$ (MPa) |
|---|---|---|
| 166 | 93.6 | 62.9 |

## 4 RESULTS & DISCUSSION

### 4.1 FIELD OBSERVATIONS

Figures 4 to 6 show the binned error frequencies of the 'new' model's stress rotation output. These errors are obtained by taking the average $\sigma_{Hmax}$ orientation from a 1 km$^2$ area (1 km by 1 km grid of 100 m intervals) around each wellbore location (which contains approximately 5 discrete zones) and comparing them with the mean stress rotations from the field observations (Reynolds et al., 2005). It is clear from these graphs that the better the World Stress Map quality ranking the borehole breakouts have, the better the correlation with the model. However for the DITF's there does not seem to be the same quality correlation. This may be due to the DITFs in general giving more accurate $\sigma_{Hmax}$ orientations or simply because the sample population is too small to be representative, however further work is required to determine which the case is.

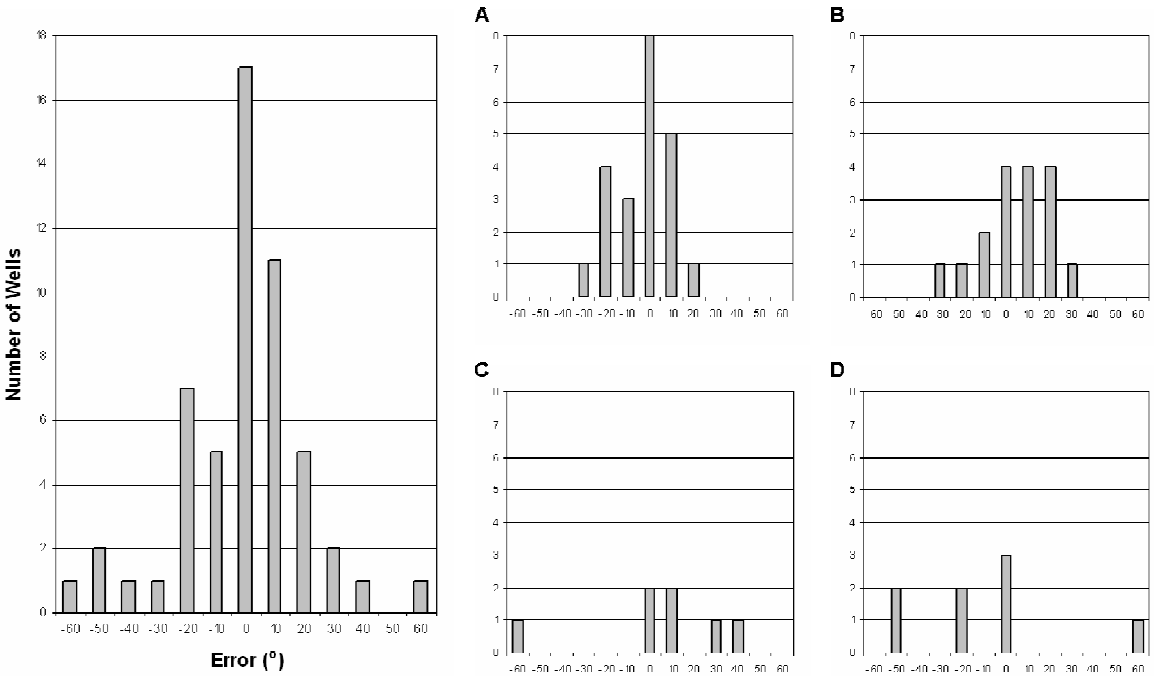


**Figure 4:** Error frequency graphs for 54 wells with borehole breakout (unweighted) determined $\sigma_{Hmax}$ orientations with World Stress Map quality rankings of (A) 22 wells, (B) 17 wells, (C) 7 wells and (D) 8 wells.

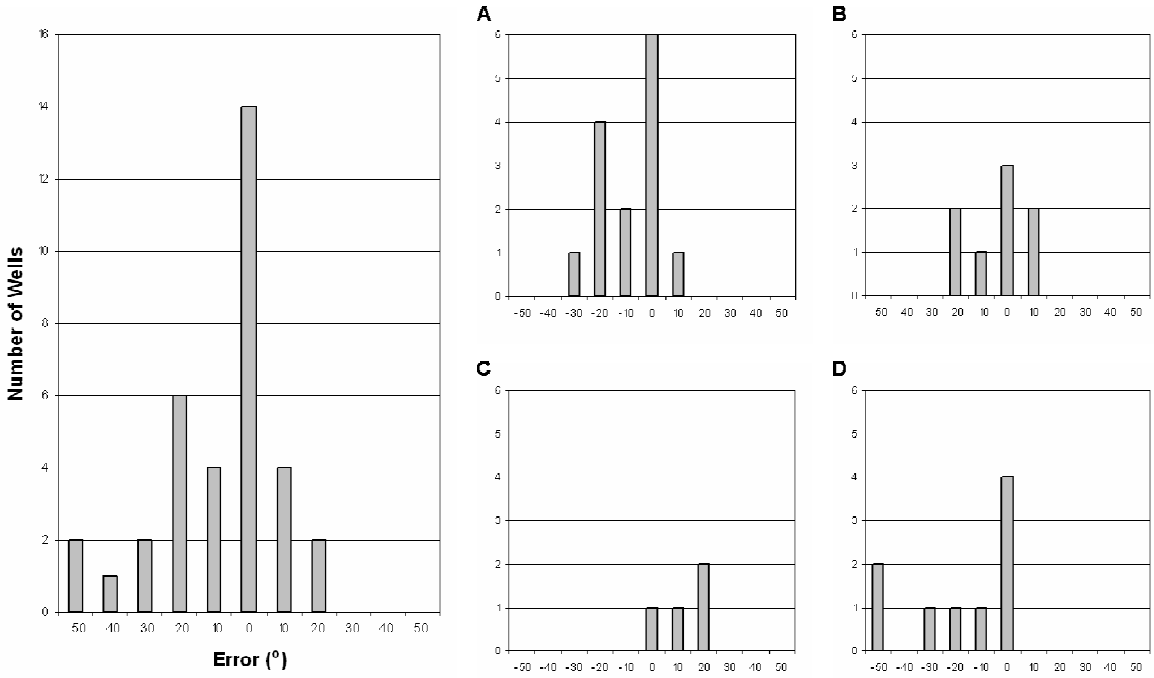


**Figure 5:** Error frequency graphs for 35 wells with borehole breakout (length weighted) determined $\sigma_{Hmax}$ orientations with World Stress Map quality rankings of (A) 14 wells, (B) 8 wells, (C) 4 wells, (D) 9 wells.

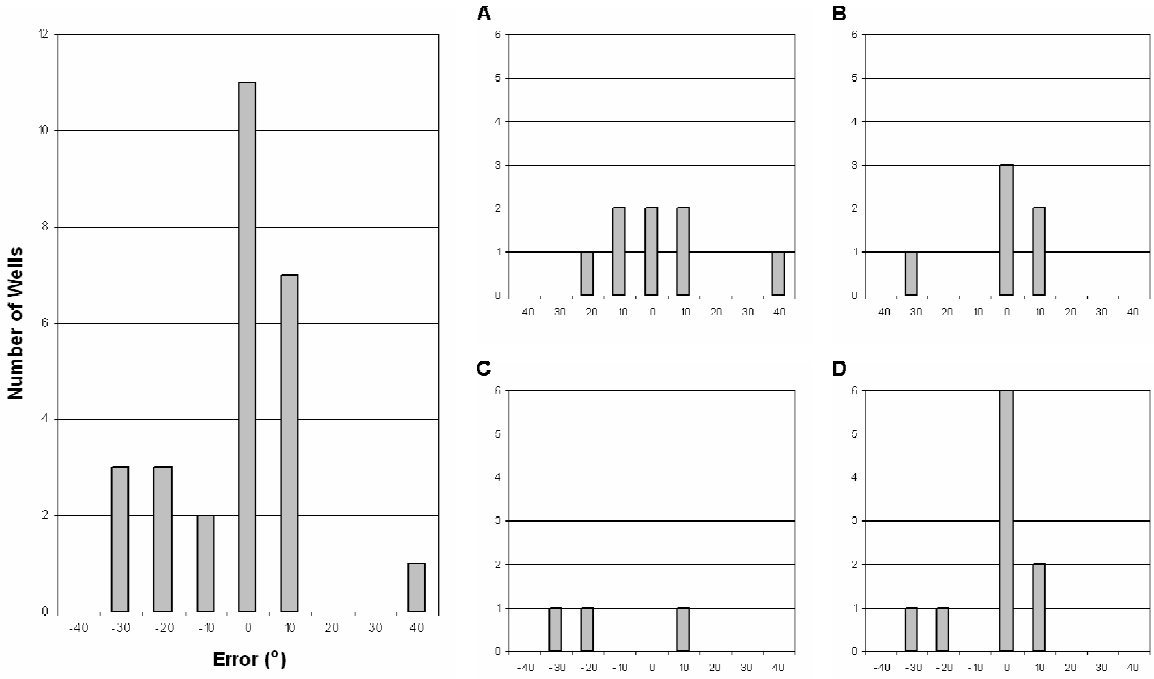


**Figure 6:** Error frequency graphs for 27 wells with DITF (unweighted) determined $\sigma_{Hmax}$ orientations with World Stress Map quality rankings of (A) 8 wells, (B) 6 wells, (C) 3 wells, (D) 10 wells.

Overall, the addition of the extra faults to the 'original' model decreased the error margin for the majority of the field observations. When compared with the 'original' model output, the 'new' model resulted in:

- Borehole breakout (unweighted) error margins decreasing for 69% of wells
- Borehole breakout (length weighted) error margins decreasing for 57% of wells
- DITF (unweighted) error margins decreasing for 52% of wells
- Error margins decreasing for 61% of field observations, indicating the faults added are likely to be correctly interpreted

### 4.2 MODEL PREDICTIONS

Generally the model can be used to predict stress rotations across the region, at basin scale. The model also allows contour maps to be produced of the volumetric strain increment based on the volumetric strain within each model element. Interestingly, when this is plotted, a correlation is observed between a region showing dilation and the Moomba field location. This implies that there could be a link between regions of dilation predicted by the model and areas of pore fluid pooling. It is possible that the faults, as large basement structures, are creating present-day regions of local stress variation that could have controlled recent fluid and gas migration.

## 5 CONCLUSIONS

The effect of adding 'missing' faults to the 'original' DE model (Morelli et al., 2006) was observed by comparing the stress rotation outputs (averaged value for 1 $km^2$ around the wellbore location, containing approximately 5 discrete zones) from each model to the mean stress rotations of the field observations (Reynolds et al., 2005) and noting the difference in error. The addition of these faults decreased the error margins for 61% of field observations in total, indicating that there are faults that are likely to be 'missing' from the 'original' basement structure map and model. Therefore this method, once perfected, could potentially be used as a large scale quality check for basin scale seismic interpretation

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# SETTLING AND CONSOLIDATION BEHAVIOUR OF DREDGED COHESIVE ESTUARINE SOIL USING COLUMN TESTING APPARATUS.

**S. R. Morrison**
*BE Hons (Civil and Environmental)*
*Senior Geotechnical Engineer, Coffey Geotechnics Pty Ltd, Wollongong, Australia*
**A. M. Tait**
*BSc (Geology)*
*Engineering Geologist, Coffey Geotechnics Pty Ltd, Newcastle, Australia*

## ABSTRACT

Consolidation testing on in-situ normally or over consolidated cohesive soils is commonly used to assess engineering parameters such as $C_v$, $C_c$ and $C_{\alpha.}$ However, the application of conventional consolidation tests to reconstituted soil slurries is often unachievable. A column testing method for assessing the settling and consolidation behaviour of dredged soils was devised and applied to five samples of estuarine soil in a brackish (somewhat salty) water environment sourced from the South Coast of N.S.W, Australia. Test results show that void ratio and bulk density values after the settling phase of the test were comparable with results of in-situ soil at the initial stage of standard laboratory oedometer testing. Calculation of compressibility coefficients $C_c$ and $C_\alpha$ after the loading stage of the test revealed similar results to oedometer testing on the soils in the in-situ state. The testing method was assessed to provide results within a more reasonable timeframe with sample thicknesses of approximately 100mm. It was assessed that soil thicknesses greater than 100mm in the columns were influenced by simultaneous primary and creep settlement and that unreasonable test duration would be required to achieve meaningful results in an economic timeframe.

## 1. INTRODUCTION

Consolidation parameters such as $C_v$, $C_c$ and $C_\alpha$ are commonly derived from laboratory oedometer tests such as AS1289 6.6.1 (1998) on 'undisturbed', 50mm or 63mm diameter soil samples retrieved from site investigation techniques. Frequently these samples show some degree of over – consolidation, enabling the test to be carried out in small diameter ring apparatus without compromising consolidation data. However, consolidation testing of soils that have been reconstituted to a suspension and subsequently settled often cannot be successfully tested using conventional testing methods (Krizek (2004) and Sridharan and Prakash (1999)).

Accessible land with favourable geotechnical properties is becoming limited in society and increasingly areas of reclaimed and contaminated land are being used for infrastructure purposes. Also the offshore disposal of dredged soil is less widely accepted by the community as an acceptable use of dredged soil. The viability of reuse of dredged cohesive soils onland is increasing where previously the cost of reuse of this type of material was considered prohibitive.

In many cases, developing reclaimed or contaminated land often requires the dredging of soils from marine, alluvial and estuarine environments or dredging from mining tailings or contaminated land. In most cases this dredged material is deposited in a holding area (i.e. tailings dam) to be subsequently used as reclaimed land. The nature of the soil as it settles from suspension and various properties such as the rate and magnitude of consolidation settlement must be assessed for decisions to be made as to whether the dredged soil can be reused as reclaimed land.

This paper presents the results of testing performed on cohesive soils sourced from an estuarine environment located on the South Coast of N.S.W, Australia. Significant research has previously been carried out on testing of dredged soil materials in freshwater environments (Krizek (2004) and Sheeran and Krizek (1971)). This paper discusses the settling behaviour of the dredged slurry suspension in brackish water for different soil to water ratios. It also provides assessment of $\rho_d$ and $e_o$ during the settling phase, assessment of consolidation parameters $C_v$, $C_c$ and $C_\alpha$. A comparison is also made between the results of the column testing for 'dredged' soil and conventional oedometer testing values from three samples of 'undisturbed' soil.

## 2. METHODOLOGY

The assessment of settling and consolidation behaviour of the soils was conducted using testing apparatus constructed exclusively for the test. The testing consisted of two phases, named Phase 1 and Phase 2. Phase 1 involved

observation of the settling of the slurry suspension, and Phase 2 involved observation of the rate of consolidation over time with the addition of different surcharge weights. Australian Standard AS 1289 6.6.1 (1998) was used as a general reference for testing the procedure; however several modifications were made due to the unique nature of the testing.

## 2.1 APPARATUS

Fifteen clear Perspex columns with an inner diameter of 94mm were used and each column had a Perspex base fixed with 'Acrifix' Adhesive and a reinforced collar to maintain a water tight seal and integrity of the joint under additional pressure. A plastic tube was drilled into the side of the column 10mm from the base, with an outlet fixed to the top of the column.

The columns were firmly fixed to a wall using a metal collar placed near the top of the column to prevent movement (accidental or intentional) during the course of testing.

Prior to the settling phase of the testing, a 20mm layer of sand was placed at the base of each Perspex column and filter paper was placed over the sand. This provided "two-way drainage" to facilitate consolidation in the laboratory condition. This was carried out in an attempt to quicken the rate of settlement with the knowledge that a theoretical correction can be applied to settlement estimates if only "one-way" drainage exists in the field.

After the settling phase of the testing was completed, an additional filter layer was constructed on the top of the settled soil surface. This included placing a layer of filter paper over the soil surface and a 20mm thick layer of sand over the paper to facilitate drainage. An additional layer of filter paper was placed on top of the sand drainage layer prior to surcharge load being added.

Surcharge loads were constructed from concrete, pre-cast into PVC pipe with an external diameter of 90mm. The weights were cured for seven days then weighed and measured to calculate load. The surcharge loads were cut to size to meet target preload pressures that were nominated for the testing procedure. The difference in annulus between the column diameter and the surcharge weight provided a drainage path for pore water to pass out of the consolidating soil, into the upper filtration layer and subsequently add to the column of 'free water' that existed above the consolidating soil.

A schematic diagram of the column testing apparatus is presented in Figure 1 and Figure 2.

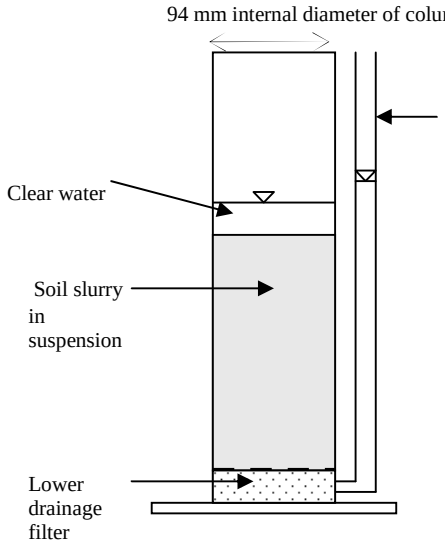


**Figure 1.** Phase 1.Settling of slurry suspension

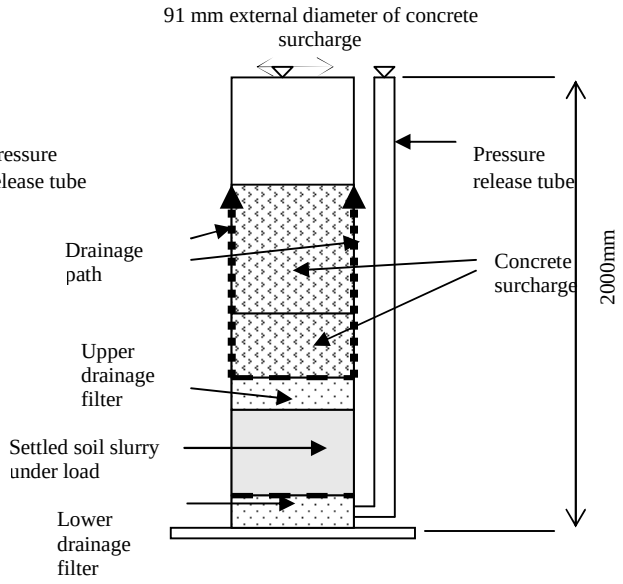


**Figure 2.** Phase 2 Settled soil under load

## 2.2 MATERIALS

The soils used for this testing program were sourced from an estuarine environment located on the south coast of New South Wales. The soils were generally described as sandy clayey silts or silty clays and are typical of Holocene aged estuarine soils found in the region. The natural (initial) moisture content of the five samples of estuarine soils used for this column test ranged between 48% and 133%. No Atterberg Limit testing was carried out on the five estuarine soil samples prior to column testing, however twelve Atterberg Limit tests had been previously carried out on the clayey or

silty estuarine soils at the site. Results from these twelve nearby tests are considered to be generally representative of the estuarine soil properties at the site. The Plastic Limit of the twelve nearby soil samples ranged between 15% and 39%, and the Liquid Limit of the twelve samples ranged from 39% to 78%.

To source material for the column test trial, six test pits (designated CGTP21 to CGTP26) were excavated on the 20 January 2004 and six samples were collected that generally represented a variety of estuarine soils at the site. The particle size distributions of the tested samples are presented below in Figure 3. Standard Particle Size Distribution testing including hydrometer analysis according to AS 1289.3.6.2 (2000) was used to assess the silt and clay fractions below 75µm.

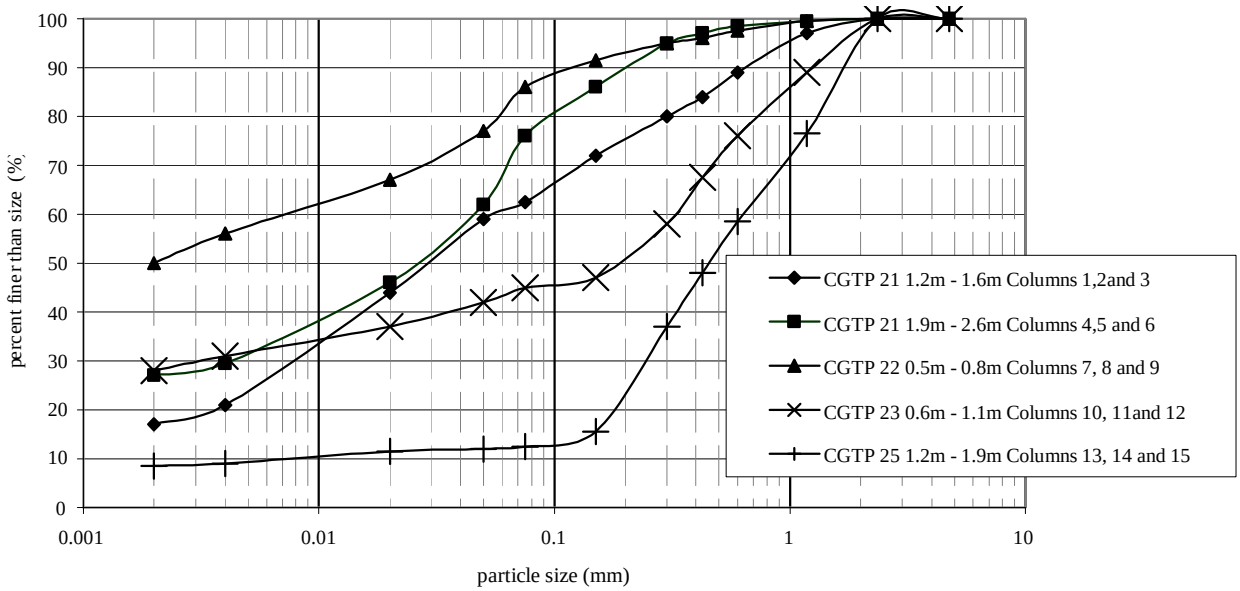


**Figure 3.** Particle size distributions of tested samples

Water from the existing swamp on the site was collected for use in the testing procedure. The water was collected approximately 500m inland and was generally brackish (somewhat salty) with a pH of 6.32. Following collection, the water was immediately taken back to the laboratory for subsequent testing.

## 2.3 PROCEDURES

***Phase 1 – Settlement of Slurry under self weight***

A bakery 'Hobart' mixer was used to mix the samples. The soils were mixed to moisture contents of 233%, 300% and 400%. The initial void ratios ($e_o$) of the slurried material ranged between 5.14 and 10.04. The Hobart mixer provided a generally homogenous and consistent mix in a controlled condition with minimal sample loss. The mixed slurry suspension was siphoned into the column using a plastic hose placed near the base of the column and this was raised as material filled the column. Standard testing sheets were developed to record relevant data and observations during the testing procedure. Data was recorded during the mixing and dispensing of the slurry. The height of settled soil was recorded at 10 minute intervals up to about 40 minutes, then at 60 minute intervals for 4 hours, then twice daily for the duration of the test.

Measurements were taken at the interface of the settling slurry suspension and clear water above the suspension. The test was continued generally until less than 1mm to 2mm of reduction in the height of the settled soil was achieved over two consecutive days.

Some deviations to this general procedure occurred in columns 6 and 13 for the 1000mm thickness soil samples. Higher rates of settlement of between 3mm and 16mm per day were reported in these columns at the completion of the test.

Periodic pH measurements of the water above the slurry were also made using an electronic pH probe calibrated to pH 4 and pH 7 reference solutions on the day of testing.

***Phase 2 – Consolidation of slurry under vertical stress***

Following completion of the Phase 1 testing, water was siphoned out of the column (taking care not to disturb the settled soil), leaving about 50mm of water covering the settled soil. The upper sand filter layer was placed over the settled soil. The amount of sand used was weighed and then placed without disturbing the settled soil. The column was carefully filled with water to the top of the column, adding water with as little turbulence as possible. The level between the settled soil and the bottom layer of sand was recorded relative to the base of the column. Phase 2 of the testing commenced with the sample being loaded with a 2kPa surcharge load. Measurements were taken immediately after the positioning of the surcharge load at time intervals of 6 sec, 15 sec, 30 sec, 1 min, 2 min 15 sec, 4 min, 6min 15 sec, 9 min, 12min 15 sec, 16min, 20 min 15 sec, 25 min then once daily until measurements of less than 1mm difference in reduction of sample height was recorded over three to eight rounds of monitoring (indicating completion of primary consolidation settlement under the given load increment). Placement of surcharge load weights was continued in this manner until between about 5 to 30kPa of load was applied to the sample.

For the columns with soil slurry heights of greater than 100mm (thicknesses generally 300mm and 1000mm in this test), completion of >90% primary consolidation was not achieved after several months of monitoring. Even though >90% primary consolidation had not been achieved in these columns, another load increment was added to check that settlement under the given loads was indeed slow and was not due to other factors such as friction. For these columns with thicker soil slurry heights, in all cases it was assessed that >90% primary consolidation had not been achieved even after 6 to 9 months of testing under the stated loads.

# 3 RESULTS

## 3.1 PHASE 1 – SETTLEMENT OF SLURRY UNDER SELF WEIGHT

Settling of the slurry suspension was generally achieved between 0.1 and 28 days following placement of the soil slurry into the columns. For two columns some higher rates of settlement were reported after 8 to 35 days of settlement under self weight. A reduction in void ratio of up to 77% was achieved at the completion of the Phase 1 testing.

A direct correlation between settling time and soil/water content could not be identified from the Phase 1 testing. Correlation of settling time and soil/water content would require a consistent slurry height (say 100mm) to be used for the 20%, 25% and 30% soil/water ratios.

The pH of the water above the slurry dropped from 6.32 to between about 2.5 and 3.3 within about 5 to 7 days following placement. This is due to the oxidation of the known Acid Sulfate Soil materials within the estuarine soil. The implications of this change in pH along with other changes in soil chemistry are beyond the scope of this paper but would need to be addressed for developments where dredging of known Acid Sulfate Soil slurry materials occurs.

The following table presents a summary of the results from the Phase 1 Laboratory Testing, along with a comparison of results from Oedometer testing of $U_{50}$ tube field samples taken from the estuarine soil unit.

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| Sample Number | Sample Diameter (mm) | Before Phase 1 Consolidation | | | | | After Phase 1 Consolidation* under self weight | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Initial Sample Thickness (mm) | Initial M.C.(%) | Initial $\rho_w$ | Initial $\rho_d$ | Initial $e_o$ | Thickness (mm) | Final M.C.(%) | Settled $\rho_w$ | Settled $\rho_d$ | Final $e_o$ |
| Column 1 | 94 | 205 | 400 | 1.21 | 0.24 | 9.24 | 120 | 111 | 1.69 | 0.80 | 2.15 |
| Column 2 | 94 | 825 | 300 | 1.20 | 0.30 | 7.39 | 562 | 194 | 1.30 | 0.44 | 4.72 |
| Column 3 | 94 | 1240 | 233 | 1.26 | 0.33 | 5.68 | 968 | 175 | 1.33 | 0.48 | 4.22 |
| Column 4 | 94 | 205 | 400 | 1.21 | 0.24 | 9.42 | 97 | 182 | 1.44 | 0.51 | 3.93 |
| Column 5 | 94 | 1290 | 300 | 1.20 | 0.30 | 7.37 | 763 | 164 | 1.35 | 0.51 | 3.95 |
| Column 6 | 94 | 1219 | 233 | 1.35 | 0.41 | 5.20 | 753 | 139 | 1.57 | 0.66 | 2.83 |
| Column 7 | 94 | 220 | 400 | 1.24 | 0.25 | 9.14 | 94 | 169.6 | 1.57 | 0.58 | 3.33 |
| Column 8 | 94 | 820 | 300 | 1.24 | 0.31 | 7.10 | 439 | 151 | 1.44 | 0.55 | 3.34 |
| Column 9 | 94 | 1273 | 233 | 1.29 | 0.39 | 5.52 | 768 | 131 | 1.48 | 0.64 | 2.93 |
| Column 10 | 94 | 220 | 400 | 1.14 | 0.23 | 10.04 | 144 | 249 | 1.22 | 0.35 | 6.23 |
| Column 11 | 94 | 785 | 300 | 1.24 | 0.31 | 7.11 | 474 | 173 | 1.40 | 0.51 | 3.90 |
| Column 12 | 94 | 1190 | 233 | 1.41 | 0.42 | 4.97 | 1107 | 217 | 1.44 | 0.45 | 4.56 |
| Column 13 | 94 | 1191 | 233 | 1.37 | 0.41 | 5.14 | 1120 | 218 | 1.39 | 0.44 | 4.77 |
| Column 14 | 94 | 780 | 300 | 1.25 | 0.31 | 7.03 | 585 | 220 | 1.34 | 0.42 | 5.03 |
| Column 15 | 94 | 177 | 400 | 1.51 | 0.30 | 7.34 | 127 | 307 | 1.71 | 0.42 | 4.99 |
| Oedometer 1 | 46.0 | - | 68.2 | 1.61 | 0.96 | 1.86 | - | - | - | - | - |
| Oedometer 2 | 46.5 | - | 111.2 | 1.29 | 0.61 | 2.77 | - | - | - | - | - |
| Oedometer 3 | 46.0 | - | 130.0 | 1.24 | 0.54 | 3.06 | - | - | - | - | - |
| Oedometer 4 (Golder) | 50 | - | 46.6 | 1.73 | 1.18 | 1.21 | - | - | - | - | - |

- ** - After Phase 1 Consolidation is defined as completion of primary consolidation settlement under self weight. These are slightly different to the values shown for 'Before Phase 2 Consolidation' in Table 2, below.*
- *$\rho_w$ – wet or bulk density, tonnes/m$^3$. $\rho_d$ – dry density, tonnes/m$^3$. $e_o$ – void ratio*

**Table 1.** Phase 1 test results

### 3.2 PHASE 2 – CONSOLIDATION OF SLURRY UNDER VERTICAL STRESS

For the Phase 2 testing, values of $C_c/(1+e_o)$ ranged between 0.180 and 0.425 and for the oedometer tests the $Cc/(1+e_o)$ values ranged between 0.179 and 0.592. For the phase 2 testing, values of $C_\alpha/(1+e_o)$ ranged between 0.0069 and 0.0110 and for the odometer tests the values of $C_\alpha/(1+e_o)$ ranged between 0.0067 and 0.0138. It is assessed that a general correlation between column testing and conventional odometer testing can be made.

Meaningful results were only recorded from the 100mm (approximate) height soil columns. It was found that soil heights greater than this were continuing to record primary consolidation settlement after 10 months of testing. It is expected that results for soil heights greater that 100mm may take many years to achieve meaningful results for the Phase 2 testing.

It should also be pointed out that sample thickness does have an influence on the consolidation characteristic of soils as hypothesised by Jamiolkowski et al (1985), and as observed by other researchers in subsequent development of time-dependent models for simultaneous consolidation and creep compression.

Plots of thickness of soil (slurry) height versus log time were prepared based on the readings taken during the course of monitoring. Due to space restrictions there is not sufficient room for presentation of these results, however an assessment of the coefficient of vertical consolidation, $C_v$, was made based on an assessment of $t_{50}$ (the time for completion of 50% primary consolidation) and standard methods contained in AS 1289 6.6.1 (1998). The results for $C_v$ for the 100mm thick soil height columns displayed some scatter and ranged between 0.06m$^2$/year and 30.36m$^2$/year, however most results ranged between 0.3m$^2$/year and 1m$^2$/year.

The following table presents a summary of the results from the Phase 2 Laboratory Testing, along with a comparison of results from Oedometer testing of $U_{50}$ tube field samples taken from the estuarine soil unit.

| Sample Number | Sample Diameter (mm) | Before Phase 2 Consolidation Testing* | | | After Phase 2 Consolidation Testing | | | |
|---|---|---|---|---|---|---|---|---|
| | | Initial Sample Thickness (mm) | Initial Moisture Content (%) | Initial $e_o$ | Final Sample Thickness (mm) | Final $e_o$ | $C_c/(1+e_o)$ | $C_\alpha/(1+e_o)$ |
| Column 1 | 94 | 120 | 228.2 | 5.10 | 74 | 2.76 | 0.425 | 0.0110 (0.0110 - 0.0053) |
| Column 4 | 94 | 97 | 181.9 | 3.93 | 41 | 1.08 | 0.250 | 0.0093 (0.0012 - 0.0093) |
| Column 7 | 94 | 94 | 169.6 | 3.33 | 64 | 1.95 | 0.180 | 0.0069 (0.0008 - 0.0069) |
| Oedometer 1 | 46.0 | 17.29 | 68.2 | 1.86 | - | 1.05 | 0.310 | 0.0067 (0.0012 - 0.0067) |
| Oedometer 2 | 46.5 | 16.15 | 111.2 | 2.77 | - | 1.15 | 0.566 | 0.0111 (0.0000 - 0.0111) |
| Oedometer 3 | 46.0 | 16.62 | 130.0 | 3.06 | - | 1.68 | 0.592 | 0.0138 (0.0110 - 0.0053) |
| Oedometer 4 (Golder) | 50 | - | 46.6 | 1.21 | - | 0.73 | 0.179 | 0.0110 (0.0000 - 0.0138) |

- ** - Before Phase 2 Consolidation testing (for the Column tests 1, 4 and 7 only) is defined as after primary consolidation settlement under self weight, placement of sand drainage layer over surface of slurry, addition of a water up to the brim of the 2m high column and immediately after placement of initial concrete weight over the settled soil slurry.*
- *$e_o$ – void ratio*

**Table 2** Results of Phase 2 testing

Results from Table 2 show that the test methods used seem effective for assessment of $C_c/(1+e_o)$ and $C_\alpha/(1+e_o)$ for materials that are reconstituted and placed at low effective stresses (<30kPa) and high initial void ratios ($e_o$ ranging between 9.14 to 9.42).

Figure 4 plots void ratio versus normal effective stress for the Phase 2 column testing and oedometer test results. The slope of the primary consolidation portion of the oedometer curves appears similar to the slope of the primary consolidation curves for the Phase 1 and 2 column testing. The results for the column testing confirms the absence of

pre-consolidation pressure, which is expected for these reconstituted soils. Values of $C_c/(1+e_o)$ and $C_\alpha/(1+e_o)$ appear similar for both test methods where a 100mm soil thickness for the column testing has been used.

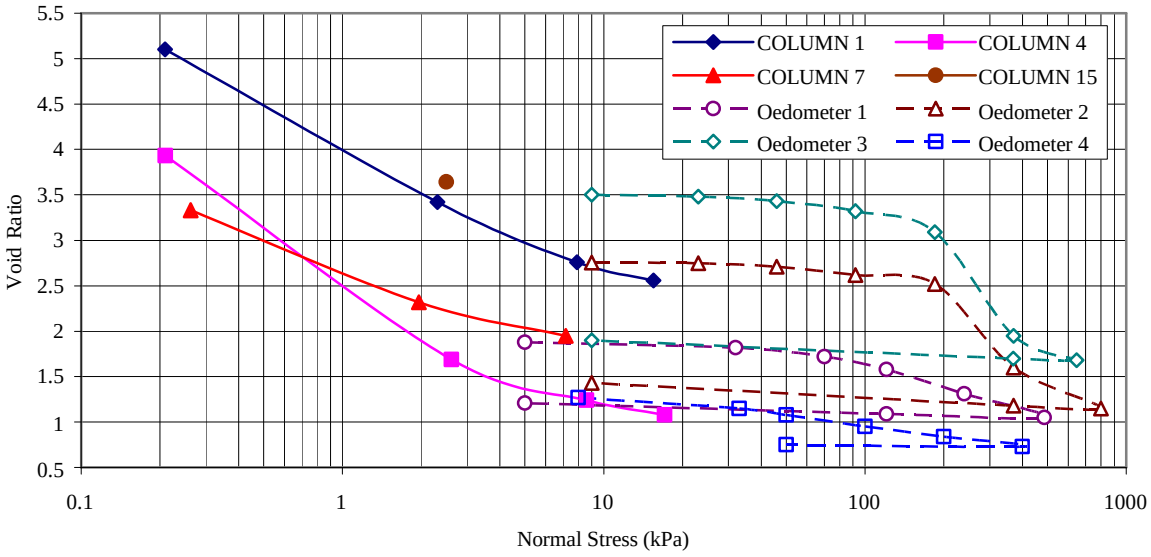


**Figure 4.** Comparison of phase 2 column testing results and oedometer testing results for void ratio versus normal effective stress

## CONCLUSIONS

The results show the test method adopted for the assessment of settling under self weight and consolidation under load is generally suitable for dredged or reconstituted estuarine type soils in a brackish (somewhat salty) water environment. The test appears suitable for soil thicknesses of about 100mm. Testing of thicknesses greater than this would require onerous timeframes or a modified testing method. For a 100mm thickness, results show that a timeframe of 9 to 12 months would be required for adequate results. Results of thick samples greater than about 100mm are also likely to be affected by simultaneous primary consolidation and secondary consolidation (creep) behaviour and would require different test methods to provide meaningful results in an economic timeframe.

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# EXPANSIVE SOILS AND TREES IN THE URBAN ENVIRONMENT: A SOUTH AUSTRALIAN EXPERIENCE

**Aaron O'Malley**
*Golder Associates Pty Ltd*
*Adelaide, South Australia*

## ABSTRACT

This paper presents the authors PhD research on the influence of immature urban street trees on soil moisture, ground movements, footing movements and species' water demand on extremely reactive soil in a newly established subdivision in Adelaide. The project is considered unique as pavement and footing soil movements and street trees' water demands have been studied simultaneously. Research sites were established to monitor deep soil moisture changes, soil suction, surface movements of house floors and pavements and native and exotic tree species water demands. House surveys were used to examine the performance of current footing designs according to AS2870 build on extremely reactive soils and in close proximity to trees. The research has determined movement patterns prior to street trees becoming established and has provided a comparative benchmark for future movements that may arise as the trees mature and gardens become established. This paper focuses on the classification and reactivity of the research sites soils and physiological water demands of the various tree species.

## 1. THE RESEARCH PROBLEM

In Adelaide, the black earth soil in the Paradise and Athelstone areas and the red brown earth soil in the Modbury, Oakden, and the Walkley Heights areas have predominately the greatest seasonal surface movements (over 70 mm) in South Australia. The combination of Adelaide's reactive soils and the prevailing semi-arid climate means that houses and pavements must withstand significant soil movements. Trees in an urban environment can exacerbate these movements and can cause indirect damage to pavements and buildings, through the extraction of moisture from clay soils, causing deep drying and shrinkage settlement. Past research has shown that tree root systems can generate shrinkage settlements in reactive clay soils at appreciable depths (Cameron 2001, Richards et al 1983 and McInnes 1986). As tree-related desiccation is likely to cause greater ground movements than normal in the dry season, the extra soil movement may result in unsightly and perhaps structural damage.

However trees are an environmentally desirable part of urban streetscapes. Street trees are provided by Local Government Authorities to improve the landscape, enhance the environment and usually to maintain or increase land values (Moore 1997). The current trend in Australia towards smaller housing lots, has put pressure on the design of footings using the prescribed safe distances for rows of street trees from houses. This may ultimately lead to a relatively treeless urban environment in areas of clay-rich soil profiles, a situation which would be aesthetically undesirable.

Generally houses over ten years old have not been designed to cater for tree-drying effects and are at a greater risk of damage than recently built houses. More recent house construction remains just as susceptible, if designed to the current Australian Standard AS2870-1996. The Standard encourages the avoidance of manageable extreme moisture changes and does not provide guidelines for designing for trees close to dwellings.

Current footing design engineers try to minimise the risk of soil shrinkage settlements by excluding trees or, more recently, by designing footings to cater for anticipated soil movement induced by trees. Current design criteria for "tree-designs" are based on simplistic empiricism, as very little information is available on the relative water usage of different tree species in the urban environment. Without this information, footings are either being over designed, adding substantially to building costs, or under designed, resulting in footing failures as the trees reach maturity.

## 2. RESEARCH SITES AND METHODOLOGY

Seven research sites and 23 soil monitoring holes were established at the Walkley Height's subdivision, north of Adelaide (See Figure 1). Each site represented a single species of planted street trees as shown in Table 1. Vertical access holes for monitoring of soil moisture to 6.5 m depths below the surface were installed near the row of trees, between rows of trees, under the sealed pavement and away from the row trees at each site. The researched trees immature heights and their relative distances from the installed access holes are shown in Table 1. Surface levelling pads were located in the tree verges and movement markers were installed into the pavements (two traverses) near each

site for monitoring of ground surface movements in reference to stable benchmarks. Soil moisture and suction data were used to confirm and account for the movements observed in the level surveys. Tree water demands were monitored by leaf water potential (LWP) methods using a Wescor in-situ hygrometer and xylem sap pressure methods using a custom built pressure chamber (pressure bomb) based on measurement techniques proposed by O'Malley (2001).

Figure 1: Research Subdivision Location

Level surveys of a number of house slabs were performed prior to mainframe construction and other slabs within a year of house construction. The development of movement of these slabs was investigated by re-survey's of the floor levels on a 6 monthly basis. The research installations were regularly monitored to gain understanding on the development of background ground movement and soils moisture patterns of on a new urban housing estate and to investigate the trees physiologies and seasonal water demands.

Further details of the sites, the methodology of the research and some early findings are reported in Cameron and O'Malley (2003).

Table 1: Trees Species and Relative Distance Parameters for each Site

| Site Name | Tree Species, Common Name | Sept. 2005 Immature Tree Heights (m) | Distance from Tree to Monitoring Hole (m) | | | |
|---|---|---|---|---|---|---|
| | | | BH 1 Near tree | BH 2 Between trees | BH 3 Below Pavement | BH 4 Away from trees |
| Homestead Av. | Chinese Elm | 4.7 | 1.2 | 6.6 | 6.4 | 9.6 |
| Legend Av. | Ornamental Pear | 2.1 | 1.6 | 8.0 | 4.5 | 50 |
| Dene St. | Ornamental Pear | 2.4 | 1.0 | 4.6 | 4.4 | 63.5 |
| Drover Ct. | Golden Raintree | 3.4 | 1.7 | 2.7 | 5.3 | 27.5 |
| Saddle Cr. | Golden Raintree | 3.1 | 1.0 | 4.5 | 4.5 | 49.0 |
| Station Ct. | Coral Gum | 2.0 | 1.2 | - | 11.0 | 44.0 |

## 3. TREE WATER DEMANDS

It has been difficult to make a conclusive comparison between the native and exotic tree species water demands (suctions) in this project as only one native species is represented, the Coral Gum on Station Ct., and the three exotic trees were all quite different. The data in Figure 2 suggests that the native tree's suctions tended to be higher overall throughout the year and dropped only marginally during the winter months. The highest recorded suction reading of -2.3 MPa was taken from both the native coral gum and the Ornamental Pear on Legend Avenue. The patterns of plant suctions with the seasons were very similar over most of the years except when the exotic trees lost their leaves and suction values dropped considerably in the cooler months (to -0.5 MPa). Therefore it can be said for the exotic tree species which tend to have their highest suction values during the summer months and drop considerably during leaf fall in the cooler months of the year. These findings and upper and lower thresholds are similar to research undertaken on mature street trees as reported in O'Malley (2001). Further tree water demand discussions are reported in Cameron et al (2006).

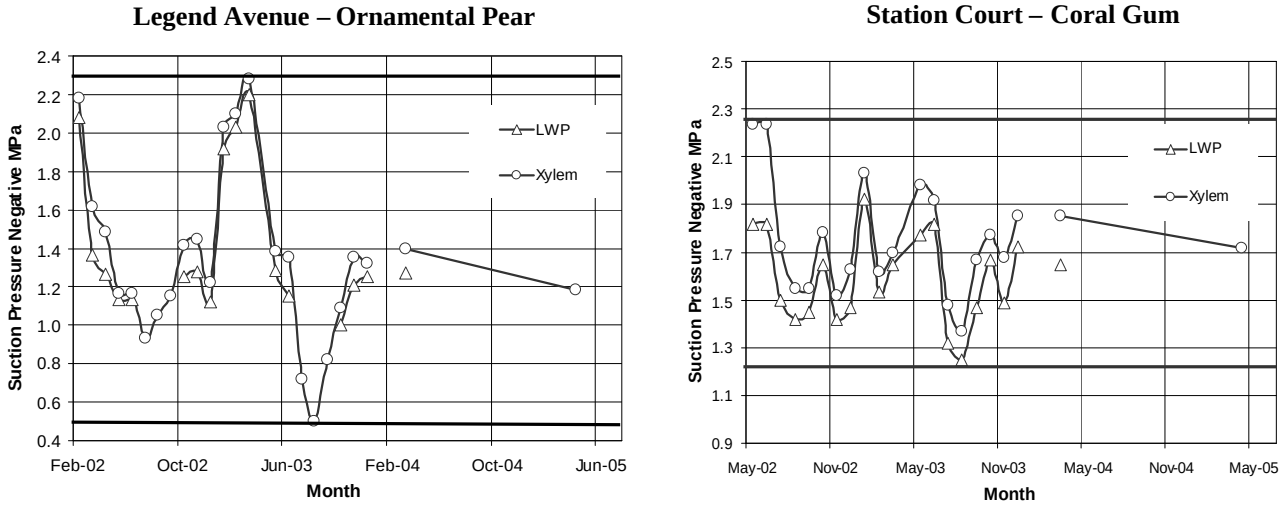


Figure 2: Exotic Species vs Native Species Water Demands with Upper and Lower Threshold Limits

## 4. RESEARCH SITE SOILS

The research sites coincided with identified extremely reactive soil profiles and >100 mm characteristic ground surface movements ($y_s$) expected over the 50 year design period. The plots on Figure 3 indicate the evident layer boundaries and reactivities for each site distinguished using the visual-tactile method and the shrink-swell values. The overall general descriptions of the natural soils of the area are high plasticity clays (CH) underlain at considerable depth by lower plasticity silty-clays (CI) or siltstone/sandstone. Shallow bedrock was encountered at two sites below the higher plasticity surface clay profile these being both Legend Avenue (4.0 m) and Dene Street (3.0 m) soil profiles. Low plasticity clays (CI) were encountered to 0.6 m in Homestead Avenue and 2.0 m in Station Court soil profiles overlying the higher plasticity clay which continued to depth. It was noted that there were minor variations between sampled borehole soil depths within each research site.

In order to assess the soil reactivity and hence site classification, four samples from selected depths from each site were tested as shrink-swell specimens. All 24 samples were classified by the Unified Soil Classification System (USCS) and visual–tactile methods. Reactivity tests (shrink-swells) were performed in accordance with AS1289.7.1.1 – 2003. Typically the shrink-swell indices from the research sites were low near the surface (1% to 2 %/pF) and considerably high at depth (a maximum of 9.5 %/pF). This difference is reflected in the soil profile changes, ranging from sandy clays at the surface to high plasticity (CH) clays at depth (to 6.0 m). It is generally accepted that 4%/pF would be regarded as a highly expansive soil, 6%/pF very highly expansive and 8%/pF an extremely expansive soil.

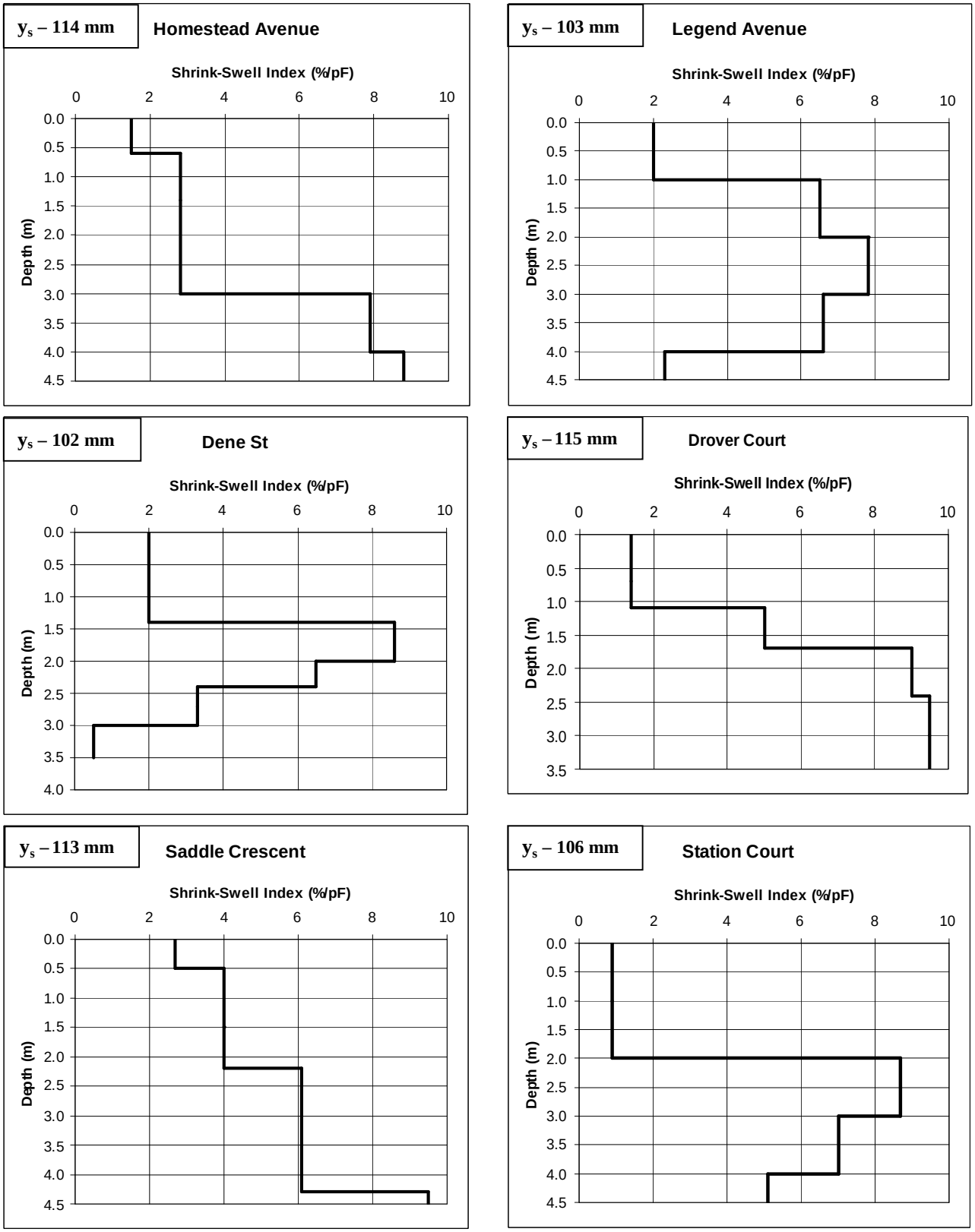


Figure 3: Site Reactivities (%/pF) vs Depth of Soil Profile and Maximum Surface Movements for Each Site

## 5. DISCUSSION OF RESULTS

From the overall analysis of the field data, there was no distinct difference in the soil moisture patterns between the "near tree" and "away from tree" profiles at any of the sites that could be directly attributed to street tree drying. This observation may be due to the immature nature of the street trees, the irrigation regime in the early stages of the development and possibly the change in land use from open paddock to a built environment. Each research site tended to have its own patterns of movement over the monitoring period, with the more heavily watered sites having the highest swelling movements, probably due to over watering of the nature strip (Homestead and Legend Avenue). Generally pavements have kept good shape despite the observed background movement.

A good correlation was found between the pressure bomb's xylem sap pressures and the Wescor in-situ leaf water potentials for the researched tree species. The xylem sap pressure approach has the distinct benefit of allowing measurement of tree water demand during periods of leaf fall as stem sections are used in the test and not necessary leaves as required by the Wescor meter for leaf water potential determination.

A comparison between the native and exotic tree species was limited by planting restrictions imposed by the developer and the community. Only one native tree species was represented in this project. Interestingly, each tree tended to have its own water demand, even trees of the same species. This may be due to a number of interacting factors, such as:

- different sourced stock
- tree locations within the subdivision
- different growth patterns
- additional watering by the developer/householder/local government authority

Preliminary indications suggest that all tree species could achieve the same level of water demand; however the water demands of both the golden raintree and the ornamental pear decreased markedly in the deciduous period, when compared to the other two tree species. This observation suggests that the golden raintree and ornamental pear may be better street trees, based solely on the annual water demand and not on ecological considerations or other factors.

## 6. SUMMARY & CONCLUSIONS

The project has established background soil moisture patterns, soil movements, house contours and water demands at the research sites. The potential influence of the trees with time was not expected to be revealed directly, it was anticipated that projections could be made on future patterns of ground movement from the preliminary data gathered during the life of the project. The extrapolated predictions may be used to compare possible future field data collection and the actual movements and patterns from the sites. The moisture patterns, soil movements and tree demands are predicted to become more pronounced as the trees mature and the sites become more established.

The information from this research can be implemented to build knowledge on native and exotic trees water demands and predict patterns of movements in the proximity of trees on reactive sites. The identification of these interactions and monitoring soil movement patterns may influence the future design and construction of road pavements, kerbs, road verges and residential housing foundations and provide guidance on the pertinent selection of species for local governments and developers.

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# AN ARTIFICIAL INTELLIGENCE EXAMINATION OF THE INFLUENCE OF GEOLOGICAL CONDITIONS AND CHANGES IN CLIMATE ON DAMAGE TO LIGHT STRUCTURES IN VICTORIA

**N.Y. Osman**
*B.Eng.( R.M.I.T)*
*M.Eng. (Swinburne)*
**D. Lopes**
*Dip. Geology ( R.M.I.T)*
*M.Eng. (Swinburne)*
*Aus I.M.M*
**K.J. McManus, AM**
*B.Eng .(Qld), M.EngSc. (Qld), PhD. (Swinburne), CPEng, F.I.E. Aust, I.E,.Aust*
*Associate Professor, Faculty of Engineering and Industrial Science*
*Swinburne University of Technology, Australia*

## ABSTRACT

Damage to light structures in the state of Victoria (South Eastern Australia) can be caused by movements of expansive soils which cover three quarters of the land mass of the state. Most of these soils are derived from Tertiary and Quaternary alkaline volcanic extrusions and lime rich sediments. The incidence of damage has been increasing, not withstanding that standard for the design of house footings have improved in the past 20 years. This paper analyses reports of increasing complaints of house damage in Victoria and particularly in the Melbourne area. There are many factors which may contribute to the movement of light structures such as vegetation, drainage, soil types, climate and others. This paper analyses the influence of geology and change in climate using Neural Network and Genetic Algorithm approaches and assesses their relative importance in contributing to the cause of the damage.

## 1. INTRODUCTION

Expansive soils are widely spread around the world, including Asia (Ohri, M. L. 2003), Australia (Osman, N. Y. et al. 2005a; Osman, N. Y. and McManus, K. J. 2005; Osman, N. Y. et al. 2005b) , Canada (Canada Mortgage and Housing Corporation (C M H C) 2001), Europe (Popa, A. et al. 2003), Middle East (Nusier and Alawneh 2002), South Africa (Wawaru, S. G. et al. 1998), South America (Camapum de Carvalho et al., 1999) and the United States (Noe, D., C 2003; Sullivan, R. A. and McClelland, B. 1969)). Expansive soils will develop wherever geological processes allow accumulation of predominantly silt and clay sized particles that contain large quantities of expansive minerals (Rollings, M. P. and Rollings, R. S. J. 1996). The unsatisfactory performance of light structures founded on expansive soils subject to seasonal movements has been frequently reported since the early 1950's (Holland, J. E. and Cameron, D. 1981).

The development of work related to climate had been done in a previous study using a different database and methodology (Osman, N. Y. et al. 2006). It showed that climate is one of the important factors and the changes had an affect on all other input parameters. Therefore, an extension of the work is done in this paper. Geology was chosen as the other input parameter due to the range of geological condition in the database. In this paper, the influence of geology and change in climate in Victoria on the damage to light structures is investigated using Neural Networks trained with Genetic Algorithms. A Neural Network is an interconnected group of artificial neurons inspired by biological nervous systems. It adapts its structure to changes based on information that flows through the network. The greatest advantage is the ability to approximate an arbitrary function by learning from observed data. It can discover complex relationships between input and output parameters or find patterns in data that cannot be handled with traditional mathematical analysis methods (Abdi, H. 2003). Genetic algorithms on the other hand are a class of search algorithms modelled on the process of natural evolution and have been shown in practice to be very effective at function optimization, efficiently searching large and complex (multimodal, discontinuous, etc.) spaces to find nearly global optima (Montana, D. 1995). The initial weights calculated by the Genetic Algorithm will be adopted to train the Neural Network. These two Artificial Intelligence techniques were chosen as they have the capacity to develop the functional relationship between input parameters and the output.

This paper gives a brief discussion about the impact of altering climatic conditions on the damage to light structures on expansive soil. The analysis of the influence of the factors of geology and change of climate is the focus of this paper.

## 2. LIGHT STRUCTURES ON EXPANSIVE SOIL

Lightly loaded buildings, such as houses, are especially vulnerable to damage because these structures are less able to suppress the differential heave of the swelling foundation soil (Canada Mortgage and Housing Corporation (C M H C) 2001). About 50% of the housing failures in Victoria investigated by Holland (1981) occurred in Quaternary basaltic clays (moderately to highly expansive), 40% occurred in the Tertiary to Ordovician clays (low to moderately expansive). Almost all the failures in the Quaternary basaltic clays were caused by edge heave or shrinking beneath the structures. Three quarters of failures which occurred in Tertiary to Ordovician clays resulted from local drying settlement caused by trees and shrubs planted too close to the structures (Holland, J. E. 1981). Highly reactive soils experience extensive volume changes related to swelling and shrinkage. When water content increases, the soil swells and when the moisture content decreases, the soil shrinks (Ferreira, R. N. et al. 1998; Ilamparuthi, K. et al. 1998). The magnitude of the ground movements associated with shrinkage/swelling phenomenon is sometimes sufficient to result in damage to light structures (Gray, C. and Frost, M. W. 2003; Padfield, C. J. and Sharrock, M. J. 1983).

## 3. CHANGE IN CLIMATE

The most common type of natural ground movement has a seasonal cycle. Seasonal movement is caused by annual cyclic drying and re-hydration of expansive soils resulting in seasonal subsidence and seasonal recovery (Biddle, G. 1998). Seasonal changes in rainfall are typically the principal cause of the change of soil moisture which leads to downward movement during a dry summer and upward movement during a wet winter. (Sorensen and Tasker (1976) cited in (Masia, M. J. et al. 2004; Freeman, T. J. et al. 1994; Smith, R. L. 1993).

Victoria, Australia has a temperate climate which varies according to the regions. Melbourne, the capital city of Victoria has generally a dry, hot summer and a wet winter (Cameron, D. A. and Walsh, P. F. 1984). The clay soil depth varies from one to three metres. The theoretical depth of ground movement in Melbourne according to AS2870 (1996) varies between 1.5 and 2.3 metres, depending upon the climatic zone. Deep-seated ground movements of greater than 3metres are possible outside Melbourne area, where semi-arid regions occur, especially in the West of Victoria (AS2870, 1996).

The climate classification of Victoria can be seen in the Thornthwaite Moisture Index map (TMI 1940-1960) (Standards Australia 1996) and the new modified TMI map (1960-1990) (McManus, K. J. et al. 2003) . It is shown that there are significant changes between the old map (1940-1960) and the new modified map (1960-1990). The positive values of the TMI indicate that the area is relatively wet while the negative values show the relatively dry areas. From the figures, it shows that the climate has changed over the years and that it is getting dryer. This results in the increase in the depth of ground movement causing an increase in the total ground movement. Thus the structures founded on expansive soils in this region suffer more damage than for which they were first designed. It has been suggested by Jayatilaka et.al (1993), cited in (Evans, R. P. et al. 1998), that regions whose TMI lies between -5 and -15 will produce the greatest seasonal vertical movement. These regions are located in the Melbourne area and areas to the West of Melbourne.

## 4. ANALYSIS

### 4.1 DATABASE

The database for this paper is based on the reports obtained from the USL Group Pty Ltd for houses in Melbourne and the West of Melbourne. The USL Group Pty Ltd is a private geotechnical company which investigates and reports on distressed residential building independently. Their reports include findings on the extent of damage, causes of distress and possible remedial measures. Other information included on the properties was the region, type of construction, presence of vegetation, change in climate, soil type and type of movements. There are approximately 250 lines in the database with approximately 40% of the data are Quaternary and 20% are Tertiary soils.

### 4.2 PROCESSES

Two Artificial Intelligence methods were adopted in this paper viz., a Neural Network trained with a Genetic Algorithm. The Neural Network and Genetic Algorithm Toolboxes for MATLAB (version 7) were used for the analysis. The Genetic Algorithm has been proven to improve the generalisation performance of the Neural Network (Osman, N. Y. et al. 2005b). In this paper, the initial weights from the Genetic Algorithm were used in the training of the networks. A two layer network structure with one hidden layer of five neurons and one output layer was designed. A Supervised Feedforward, Backpropagation network was used in the training process. Log-Sigmoid transfer functions were adopted for both the input layer -hidden layer and the hidden layer-output layer transitions. The output layer was the movement of the light structure. Seven input parameters were used in the network viz, Region (R), Construction

Wall (CW), Construction Footing (CF), Geology (G), Change in climate, Vegetation (V) and Age (A). The original network is shown in Figure **1** below.

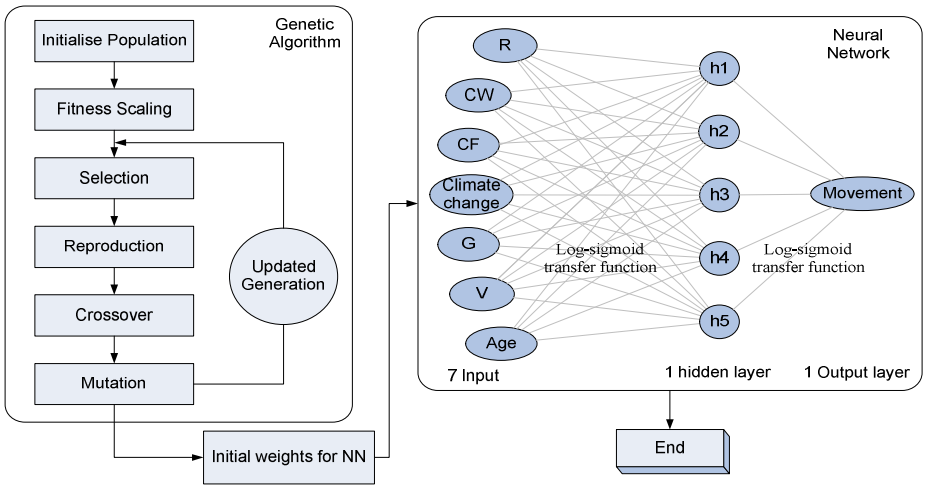



Figure 1: Neural Network trained with Genetic Algorithm for the original network

The data was divided into two sets. The first set for the training of the network used 70% of the data and another 30% is used for validation. The Training set is the data that is fed into the network so it can learn the functional relationship between input and output parameters and to provide the weights. The weights obtained are used to evaluate the results. The Validation set is used to stop the training and to avoid overfitting the network. Overfitting the network will lead to training being obsolete as the network will not learn from the data set, as it can easily lead to predictions that are far beyond the range of the training data. Overfitting can also produce poor generalisation (Karystinos, G. N. and Pados, D. A. 2000) and wild predictions and even with noise- free data

Three networks were used in this paper. The Original network (ON) with seven input parameters, a pruned geology network (PGN) with geology being removed or "pruned" and a pruned change climate network (PCCN) where change in climate was removed. All the networks were trained using the initial weights produced from the Genetic Algorithm. To assess the impact of the input parameters with the other parameters, Connection Weights Analysis (CWA) was used. The CWA calculates the product of the raw input-hidden and hidden-output connection weights between each input neuron and output neuron and sums the products across all hidden neurons (Olden, J., D and Jackson, D., A. 2002).

The Weights for all input parameters that were obtained from the training were evaluated using CWA. Since the resulting weights can be positive and negative, the weights were normalised. A constant value was added to all weights to shift these values in the positive range. For example, if the connection weight of an input has the value of -4.3, a constant value of 5 was added to shift the value in the positive range. These values were than converted into a percentage value.

### 4.3 PRUNING METHODS

Input Pruning involves removing a particular input parameter in order to check its relevance in the network in regards to the other input parameters. The input parameters with the least sensitivity to the network can be removed as the input parameter may be redundant or does not affect the other input parameters. "When one or more of the inputs have relatively small sensitivity in comparison to others, the dimension of neural network can be reduced by dropping them…" (Zurada, J. M. 1994).

Two sets of networks with the same number of input parameters were used in this paper to determine the significant of the pruned input parameters with the other parameters. These networks were compared with the original network with all the input parameters present.

(1) PGN

This network contains six input parameters which are Region, Construction Wall, Construction Footing, Change in climate, Vegetation and Age. The change in climate in this case is equal to the change in Thornthwaite Moisture Index (1940-1960) and Thornthwaite Moisture Index (1960-1990). A negative value of Change in climate indicates that the climate is getting dry. A positive indicates that it is getting wet. Zero indicates that no change has occurred

(2) PCCN

There are six input parameters; Region, Construction Wall, Construction Footing, Geology, Vegetation and Age. The networks were trained using the initial weights from the genetic algorithm. The results are shown below.

# 5. RESULTS AND DISCUSSION

## 5.1 PERFORMANCE ANALYSIS BASED ON MEAN SQUARED ERROR

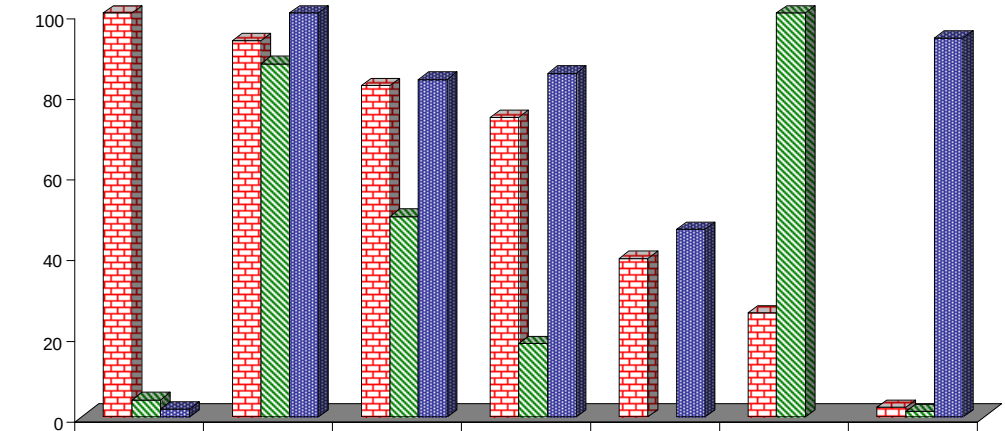
Table 1 shows the performance of the entire networks. From the table, it can be seen that the original network performs to the lowest standard as the Mean Squared Error (MSE) obtained after training the network is the largest. The PGN showed the best performance as the MSE obtained after training the network gave the smallest values of the three networks. The performance of the PGN and PCCN improved by 10% and 8% respectively compared with the performance of the original network. This indicates that the pruned input parameters influenced the network performance. However, the results are not conclusive and hence a Sensitivity Analysis was made.

| Network | Mean Squared Error |
|---|---|
| *Original (ON)* | 0.039 |
| *Pruned Geology (PGN)* | 0.035 |
| *Pruned Change Climate (PCCN)* | 0.036 |

Table 1: Performance of three networks

## 5.2 SENSITIVITY ANALYSIS BASED ON CONNECTION WEIGHTS ANALYSIS

A validation of results in terms of the sensitivity of input parameters when the network has been altered has to be evaluated. Therefore, a Sensitivity Analysis of each individual parameter was made using CWA.




| | Vegetation | Year built | Region | Construction Wall | Geology | Climate Change | Construction Footing |
|---|---|---|---|---|---|---|---|
| ON | 100 | 93 | 82 | 74 | 39 | 26 | 2 |
| PGN | 4 | 87 | 50 | 18 | | 100 | 1 |
| PCCN | 2 | 100 | 84 | 85 | 46 | | 94 |

Figure 2: Sensitivity (%) of the input parameters using Connection Weights Analysis

Figure 2 shows the sensitivity of the individual input parameters using CWA for all the networks. The purpose of the evaluation was to observe the consistency and the sensitivity of the individual parameters when the network is changed by pruning. If the results of the percentages of the CWA were consistent, then the input parameters do not have any affect when changes to the original network were made. This also indicates that the input parameter is not sensitive to the changes. From Figure 2, comparing the sensitivities of ON and PGN and ON and PCCN, there are two input parameters that clearly do not seem to show that the change in the network affects the percentages of the CWA to any great extent. These are the input parameters "Year Built" and "Region" where the percentages of the two input parameters are consistent (especially Year Built). This showed that these two input parameters are not as important as the others, as they are the least sensitive to the changes.

It is obvious, from the column chart, that the factors of Vegetation, Construction Footing and Climate Change have a dramatic affect when the original network was modified. The changes obtained from Geology when the original

network was modified were small. This demonstrates that the changes affect Geology only slightly. It is clear that when the original network was modified, as in this case where two input parameters were pruned independently, the impact of the other input parameters in the network was influenced by the changes. However, as can be seen, not all the input parameters were affected, but mainly Year built and Region. By pruning the input parameters, it enables the evaluation of the results of CWA. The pruned geology network seemed to show a more dramatic affect than the pruned change in climate network. This is because when geology was pruned, the majority of the percentages of the input parameters increased (climate change) and decreased (Vegetation, Year built, Region, Construction wall and Construction Footing) to a great extent compared with the percentages in the original network. On the other hand, from the analysis of the connection weights of the input parameters in the PCCN it appears that they have almost consistent percentages when compared with the original network, except for Vegetation and Construction Footing. Nevertheless, it affected the network even though not to the same extent as for the PGN.

## 6. CONCLUSION

Using a Neural Network trained with a Genetic Algorithm to analyze this database, geology and change in climate are significant factors in influencing the movement of light structure due to soil movement. This is evident in the results obtained when original network was compared with the two pruned networks. From the two input parameters pruned, it can be concluded that the impact of the other input parameters when Geology was pruned was more than when change in climate was pruned. The PGN showed the best performance, as the Mean Squared Error obtained after training the network gave the smallest values compared to the ON and PCCN. Most of the input parameters when geology was pruned are sensitive towards the change in the network from having 7 inputs to 6 inputs (minus Geology). This is true except for the factors: Year built and Region.

Nonetheless, most of the input parameters, when change in climate was pruned, were also sensitive to the changes. The performance of the PGN and PCCN improved by 10% and 8% respectively compared to the performance of the ON. This indicates that the pruned input parameters influenced the network performance. It is clear that when the original network was modified, as in this case where two input parameters were pruned independently, the impact of the other input parameters in the network were influenced by the changes. By pruning the input parameters, it enabled the evaluation of the results of the CWA.

In conclusion, except for Year built factor and Region factor which were less sensitive to the change the in network, the rest of the input parameters are affected when one or more of the input parameters were removed or pruned. In this paper, it was demonstrated, for this database, that geology, as well as changes in climate, has the greatest influence on damage to light structure due to movement with geology being more important.

Future work on examining other input parameters in this database should be done to determine which input parameters could have the highest impact or could be more sensitive to the changes to the network.

## 7. ACKNOWLEDGEMENT

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# PREDICTION OF FILL INDUCED SETTLEMENTS IN PEAT DEPOSITS, TAURANGA, NEW ZEALAND

**Matthew Packard**
*Foundation Engineering Consultants Limited*
*Tauranga*

## ABSTRACT

Fill induced consolidation settlements over geologically recent (Holocene) New Zealand peat deposits are often difficult to determine using conventional prediction techniques, due largely to the high and often variable organic content of such deposits, sampling difficulties and scale effects. This can lead to earthworks and building design uncertainties when development occurs over such areas.

A trial fill embankment was constructed at the completion of the investigation phase for the Tauriko Industrial Estate, Tauranga. Observed settlements beneath the embankment were monitored using surface settlement markers and hydraulic profilometers and results were compared with conventional prediction techniques from investigation data. This paper provides a review of the prediction techniques used, their relative levels of accuracy with respect to measured initial construction settlements and the results of ongoing monitoring during the construction phase of the development.

## 1 INTRODUCTION

The Tauriko Business Estate forms a large and irregularly shaped piece of land located some 10 kilometres south-west of central Tauranga City, Bay of Plenty, New Zealand. Earthworks proposals include up to 4 metres of fill across the low lying areas of the development to facilitate the construction of roads, services and light industrial and commercial buildings.

As part of pre-purchase due diligence assessments, IMF Backstop Limited engaged Foundation Engineering Consultants Limited to undertake a staged comprehensive geotechnical investigation of the land to determine a site-specific geological model and to establish realistic guidelines and parameters for the development of the site. In addition to conventional settlement predictions based on results of the field and laboratory assessments, a trial fill embankment was constructed in order to gauge likely rates of settlement within identified peat deposits over the lower lying portion of the site under proposed fill loads. Foundation Engineering Consultants Limited, 2005 (Geotechnical Investigation Report on Tauriko Business Estate – Stage 1).

## 2 GEOLOGICAL MODEL

The general landform across the Tauriko locality is reported by Briggs et al., 1996 (Geology of the Tauranga Area) to comprise a series of elevated Pleistocene-aged ignimbrite terrace deposits. Due to the unwelded and pumiceous nature of the ignimbrite, it is prone to erosion resulting in the formation of deeply incised gully systems. Holocene deposits within the base of these gullies typically comprise silts, sands and gravels with interspersed peat deposits. Groundwater levels are generally very shallow being located near the present ground surface.

The subject site is contained within the Kopurererua valley system, formed through the processes described above.

On account of the presence of broad areas of peat identified within drainage trenches during the initial site walkover, the site was suited to investigations by cone penetration testing (CPT), which provides a continuous in situ ground profile. Information from test pits, hand auger and machine holes also undertaken as part of the site investigation was reviewed to 'ground truth' the inferred soil profile encountered during the CPT investigation soundings. Push tube samples were obtained for laboratory consolidation testing to provide additional estimates of engineering characteristics for the peat.

From the investigation findings, a typical subsurface model was determined for the valley floor geological environment and is summarised as follows:

- A firm to stiff crust up to 0.4 metres thick, comprising topsoil and consolidated inorganic soils
- Predominately peat deposits to an average depth of 6 metres, which contained abundant fibrous inclusions including roots and wood that have only undergone relatively minor amounts of decomposition and are therefore considered to be relatively fresh. Within the peat deposits are rare lenses of sands and silts.
- Dense becoming very dense sands were encountered beneath the peat deposits to the maximum extent of the investigation
- Groundwater was typically encountered at a depth of 0.4 metres below the ground surface.

Typical CPT plots for the valley floor geological environment are illustrated in Figure 1:

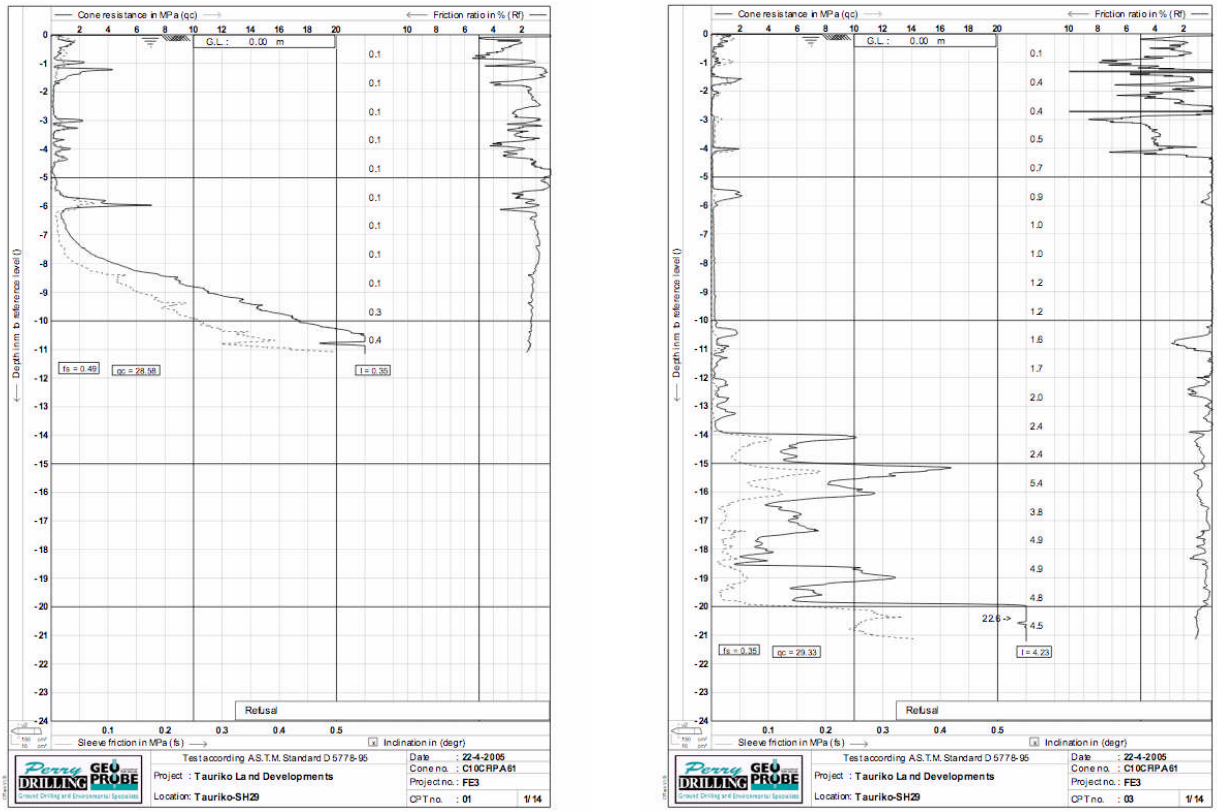


Figure 1 : Typical Cone Penetration Test Plots

# 3 CONVENTIONAL SETTLEMENT PREDICTION TECHNIQUES

Initial broad range predictions of fill induced settlements were attempted using two well known techniques from available laboratory and field data. These methods are summarised in the following sub-sections.

## 3.1 SCHMERTMANN'S METHOD

Each of the cone penetrometer records were divided into a succession of layers with constant cone resistances ($q_C$). Total settlement was calculated from the sum of the settlements of each layer based on layer thickness (z) and cone resistance following the method of Schmertmann et al., 1978 (Improved Strain Influence Factor Diagrams).

A sensitivity analysis of the results was undertaken by adjusting the embankment dimensions to achieve maximum settlements beneath an applied fill load of 5 metres, which represents 3.5 metres of structural filling plus an additional 1.5 metres of preload filling. These analyses indicated likely settlements ranged from 600 mm to 1900 mm.

### 3.2 TERZAGHI METHOD

Settlement predictions were also modelled using the Terzaghi One-dimensional Consolidation Theory as outlined in Terzaghi, K. (1943), (Theoretical Soil Mechanics). By using a compression index ($C_C$) of 4.8 and a preconsolidation voids ratio ($e_O$) of 13 as determined from the laboratory consolidation testing, the predicted settlements ranged from 1800 mm to 2600 mm.

The very high compression index suggests that these soils are sensitive to applied loads. Due to the known variability within the peat deposits, settlement estimates may be subjected to sampling and scale effects and because of this, it is not known if the values determined from laboratory testing are representative.

## 4 FIELD TRIAL EMBANKMENT

Due to known difficulties with predicting settlement in peat soils using conventional techniques, a trial fill embankment was constructed over the valley floor to observe actual settlements. The embankment footprint was 37 metres square with a height of 5 metres. The sides of the embankment were battered to approximately 40 degrees above the horizontal.

Prior to construction of the embankment, two profilometers and a series of plate markers were installed over the natural ground surface to monitor vertical displacements. Due to the poor quality of the natural ground, construction of the initially 3.5 metre high embankment was undertaken in two phases to reduce the likelihood of a bearing capacity failure within the peat deposits. Several months following this, the embankment was loaded with an additional 1.5 metres depth of soil to simulate a preload situation. Notable settlement trends, as determined by measured deflections of the profilometers beneath the centre of the embankment, are illustrated in Figure 2:

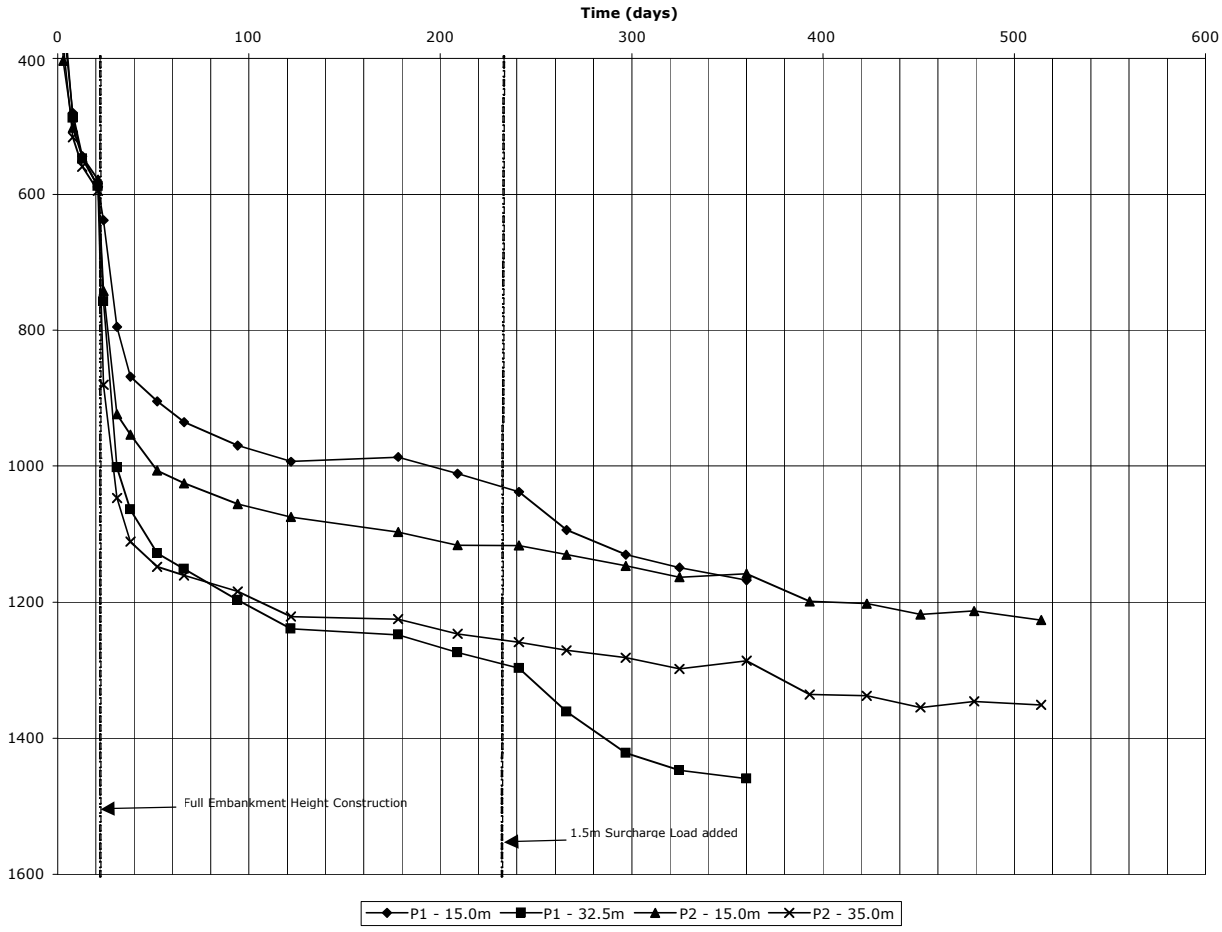


Figure 2 : Total Settlement versus Time

A summary of settlements estimated from conventional techniques and measured from the trial fill embankment is presented in Table 1:

| Summary of Settlements | |
|---|---|
| **Assessment Method** | **Settlement (mm)** |
| Schmertmann | 600 – 1900 |
| Terzaghi | 1800 – 2600 |
| Trial Embankment | 1200 – 1500 |

Table 1 : Summary of settlement predictions derived from the methods used.

As shown in Table 1, a broad range of results were obtained by using conventional settlement prediction methods when compared to measured settlements beneath the embankment. Therefore, the embankment settlement data was back analysed to determine the compression coefficient that best modelled settlements under the embankment. This could then be applied to proposed fill loads and for planning cut to fill earthworks models.

The back analysis indicated that a $C_C$ value of 1.12 was required to achieve settlements similar to those observed beneath the embankment. This value is much lower than the value of 4.8 as determined from laboratory analysis and it is likely that given the known variable nature of these soils, the push tube sample taken for the laboratory consolidation testing may not be representative of the peat soil profile. Disturbance effects associated with the sample collection and processing are also very likely.

## 5 CONSTRUCTION MONITORING

Prior to commencement of earthworks, a series of plate markers were installed over the lower lying portion of the site for the purpose of ongoing settlement monitoring. During the first season of earthworks, these areas were filled to depths of between 1 and 9 metres, inducing a wide range of settlements from 100mm to 1400mm. This relationship is illustrated in Figure 3.

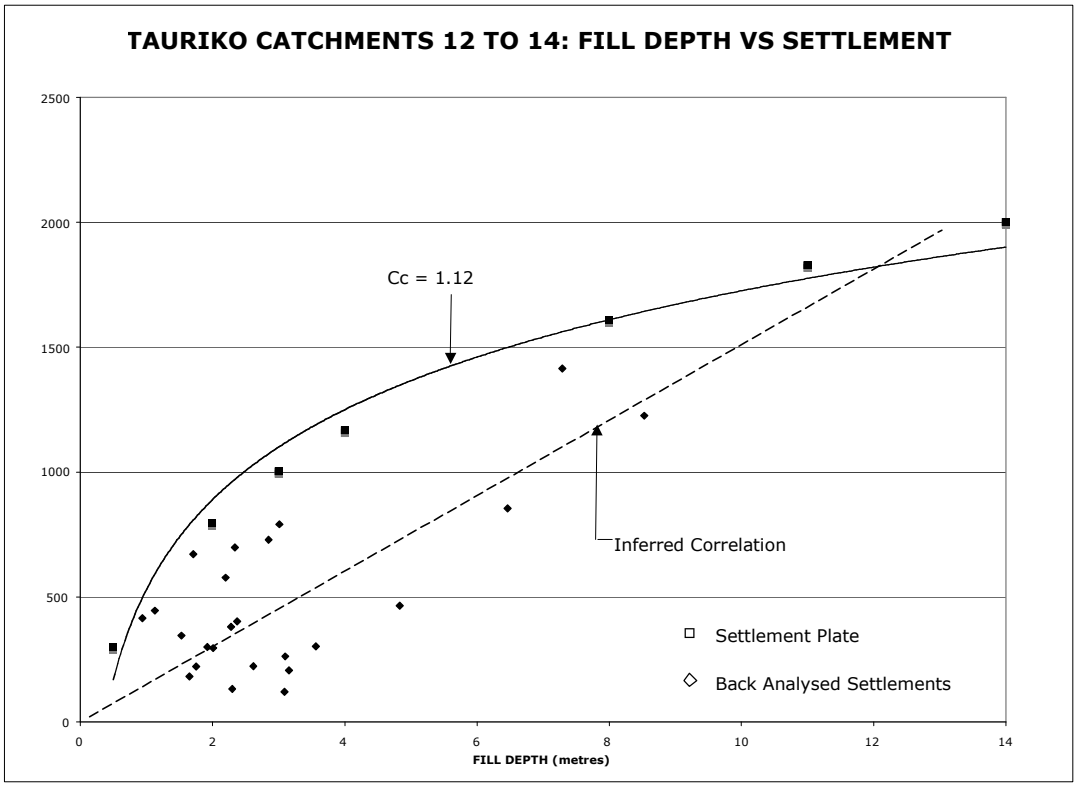


Figure 3 : Relationship between Fill Depth and Settlement

Figure 3 shows how variable the relationship is between the depth of fill and resulting settlements at each of the settlement plate locations, which suggests variable ground conditions. By using the back analysed compression coefficient a plot of predicted settlements on the basis of applied fill loads has also been shown, which indicates greater settlements than those observed at the settlement plate locations.

This could be due to the variable nature of the peat deposits, punching of the embankment into the underlying peat, embankment bearing capacity failure and/or settlements of the peat during the embankment construction which would lead to a greater than anticipated fill thickness and associated surcharge load.

The dashed line of best fit was drawn to provide preliminary earthworks design and volumes only.

## 6 CONCLUSION

The prediction of settlements within peat deposits using conventional methods is difficult and can result in a wide range of values. Further, observed settlements with respect to applied fill loads at each plate location do not appear to fit a distinctive trend, indicating an extreme lateral and depth variability of the peat. Difficulties with sample representativeness and disturbance effects can also affect the end result. On this basis, an observational and design approach should be adopted to account for such variances.

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# LANDSLIP ASSESSMENT AND REMEDIAL WORKS FOR THE NEW ZEALAND EARTHQUAKE COMMISSION

**Nick Peters**
*Tonkin & Taylor Ltd*
*Wellington, New Zealand*

## ABSTRACT

Over the last 2.5 years, Tonkin & Taylor's Wellington office has inspected nearly 500 claims for the New Zealand Government's Earthquake Commission (EQC). EQC is New Zealand's primary provider of natural disaster insurance to residential property owners. It insures against damage caused by earthquakes, landslips, storms, floods and other natural disasters as defined in the EQC Act.

Of the 500 inspections, 90% were for landslips. Of these landslip claims, approximately 25% have posed a threat to undermine or inundate private dwellings. In these cases Tonkin & Taylor has designed and monitored the construction of the remedial works. Remedial works on landslips may include removal of landslip debris (from $2m^3$ to $2000m^3$), slope regrading and scaling, installation of slope drainage, hydroseeding, construction of retention works including timber pole, concrete and timber crib, mass block and sprayed concrete walls, rock anchors, micropiles and tie-backs.

Three case studies of landslips and associated remedial works are presented and discussed. Each case study had both emergency and long term remedial works implemented. The emergency works removed the immediate risk of expensive damage to the dwelling. The long term works have removed the imminent risk threat and where practical have reinstated the land that had been lost or damaged.

## 1. INTRODUCTION

The New Zealand Government's Earthquake Commission (EQC) engages Tonkin & Taylor (T&T) countrywide to inspect and report on natural disasters causing damage to residential properties.

When T&T are engaged for a natural disaster claim, T&T will inspect the property and prepare a report providing information on the natural disaster's description, the damage it has caused to the property and the likely hazards and risks it may present to the property in the next 12 months (known as the imminent risk threat). T&T will also scope remedial works and their indicative costings (including any emergency works where necessary).

Since February 2004, T&T Wellington has inspected nearly 500 natural disaster damage claims within the Wellington region. 90% of these claims are for natural landslip damage. Over 70 of these claims have had remedial measures designed and construction monitored by T&T.

This paper presents a summary of the EQC Act, Wellington's geology, slope instability mechanisms, identification of hazards and risks during site inspections, and three case studies of landslips and their associated remedial works.

## 2. EQC INSURANCE COVERAGE

EQC provides an insurance cover for natural disasters affecting residential properties. Examples of natural disasters include natural landslip (landslides, rock slides, rockfalls, debris flows); earthquakes, volcanic eruptions, hydrothermal activity, tsunami, storm or flood damage to land or fire caused by any of the above.

The EQC Act 1993 defines what parts of the property are covered, conditions of insurance, its powers and regulations. On any residential property, EQC provides coverage to:

- The dwelling or appurtenant structure (i.e., garage, shed, deck, etc.);
- Land within 8m of the dwelling or appurtenant structure;
- Land supporting the main access way within 60m of the dwelling;
- Land and structures considered to be at imminent risk as a result of the event that has occurred.

Understanding the EQC Act is important to the engineer or geologist when the landslip is inspected. The value of compensation the owner will receive from the landslip event is dependent upon the areas that are considered to have been lost, damaged or are at imminent risk. The cost of the remedial works may be more or less than the value of

damage that the landslip has, or could cause. If a claim is accepted, EQC can either pay the amount of the damage or they can undertake remedial work, although EQC's preference is for the former.

# 3. WELLINGTON'S GEOLOGY & GEOMORPHOLOGY

The principal geological materials encountered on hillsides in Wellington are greywacke rock, surficial soils and fill.

**Basement Rocks**: The Wellington area is underlain by interbedded sandstone and argillite known as "greywacke". The rocks are typically closely fractured and folded and range from very strong (fresh) to weak (completely weathered). As they weather, failures may occur preferentially through the weakened rock mass along conjugate joint surfaces.

**Surficial Soils – Residual soils / Colluvium / Loess**: The surficial soils range from a loose to dense sandy silty GRAVEL, to a stiff to hard clayey SILT. These soils are typically about 1-2m thick, but can be up to 5m thick, particularly at the base of slopes. Colluvium around Wellington is primarily formed by previous slope failure (i.e., earth flows or soil creep).

**Engineered/Non-engineered Fill**: Fills are usually derived from surface soils and weathered bedrock. In practice, many fills (particularly on older sites) end up being a jumbled mass of non engineered, end-tipped material. In these circumstances, fill is prone to settlement or landslip.

Wellington is located in a seismically active area. The area is subject to rapid tectonic uplift and erosion, resulting in steep (25° -55° from the horizontal), deeply dissected greywacke terrain with thin, marginally stable soils.

# 4. TRIGGERS & MECHANISMS OF SLOPE INSTABILITY

In the Wellington Region, the typical triggers and mechanisms of slope instability are as follows.

## 4.1 TRIGGERS

**Rainfall:** Severe storms with heavy rainfall periodically occur in the Wellington Region (eg. 220mm over 3 days in February 2004). The intense rainfall causes large amounts of run-off and surface soil saturation.

**Earthquakes:** The effect of strong ground-shaking can cause significant landslips. No significant earthquake induced landslips have occurred in the Wellington Region in the last 50 years, but historically they have.

## 4.2 MECHANISMS

In summary, the typical landslip mechanisms affecting Wellington slopes are:

(1) Translational Slides and Debris / Earth Flows: Surficial soil/fill sliding over bedrock. Slides are typically tens of metres long and volumes can be tens to hundreds of cubic metres. Typically they are less than 2.0m in thickness.

(2) Shallow Rockfalls or Slides: Typically in steeply cut (45°+ from the horizontal) rock faces, the closely jointed and faulted greywacke weathers, resulting in relaxation and slippage along defects (i.e., joints or bedding planes). This typically produces planar and wedge sliding or toppling failures.

(3) Deeper Seated Rock Slides: Larger scale instability occurring along significant faults or bedding planes.

# 5. HAZARD & RISK

## 5.1 HAZARD & RISK IDENTIFICATION DURING THE SITE INSPECTION

When an engineer / geologist inspects a landslip, they need to:

- Define the geological setting, model, material types and the landslip model, identifying the key components (i.e., headscarp, failure plane, effects of groundwater and surface water, landslip toe or run-out);
- Understand the triggers / mechanisms causing the landslip (i.e., soil softening, damaged services, vegetation clearance etc). It is important to look beyond the site to see what could happen in the future;
- Confirm the extent of damage caused to the house, structures and land;
- Identify the risks to the house and land, arising out of the landslip that has occurred.

To do this they may need to undertake multiple inspections to build a better picture of the landslip / geological model or get a second opinion from colleagues. It is also useful to look at similar landslips to see how they behave after the initial event. When the engineer / geologist has made a full appraisal of the landslip, the hazards and likely risks, it is then possible to consider options for remediation.

### 5.2 DEFINING REMEDIAL WORKS BASED UPON HAZARD & RISK

For each landslip assessment the engineer / geologist needs to decide the most appropriate remedial option for the property. The preferred option should be designed (ideally) to meet the requirements of the EQC Act, namely:

- Stabilise the landslip that has occurred and reinstate the land that has been lost or damaged;
- Remove the threat of further damage occurring in the next 12 months from the landslip that has occurred;
- Remove the dwelling if the threat cannot be economically mitigated.

Remedial works on any property can be undertaken as either emergency works or as a long-term solution:

- Emergency works are usually to provide a quick solution to mitigate an immediate risk to the property. They can form part of the long-term remedial measures. They are usually arranged immediately after the Engineer's inspection and typically undertaken when the cost of the works is going to be substantially less than the predicted cost of further damage if nothing is done.
- Long-term solutions may not need to be implemented immediately. They are subject to detailed analysis and achieve normally expected requirements for stability. They should include reinstatement of the lost land. The works are generally more closely specified but not necessarily more costly than the emergency solution. They are used to supplement or replace emergency works (if these have been carried out).

Deciding the most suitable remedial option for any landslip is done from assessing a number of factors or constraints. These may be:

- Cost of remedial works (i.e., is the cost less than the value of damage that has occurred or could occur);
- Ground conditions (i.e., conditions may suit a particular type of option);
- Availability of suitable or specialist contractors;
- Speed/urgency of getting the works in place;
- Feasibility of controlling stormwater and groundwater;
- Access to the area of instability may be restricted for some types of equipment.

## 6. CASE STUDIES OF LANDSLIP ASSESSMENTS & REMEDIAL WORKS

### 6.1 INTRODUCTION

The following three case studies of landslip assessments and remedial works involved emergency and permanent works to remove the high risk of serious (and expensive) damage to dwellings.

**Case Study 1- Stokes Valley**

*Landslip Assessment*

| | |
|---|---|
| Landslip Type: | Translational slide - Earthflow in soil (residual soil and colluvium). |
| Original Slope Profile: | 30° to horizontal, vegetated gully covered with gorse, manukas and ferns above rear of house. The house is located on a cut / fill platform across the gully. |
| Landslip Description: | - Approximately 30m long, 5m wide and 1 to 2m deep.<br>- Tension cracking extending 10m uphill from headscarp with vertical offsets of 1 to 2m.<br>- Debris comprises saturated earth and vegetation. |
| Damage to property: | Debris has inundated land and has flowed around the house, building up against rear of house by a depth of 1.2m, causing some cosmetic damage. It has also demolished and buried part of a concrete crib wall at the side of the house. |
| Cause: | Heavy rainfall and earthworks on upslope property directed additional stormwater into the gully above the landslip. |
| Imminent Risks: | Further inundation of land and build-up against house causing extensive damage and possible partial collapse to rear of house resulting in potential damage of approximately $40,000. |

*Emergency Remedial Works*

Following our site inspection and assessment of the cause, damage, hazard and risks, we installed movement monitoring pegs to check if any further slope movement was occurring and recommended the following emergency works to control the stormwater (trigger) and reduce the future threats to the dwelling:

1. Minimise water infiltration into the ground, and run-off onto the landslip, by deepening a drain on the access road to the upslope property to divert water past the head of the gully. Earthworks on the upslope property were identified as a contributing trigger to the landslip. Also construct a stormwater intake upslope of the tension cracks and pipe the water via a sealed pipe to below the toe of the landslip debris.
2. Remove soil against rear wall of house, taking care not to remove too much to remobilise the landslip debris.
3. Provide a toe buttress to the landslip by installing an 8m long, 2m high mass concrete block wall. This is also to act as deflector of any future debris around the house and down the gully. A mass concrete block wall was chosen because the blocks were readily available, easy and quick to install (i.e., didn't require specialist contractors). To gain access to do these works, a car port had to be demolished.
4. Regrade slope behind the mass block wall to a shallower / stable angle with hydroseeding of all bare areas.

*Permanent Works*

Following review of the movement monitoring data, it was found that creep movement was occurring on the east margin that was not stabilised by the mass block wall (below shed in Photograph 1). As a result, long term remedial measures to prevent further damage and loss were implemented and incorporated into the emergency works. These involved:

- Installing additional blocks on the eastern side of the mass block wall with compacted backfill and drainage;
- Clearing all drains to ensure unrestricted stormwater flow.
- Installing a geogrid reinforced slope upslope of the wall to buttress land below the shed.

Photograph 1 Mass concrete block wall

A critical part of these works was to ensure that, when excavation of the mass block wall foundations was occurring, the excavation, block placement and backfilling took place on the same day to prevent landslip remobilisation. In conjunction, the local Council upgraded the drains adjacent to the house and on the upslope property.

It was important to be able to make on site decisions and change the design of the works to meet the encountered ground conditions. It was also important to change the design without adversely affecting the existing works. Following completion of the engineering remedial works, damage to the house was repaired and the car port was rebuilt.

**Case Study 2 –Eastbourne**

*Landslip Assessment*

| | |
|---|---|
| Landslip Type: | Rapid earthslide/flow in fill and colluvium overlying highly weathered greywacke. |
| Original Slope Profile: | 45° to horizontal, partly vegetated slope, 10m high with house at the crest. |
| Landslip Description: | - Approximately 25m long, 6m wide and 1.0m deep with debris comprising wet silty gravels.<br>- Tension cracking on either side of headscarp with vertical displacement of up to 400mm. |
| Damage to property: | - Undermining of concrete driveway by up to 400mm.<br>- Undermining of two house foundation poles and deck foundation pole displaced away from the deck bearers.<br>- Loss of land below the house and debris accumulation on neighbouring property. |
| Cause: | Heavy rainfall causing softening of the fill and colluvial soil on the slope. |
| Imminent Risks: | - Further undermining of driveway, making it unusable.<br>- Further loss of land on the slope and inundation of the neighbouring property. |

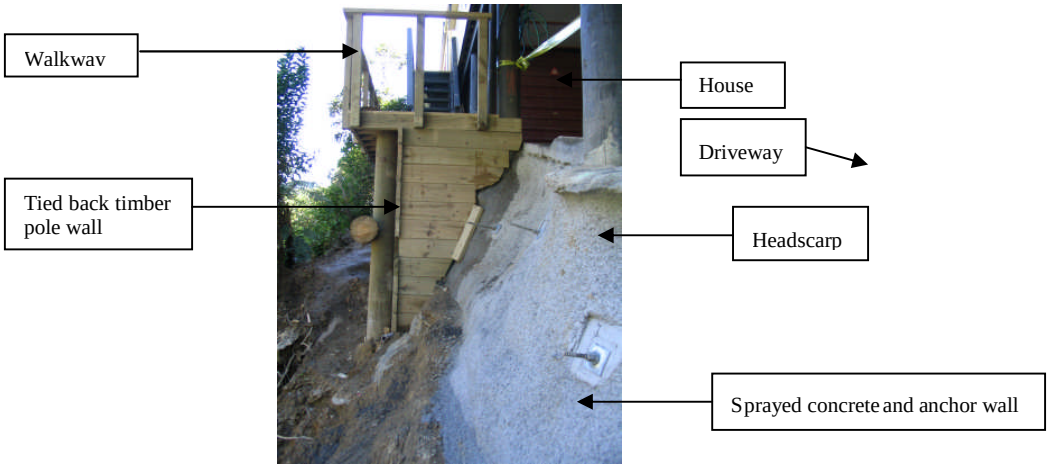
- Further undermining of house foundations causing structural damage (>$100,000).

*Emergency Works*

Following our inspection, we implemented emergency remedial works to limit the risks to the house structure and the driveway. These would retain the unstable headscarp below the house and maintain safe access along the driveway.

The emergency works comprised:

- A sprayed concrete wall with tensioned anchors extending into rock across the landslip headscarp area;
- A cantilevered timber pole wall along the area of tension cracking to the east of the landslip;
- A timber pole wall to the west of the landslip under the house with tie back anchors.




Photograph 2 Sprayed concrete and tied back timber pole wall.

A sprayed concrete wall was chosen over a timber pole wall across the landslip area because the slope profile was too steep and the soil depth too great to allow adequate pole embedment. The sprayed concrete wall included anchors embedded 4.0m into competent bedrock. The anchors were tensioned to minimise relaxation of the ground and cracking of the concrete driveway slab.

The timber pole wall designs were based upon the results of Scala penetrometer testing undertaken following our initial inspection. The pole wall was designed to retain the soils behind and allow for the loadings of the driveway and the house. As the access was difficult around the house, it was not possible to install a conventional timber pole wall. The wall under the house comprised 200 SED poles embedded 1.5m into rock with tied-back anchors and walers.

*Permanent Works*

The emergency works provided adequate stability to the house and driveway and no additional slope stabilisation works were considered necessary. Permanent works on the property included removal and reshaping of debris from the toe of the slope, hydroseeding of the slip face, construction of a safety barrier and reinstatement of a timber walkway.

**Case Study 3- Newlands**

*Landslip Assessment*

Landslip Type: Creeping Translational rock slide in highly to slightly weathered greywacke, a complex planar and wedge failure along multiple slickensided silt / clay infilled defects.

Original Slope Profile: 30m long, 45° to horizontal, bush covered cut slope with houses at the bottom.

Landslip Description:
- Approximately 30m long, 30m wide and up to 5.0m deep.
- Tension cracking above headscarp with up to 200mm vertical displacement. Overthrusting and en-echelon tears on flanks. Toe bulging along fault plane at bottom of slope.
- Movement of landslip area of up to 32mm per day.

Damage to property: Movement affected four properties, with the worst damage involving:
- Bowing of rear wall of one house by up to 75mm;
- Deformation of timber walls, block walls, steps and decking;
- Widespread cracking and dilation of rock on slope.

Cause: Combined heavy rainfall, steepening of slopes from sub division and house construction earthworks and adversely orientated defects.

Imminent Risks: Enlargement of the landslip and continued creep or rapid movement, causing extensive structural damage to two houses (>$100,000 to each) and lesser damage to two others.

*Emergency Works*

Immediately after our inspection, we implemented emergency remedial works to limit the risk of damage to the houses. Our emergency works comprised:

- Excavation of trenches and removal of soil/rock from behind back walls of two houses;
- Covering and sealing cracking;
- Dismantling a block wall and timber retaining wall to remove loading on back wall of house.

Approximately three weeks after these works were initiated, further movement was reported. An inspection showed that creep was increasing and the landslip had enlarged. A second set of emergency works were initiated including:

- Installation of temporary fences and concrete blocks to protect the dwellings from rockfall off the upper slope;
- Installing movement monitoring pins across landslip and undertaking of a topographic survey;
- Excavation of approximately 2000m$^3$ of soil and rock from the top of the slope to reduce the driving mass

The excavation of the soil and rock caused the rate of creep to slow down and stop. The rate of movement was regularly monitored to determine when the final emergency works could be completed. These works comprised the installation of tensioned rock anchors to support the toe of the slope and bored drains to relieve groundwater pressure.

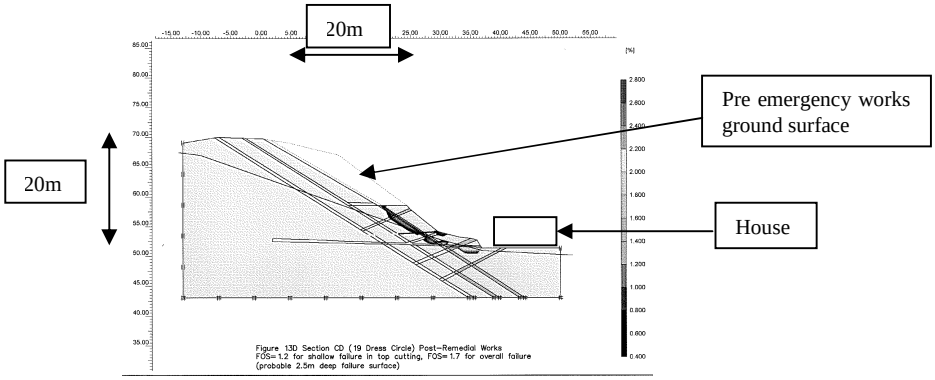



Figure 1 Plaxis modelling of remedial works

Following completion, house repairs were undertaken and the block walls, decking, steps and lawns were reinstated.

## 7. CONCLUSIONS

All the described landslips were triggered after a single large storm event, but the mechanisms of failure all varied. Being able to correctly define the geological setting, the type of mechanism and the cause of the instability, meant that the engineer / geologist could implement the most appropriate remedial measure to address the hazards and risks.

The emergency works were usually finished within one week of our first inspection. They showed how the mechanism could be identified, remedial works designed, contractors mobilised and works completed in a short time period.

The long-term works were adapted to suit the sites and were revised as works progressed to suit the ground conditions exposed and changes to the landslip. For Case Studies 1 and 3, the remedial works had to be increased from those originally planned because monitoring and observations showed that the landslips were reactivating or increasing in size. Access restrictions on both these sites meant that some building structures on the property had to be removed to install the necessary measures. All the properties required more than one type of remedial measure to address the hazards and risks.

## 8. REFERENCES AND ACKNOWLEDGEMENTS

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I would like to thank the Earthquake Commission for their permission in letting me use these case studies and I would also like to thank T&T for supporting me in preparing this paper and attending the 7th YGPC.

# ENGINEERING ISSUES RELATING TO FILLING OF DEEP PITS FOR RESIDENTIAL AND LIGHT INDUSTRIAL DEVELOPMENT

**D. Piccolo**
*Senior Geotechnical Engineer, Pells Sullivan Meynink Pty Ltd*

**G. Mostyn**
*Principal, Pells Sullivan Meynink Pty Ltd*

## ABSTRACT

This paper identifies the post end of filling surface settlement profile as the main issue affecting the suitability of deep fills (> 20 m) for redevelopment. The appropriateness of standards and codes in addressing the settlement issue are discussed. This surface settlement profile is governed by often complex pit geometries, fill properties, and various settlement mechanisms. The paper presents the results of a literature review of deep fill settlement monitoring data and uses this to develop a method for estimating both the total post end of filling settlement and the effect of time, fill material, compaction and uniformity, zoning, fill depth and quarry geometry on differential settlement and the expected surface settlement profile. The application of this method for estimating the surface settlement profile at Enfield Brick Pit, Sydney is presented. The use of plate load testing as an index of settlement performance of fill is discussed. Results of an extensive plate load testing program on sandstone fill and clay fill are presented. Finally, the appropriate use of monitoring to confirm settlement predictions is discussed.

## 1 INTRODUCTION

In early 2002 PSM was commissioned to provide advice regarding filling of the decommissioned Enfield Brick Pit in Sydney's inner western suburbs. Enfield Brick Pit (shown in Figure 1 below) had a surface area of 40,000 $m^2$ and was up to 40 m deep. The batters of the pit were in Ashfield Shale and were steep and subvertical in areas. The base of the pit was complex and irregular with a number of steps, sumps, haul roads and a heritage dyke across the centre etc.

**Figure 1** Aerial photograph of the Enfield Brick Pit prior to dewatering and backfilling.

The owner of the pit proposed to fill the brick pit with virgin excavated natural material (VENM) to produce a final landform for immediate residential, commercial or light industrial development. PSM assessed the ability of conventional earthworks design, specifications and controls to deliver the product required by the owner.

## 2 APPLICABILITY OF CONVENTIONAL EARTHWORKS SPECIFICATIONS

### 2.1 TYPICAL EARTHWORKS SPECIFICATION

In Australia conventional earthworks specifications often rely on the guidelines presented in AS3798 "Guidelines on earthworks commercial and residential developments" (1996). Earthworks specification reflecting the guidelines in AS3798 generally provide varying degrees of control on the subgrade condition, fill material, placement, compaction and moisture content and generally define the degree of control of the earthworks identifying possible testing and inspection regimes. Such a typical specification may comprise:

- Proof rolling

- Virgin Excavated Natural Material (VENM) only
- Maximum particle size of 200 mm
- Compacted layer thickness of 300 mm
- Compaction ratio of 98% - 102% SMDD
- Moisture variation of OMC ± 2%
- Level 1 Control and Testing as per AS3798

It is generally acknowledged that earthworks specifications developed in accordance with the guidelines in AS3798 will result in a well compacted, uniformly graded, structural fill with controlled properties and adhering to the guidelines in AS3798. The wide acceptance of such specifications has resulted in contractors, consultants and testing authorities which are well equipped and accustomed at implementing such a specification.

Furthermore recorded performance of fills shallower than 10 m indicates that such a specification has successfully delivered a final landform which allows immediate residential, commercial or light industrial development.

Experience in filling ground deeper than 20 m and developing immediately following completion of filling is very limited. Areas of deep fill have often been lay fallow, prior to delayed development, or zoned for limited uses e.g. parkland, sport fields.

### 2.2 APPLICABILITY TO ENFIELD BRICK

Preliminary assessment of the filling options at the Enfield Brick Pit indicated that a conventional earthworks specification such as described above would not necessarily deliver a landform meeting the development objectives of the owner. That is the unstated development objectives of AS3798 did not match with the development objectives of the Enfield Brick Pit owner. Delivery of the landform requested by the owner required a detailed understanding of the project specific development objectives and constraints.

## 3 OBJECTIVE DRIVEN FILLING METHODOLOGY

### 3.1 ENFIELD DEVELOPMENT OBJECTIVE

At the time of PSM involvement, the primary development objective at Enfield Brick Pit was to produce a landform suitable for medium density residential development immediately upon completion of filling at the minimum costs.

### 3.2 GENERALISED DEVELOPMENT OBJECTIVE FOR LARGE SCALE EARTHWORKS

Based on the Enfield Brick Pit experience and further experience with large earthworks projects it is considered that, while often unstated, the generalised development objectives for large scale earthworks can be summarised as follows:

1. To produce a landform suitable for proposed development (e.g. residential, commercial, high rise, low rise etc) at a given time following completion of the filling operations.
2. To ensure the earthworks are controlled in such a way to ensure that Objective 1 is achieved.
3. To ensure that the earthworks are documented and certified in terms that provide confidence to all parties that Objective 1 has been achieved.
4. To ensure that Objectives 1, 2 and 3 are achieved at a minimum cost.

This paper addresses Objective 1 as constrained by Objective 4.

### 3.3 OBJECTIVE 1 – LANDFORM SUITABLE FOR PROPOSED DEVELOPMENT AT A GIVEN TIME

The suitability of a landform for the proposed development can be defined in the following measurable engineering terms:

1. Bearing capacity of the landform.
2. Total and differential post end of filling settlements under self weight over the life of the proposed development.
3. Settlement response to structural loading.
4. Shrink swell potential.

The conventional earthworks specifications discussed in Section 2 provide adequate controls on items 1 and 4 of the above list, some control on item 3 and uncertain control on item 2, i.e. long term total and differential settlement of the fill over the life of the development. Item 2 is unlikely to govern the suitability of the landform where the fill is shallower than 10 m. This is consistent with the recorded performance of fills shallower than 10 m and placed in accordance with conventional earthworks specifications. Item 2 of the above list is discussed in detail in the following sections. Some useful data regarding item 3 is also provided.

## 3.4 TOTAL POST END OF FILLING SETTLEMENT

### 3.4.1 Sources and magnitude of settlement

An extensive literature review on the settlement behaviour of deep fills was undertaken to investigate the settlement mechanisms that govern the post end of filling settlements of deep fills. It is proposed to present and discuss this literature review in a subsequent paper. The review indicated that the post end of filling settlement is controlled by complex settlement mechanisms the effects of which are often difficult to clearly distinguish. The three identified mechanisms are discussed below:

1. Short term settlement (often referred to as immediate, elastic or consolidation settlement) describes the short term settlement of the fill under its own self weight. Much of this occurs during filling for the time frame of urban development.
2. Creep settlement is the major source of settlement of deep fills. These settlements are time dependant and occur over the duration of the life of the fill. The relationship between the magnitude of creep settlement and time, is often described in the literature in terms of a creep coefficient, $\alpha$, and the following equation:

$$S_{cr} = D\ \alpha\ log_{10}\ [(t - t_0)/(t_1 - t_0)](1) \qquad (1)$$

Where:

$S_{cr}$ is creep settlement (mm) at time t
D thickness of fill (mm)
$\alpha$ (or creep coefficient), is the creep strain per log cycle of time
$t_0$ is the start of creep settlement
$t_1$ and t are the start and end times of the settlement period of interest.

The application of the above relation to estimate creep settlements requires an estimate of time $t_0$ and the creep coefficient. The creep coefficient is in turn a factor of fill material, fill compaction, stress level i.e. thickness of fill and stress history i.e. rate of filling.

3. Inundation settlements. The literature indicates that deep fills when inundated by the returning groundwater table may experience an additional episode of settlement. The magnitude of inundation settlement is closely related to the material type, the level of compaction and as placed moisture content.

### 3.4.2 Estimates of creep coefficient, $\alpha$

The literature review identified approximately twenty projects where settlement monitoring of deep fills was undertaken. The settlement monitoring data presented the post end of filling settlement with respect to time. For much of this data, details of fill thickness, fill material, fill compaction, fill construction and ground water inundation was also available. These details were used to estimate creep coefficients for various material types and degrees of compaction.

These estimates were presented in a talk to AGS Sydney in 2004 and are summarised in Table 1. These values are considered to capture the spectrum of post end of filling behavior of deep fills (depth between 15 m and 40 m). Waddell and Wong (2005) present results of laboratory testing undertaken to assess the creep coefficient and discuss the stress dependency of the creep coefficient.

Although the estimates in Table 1 are referred to as creep coefficients, $\alpha$, it is likely they also include at least in part the effects of inundation and short term settlement.

| | | Material Type | | | |
|---|---|---|---|---|---|
| | | **Unknown** | **Rockfill** | **50:50** | **Clay** |
| **Degree of compaction** | **Compacted** ↑ | 0.3% to 0.5% | 0.025% to 0.05% | 0.2%<br>0.25%<br>0.2% to 0.3%<br>0.1% to 0.3% | 0.1% to 0.4%<br>0.25% to 0.5% |
| | ↕ | | 0.15% to 0.3% | 0.1%<br>0.225% | - |
| | | 1.0% | - | 0.4 | - |
| | **Loose** ↓ | 1.0% | 0.6% | 0.1%<br>0.4%<br>0.2% to 1.0%<br>1.0%<br>0.8% | - |

**Table 1** Creep coefficient, $\alpha$, in the literature. Unit of $\alpha$, is strain per log cycle of time.

### 3.5 DIFFERENTIAL POST END OF FILLING SETTLEMENT AND SURFACE SETTLEMENT PROFILE

The following factors affecting the differential settlement of the deep fill, and ultimately the surface settlement profile experienced by the landform have been identified:

1. Total settlement and rate of settlement as discussed in Section 3.4.
2. Side batter slope and irregular base shape. The pit base geometry has a major impact on the total and differential settlement profile surface settlement. The pit base geometry, comprising buried rock ledges, batter slopes and variations in floor levels will be reflected as surface settlement, with the buried features affecting a larger area than just directly above the feature.
3. Base preparation. The materials present or left in the base of the pit and their condition will effect the surface settlements.
4. Fill uniformity. Fill placement and compaction methods should result in contemporaneous areas of similar materials having similar long term settlement behaviour.
5. Fill zonation. Horizontal and vertical variation in material types results in differential settlement at the surface.

### 3.6 PREDICTING THE SURFACE SETTLEMENT PROFILE AT ENFIELD BRICK PIT

#### 3.6.1 Prediction methodology

At Enfield Brick Pit the development objective of being able to develop the landform immediately following the end of filling required the post end of filling settlement profile of the final landform to be predicted. The following prediction methodology was adopted:

1. The fill material was characterised in such a way to locate it within the spectrum of the post end of filling settlement behavior presented in Table 1.
2. The significant times controlling the post end of filling settlement behavior of the fill where defined as follows:
   a. Time $t_0$, i.e. the start of creep settlements, was taken as end of filling. This definition of $t_0$ is consistent with the assumption adopted when interpreting the settlement data to estimate the creep coefficients.
   b. Time $t_1$, was taken as the start of development (assumed to be 0.25 years after end of filling).
   c. Time t, was taken as the end of life of the proposed structures (assumed 50 year design life).
3. Undertake numerical modeling to predict surface settlement profile. An analysis method able to model the effect of material type, fill zoning and side batter slopes and irregular base shape on settlement profile was developed.

#### 3.6.2 Characterisation of fill type

Initial Characterisation of the fill type was undertaken based on fill material and compaction level. Two fill types were identified:

1. Clay Fill – Comprising excavated shales and residual soils from the Wianamatta Group, the Hawkesbury Sandstone, the Bald Hill Claystone, and the Narabeen Group. Clay fill placed and compacted to a Hilf (Standard) density ratio equal to or greater than 98%. Clay fill was compacted using a CAT825 roller.
2. Sandstone Fill – Comprising excavated sandstones and siltstones from the Hawkesbury Sandstone and the Narrabeen Group. Sandstone Fill as placed to a target minimum in situ modulus of 40 MPa as measured using plate load testing (PLT). Spreading of the sandstone fill was undertaken using a CAT825 roller, which also provided some compaction. The fill was then further compacted using a 25 tonne vibrating pad foot roller. Control of the sandstone compaction was undertaken using PLT rather than in situ density and compaction testing as this requires less than 20% of the particles to be retained on the 37.5 mm sieve (AS1289.5.4.1).

PLT was then used as a practical index to further assist in locating the above two material types within the creep coefficient spectrum. A total of 208 PLTs were conducted, of these 34 were in Clay Fill, 162 on Sandstone Fill. Twelve (12) tests were conducted on sandstone fill that was not well compacted as a comparison with the well compacted fill. Typical PLT results on the Clay Fill and on Sandstone Fill are shown on Figure 2. A summary of all the PLT results are shown on Figure 3.

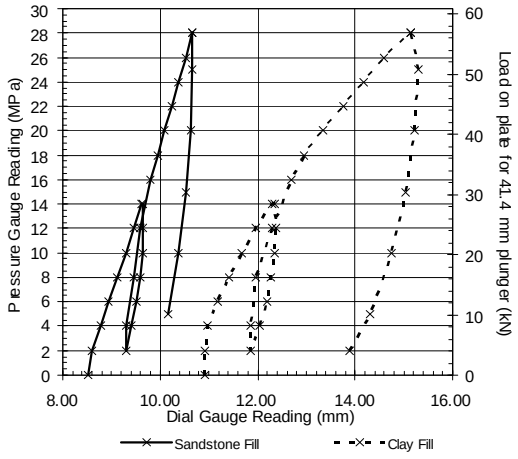


**Figure 2** Typical PLT results.

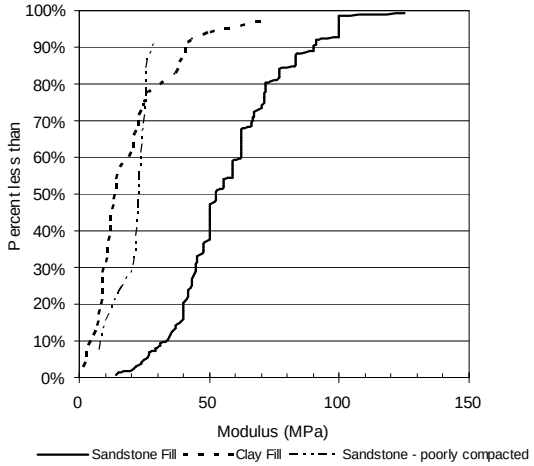


**Figure 3** Summary of plate load test results

Based on the fill thickness, the fill type and the PLT results the creep coefficients of $\alpha_{clay}$ = 0.5% and $\alpha_{sast}$ = 0.1% were adopted. The results reported in Waddell and Wong (2005), have confirmed these estimates. It is noted that for smaller fill depths the rates of settlement are expected to be smaller than those provided above.

### 3.6.3 Numerical analysis to predict the surface settlement profile

Published methods such as the Charles Method (Charles and Skinner, 2001) do allow analysis of settlement versus time and have methods of estimating the effect of simple buried batters on the surface settlement profile. These methods however are of limited application as they are not able to analyse the effects of side batter, and irregular and complex base shapes, base preparation, fill type and fill zonation on the surface settlement profile.

The creep settlement of the fill was modelled using a finite element package Phase 2D. Since this package did not allow creep settlements to be modelled directly, the following analysis approach was adopted. A vertical section through the landform was modelled, the fill was divided in layers of equal thickness, and the unit weight increased over a number of stages to model the creep settlements over logarithmic time. The load increments were calibrated so that in areas where to the effect of batters and changes in base geometry on surface settlements are least, and the fill type is uniform, settlements resulting the numerical analysis agreed with the settlement resulting from the simple application of Equation 1. Comparison of the settlement profiles resulting from this form of modelling to those predicted by the Charles Method for uniform fill over simple base geometries indicate that the two methodologies produce similar results.

On this basis the methodology was considered valid and was used to assess the effect of fill type, fill zonation, buried batter slopes and irregular base geometry, and time on the surface settlement profile of the Enfield Brick Pit. Furthermore, the methodology was also successful at modelling the reducing creep coefficient for reducing stress levels. The methodology could also be used to model the time settlement relationship of the fill.

Figure 4 provides differential settlement profiles for the period between $t_1 = 0.25$ years after end of filling, to time t = 50 years (i.e. over design life of structure) as predicted using the above modelling methodology for a section through the Enfield Brick Pit and various fill arrangements. It is noted that the three arrangements shown could all be the result of earthworks undertaken in accordance with the conventional earthworks specification presented in Section 2.1 of this letter. As shown on the charts the resulting landform would experience very different surface settlements and degrees of suitability for the proposed development. In particular, placing sandstone fill over clay fill may appear to be an attractive option, but is not desirable in deep pits.

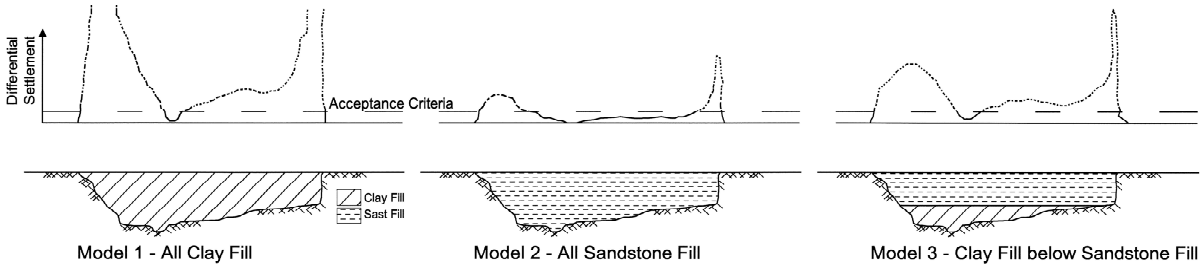


**Figure 4** Differential surface settlement profiles at Enfield Brick Pit for various fill arrangements.

This prediction methodology allowed a fill arrangement to be designed to result in a settlement profile allowing the vast majority of the pit to be developed immediately after end of filling. The adopted fill arrangement comprised sandstone fill at the base and over the batters, clay fill over the rest of the profile, and 'slip joints' concentrating the differential settlements at particular predefined locations. The fill arrangement is shown in Figure 5.

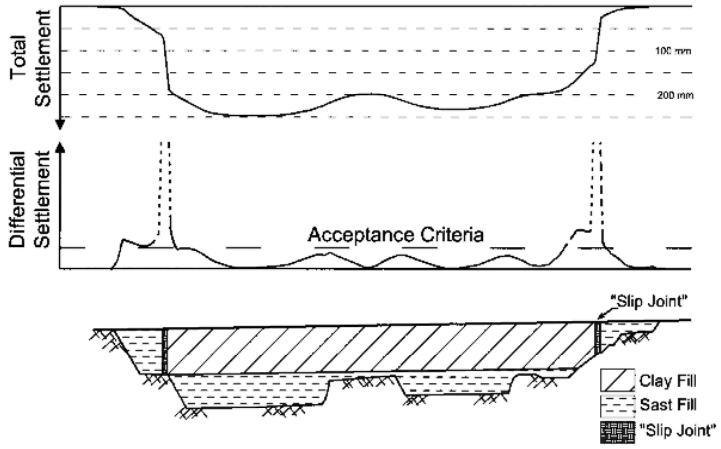


**Figure 5** Total and differential surface settlement profile at Enfield Brick Pit for adopted fill arrangement.

## 4 SETTLEMENT MONITORING

Filling of the Enfield Brick Pit was effectively over on 22 July 2005. A delay in the development schedule allowed the owner to undertake settlement monitoring of the final landform. The monitoring comprised monitoring of the vertical displacement of surface pegs spaced on a regular grid over the site using conventional survey methods. Base readings of the pegs were on completed in August 2005 and the pegs were then monitored on five other occasions between September 2005 and April 2006. The resulting data set, while noisy, (with an accuracy of ± 5 mm) broadly confirmed the settlement model. The survey methodology has since been changed to attempt to increase the accuracy. It is intended to publish the results of this monitoring in subsequent papers.

## 5 REFERENCES

AS3798 (1996) "Australian Standard: Guidelines on earthworks for commercial and residential developments", Standards Australia.

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# GEOTECHNICAL ASSESSMENT OF FOUNDING CONDITIONS FOR LARGE WIND TURBINES TARARUA RANGES, NEW ZEALAND

**Steven Price**
*Riley Consultants Ltd*
*Auckland, New Zealand*

## ABSTRACT

The Tararua Windfarm is one of the world's top three wind farm sites for mean wind speeds. Its owner has committed to a stage 3 extension involving construction of 35 large wind turbines standing 65m, producing 3MW each.

The Tararua Ranges are located immediately adjacent to an active fault zone and have recently been up-thrusted. The ranges consist of shattered greywacke sandstone with a complex capping of Tertiary and Quaternary marine deposits, along with re-worked loess.

The proposed sites for the turbines are generally challenging as 'easier' sites are already occupied by in excess of 100 smaller turbines.

Theoretically relatively low strength material is required to support the turbine weight resting on 15m diameter octagonal pads. However, cyclical rocking under wind and possibly seismic load, requires strong material of high stiffness. Thus, greywacke rock is the preferred founding material.

In January 2006 construction of the stage 3 extension commenced. Large earthworks are required to form foundations and adjacent crane platforms. Excavations to date have generally been consistent with conditions inferred from the geotechnical investigation, however, some sites have revealed geology more complex than envisaged, such as localised low strength gully fills on ridge crests. Thus, foundation designs have been adapted to the subsurface conditions encountered.

## 1. INTRODUCTION

In order to meet growing electricity demand New Zealand generation companies have been identifying potential new sources of power. Whilst hydro and fossil fuels provide the majority of New Zealand's electricity needs wind is becoming increasingly important. With the New Zealand Resource Management Act and the international Kyoto Protocols, establishing new hydro or fossil fuel generation schemes is increasingly difficult.

TrustPower Ltd, a private New Zealand owned generation company, has committed to a stage 3 expansion of an existing wind farm on the Tararua Ranges, east of Palmerston North city. This expansion is not yet complete at the time of this paper's preparation. A general view and site location is shown in Figures 1 and 2.

**Figure 1:** View of pre-existing Tararua Windfarm looking northeast.

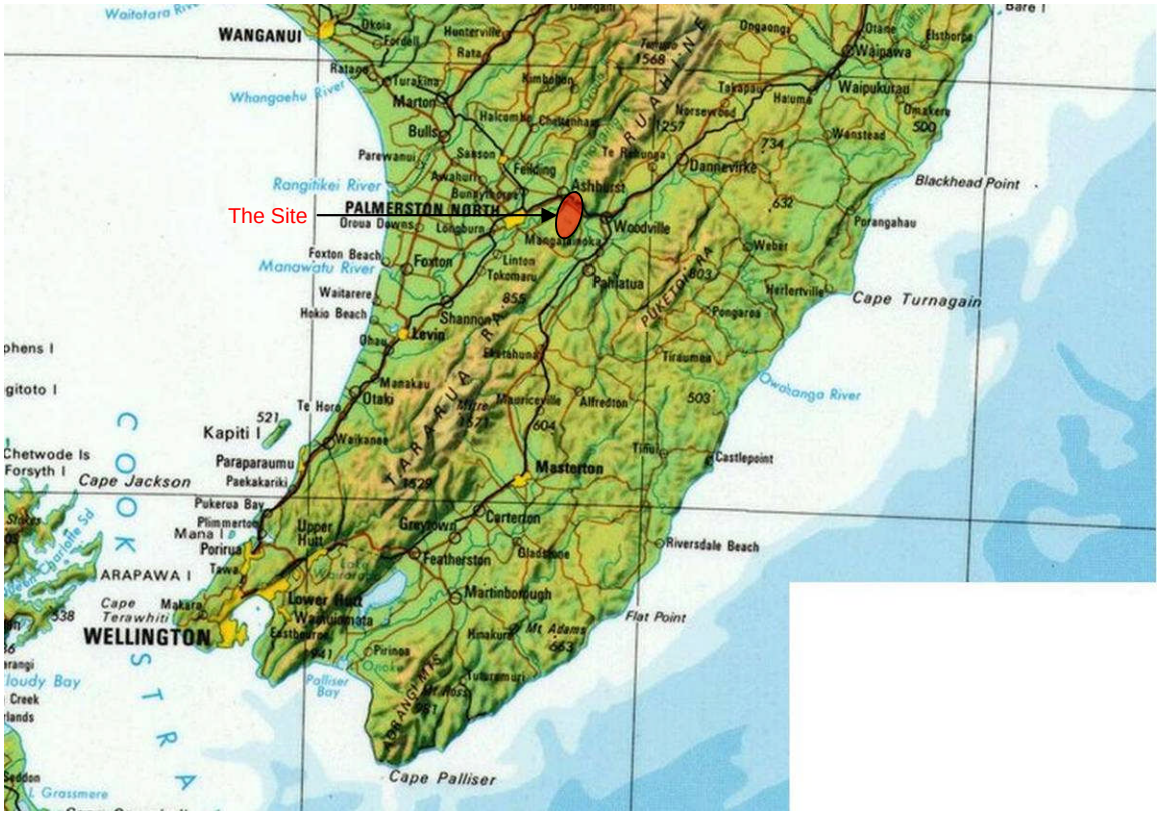


**Figure 2**: Location plan.

The existing wind farm, comprising the original two stages of construction, has approximately 105 turbines each producing a maximum of 0.6MW. The Stage 3 expansion includes a further 35 wind turbines producing 3MW each. These new machines are significantly larger than the older standing 65m high, each supported on an octagonal foundation 15m in diameter. The foundation pads are approximately 2m thick.

## 2. SITE DESCRIPTION

The existing wind farm spans a generally broad ridge which rises steeply above the flat Manawatu plains and Woodville valley to the west and east respectively. The plains either side of the Tararua Ranges are approximately 100m above sea level whilst the Ranges are in the general order of 400 – 600m above sea level. The wind farm is located on a low point in the Ranges (a saddle) immediately south of the Manawatu Gorge, where the Manawatu River flows east to west through the Ranges. In this area, approximately 9km in lateral length, two wind farms have been located either side of the Gorge.

The subject site extends 5.3km south of the Gorge and was selected because of high annual mean wind speed of around 10m/s at hub height. The Tararua Windfarm is recognised as one of the best generation sites in the world.

The earlier smaller turbines typically occupy flat to gently terrain at the crest of the Ranges. Subsequently sites for the newer and larger turbines are often on smaller, local ridge spurs flanked by steep slopes elevated of typically 340 – 580m above sea level. The site encompasses many different landowners who raise livestock.

## 3. GEOLOGY

The predominant material forming the Tararua Ranges is Mesozoic age greywacke sandstone and argillite. Overlying this is localised Tertiary sedimentary deposits partially overlain by Pleistocene/Holocene alluvial material forming terraces and filling gullies in older deposits. A general geological plan is shown in Figure 3.

The Tararua Ranges are bordered to the immediate east by the Wellington Fault which divides into the Ruahine and Mohaka Faults to the north. These faults are considered geologically active and have in recent times uplifted and tilted the Tararua Ranges. During the initial period of uplift it would appear the Manawatu River meandered across the saddle area of the Ranges (where the existing wind farm is sited) until eroding a gorge in its present position.

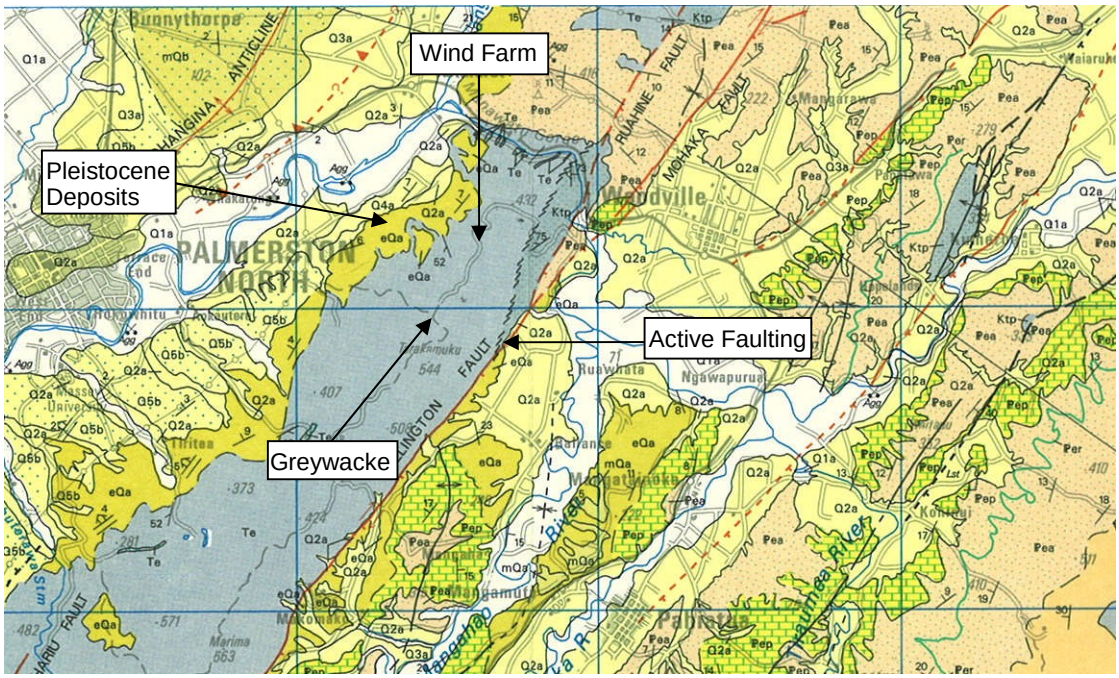


**Figure 3:** Published geology of the Tararua Windfarm and surrounding area (Lee & Begg 2002).

### 3.1 FAULTING

Though no active faults are publicly mapped directly crossing the wind farm site, many lineaments (lineations in the ground surface) of varying length (hundreds to thousands of metres) have been observed to cross the site from site walkovers and study of aerial photos.

### 3.2 GREYWACKE

The Mesozoic age sandstone and argillite beds are generally referred to as 'greywacke,' which more correctly refers to the sandstones. These deposits have generally undergone low grade metamorphism and have been subject to multiple phases of mountain building. Bedding is often difficult to determine and when distinguishable is typically subvertical or steeply dipping. Localised bodies of chert are also found within the greywacke. Argillite is also occasionally present generally exhibiting lower strengths than greywacke.

The greywacke is typically moderately to very closely fractured and has tested unconfined compressive strengths near surface of 3MPa to 24MPa. It is common for the greywacke to be crossed by steeply dipping clay filled defects extending 10m+ in horizontal extent.

### 3.3 TERTIARY DEPOSITS

Tertiary age sedimentary deposits south of the Manawatu Gorge are relatively uncommon although common north of the Gorge. They occur sporadically both on ridge crests and stream bases, typically comprising fine grained, very weak, calcareous and non-calcareous mudstones. Some deposits exhibit rock strength when dry, but when saturated and vibrated by machinery lose strength.

### 3.4 PLEISTOCENE DEPOSITS

These younger materials consist of complex layering of silts, sands and gravels exhibiting varying densities from loose to very dense. These deposits cover approximately half of the existing wind farm site becoming thicker to the north (exceeding 20m depth adjacent to the Manawatu Gorge). Though probably laid down relatively flat these deposits on the western side of the Ranges have now been tilted westward by 4º to 9º due to uplift of the Ranges.

### 3.5 HOLOCENE DEPOSITS

These recent deposits comprise clays, silts and occasional fine sands typically constrained to filling of old gullies or at the base of modern streams. Surficial loess (wind blown) deposits were also encountered sporadically mantling the surface across the wind farm and interbedded with alluvial deposits, varying from 0.5 – 3.0m thick.

## 4. INVESTIGATIONS

Riley Consultants Ltd was engaged by TrustPower Ltd to investigate geotechnical conditions for 40 proposed turbine sites. With tight construction timeframes the majority of investigation and reporting was undertaken in January and February 2005 with additional work performed on selected sites in March and April 2005. Geotechnical investigations at each site typically comprised a site walkover by an engineering geologist, followed by subsurface investigation consisting of at least 3 – 4 test pits and either machine boreholes or hand auger boreholes. At many sites depth to rock strength material was highly variable and additional relatively inexpensive percussion boreholes were sunk.

## 5. STABILITY

With much of the gentle terrain occupied by the older existing turbines, the proposed turbines were often sited on relatively narrow, steep sided ridges. In many situations the proposed foundations were of a similar size to the width of the ridge crest. Earthworks associated with the creation of construction platforms and roading meant the ridgelines were typically cut down by up to 6.0m creating broader ridge crests. However, following earthworks steep slopes were still in close proximity to some proposed foundations. Mapping and subsurface investigation of these slopes was undertaken often followed by computer based stability analyses to assess the current and post earthworks stability of the slope, along with any setbacks or protection measures that maybe required.

For practical design purposes a setback of 15m was adopted between the edge of a proposed turbine foundation and steeper slopes (i.e. slopes typically >13º). The 15m setback adopted was based on a local geology review, inspection of past slope failures, computer based analyses and experience with similar materials and slopes at other locations. This setback would typically allow a Factor of Safety against instability of exceeding 1.5 for the foundation.

In some situations, minimum setbacks from steeper slopes or those considered a stability risk, could not be achieved and either palisade (inground) retaining walls, rock anchors or a combination of both were recommended. Of 40 sites investigated eight were recommended for either palisade walls and/or ground anchors following geotechnical investigations. Following the bulk of earthworks sites recommended for protection measures reduced to 2.

## 6. FOUNDATIONS

The relatively large size of the proposed turbine foundation means the turbine weight could generally be supported on stiff soils, with an undrained shear strength of approximately 80kPa. However, cyclical rocking under wind and possibly seismic load, requires strong material of high stiffness with an equivalent Young's Modulus of 200MPa. Thus, rock strength material is the preferred foundation support. For some sites greywacke rock was present 0.5m to 2.0m below the proposed foundation base. In these situations it was proposed to undercut to rock and replace excavated material with 'as-dug' greywacke gravel compacted into place. This was later changed to AP65 crushed alluvial gravel.

Substantial depths of Pleistocene granular alluvium underlay some of the western sites with greywacke at substantial depth (10m+). In these situations it was anticipated the turbine foundations would bear on dense gravels and sands, encountered during the geotechnical investigation.

## 7. CONSTRUCTION

In January 2006 construction of TrustPower's stage 3 Tararua Windfarm expansion commenced. Three earthworks contractors constructed roads and platforms at the turbine sites, then 19m diameter by 2m deep foundation excavations.

The holes are inspected by representatives of Riley Consultants Ltd to check bearing capacity, depths of undercut if required, significant defects and any potential stability risks to the proposed foundation.

Testing of the rock at the excavation base is an important component to ensure stiffness requirements are met. Methods for testing have included Clegg hammer, Schmidt hammer and Scala penetrometer. The most convenient and efficient testing method has proved to be systematic use of the Clegg hammer. Clegg Impact Values can be related to Young's Modulus in a fairly direct manner. Where CIVs identify weaker areas Scalas are often performed to estimate depth to more competent material. The Clegg was found to be convenient, easy to operate and sufficiently sensitive to small variations in rock strength. Care must be taken though as surficial fracturing and water affect recorded CIVs.

With some sites, particularly on alluvial materials, soils of insufficient strength were encountered at the time of platform formation. In this situation additional percussion and Standard Penetration Testing (STP) were performed by drilling contractors along with test pits.

The majority of foundation excavations have exposed closely jointed greywacke rock with minor argillite, chert and breccia. In some excavations defect mapping can identify 'sets' to the jointing, however, other excavations reveal random orientations or very local sets. Jointing is predominantly steeply dipping (50º+) to subvertical. Within nearly all excavations in greywacke can be observed clay lined joints or faults, often of 1 – 5mm aperture. These are typically 'wavey' in their orientation, and are inclined at angles less than the jointing but usually steeper than adjacent slopes (25º+).

At many of the sites, particularly where adjacent to steep slopes, detailed defect mapping followed by stereonet construction was undertaken allowing an assessment of potential defect controlled slope failures. In rare occasions potential defect related failures were identified and either the foundation position adjusted or protection measures such as palisade wall or rock anchoring recommended.

There was awareness of potentially active faults either crossing or in close proximity to the turbine foundation given the proximity of the adjacent Wellington fault zone. To date, with the exception of one site (site 9 discussed below), no excavations have revealed any obvious faults that may be active.

Groundwater entering the foundation excavations has generally not been a problem, particularly as the majority of excavation was in late summer and autumn. Through winter more groundwater has been encountered but this has not significantly affected construction.

## 8. EXAMPLES OF FOUNDATION EXCAVATIONS

Presented below are examples of foundation excavations ranging from typical (site 5) to extraordinary (Sites 9 & 34).

### 8.1 SITE 5

The foundation excavation exposed highly to moderately weathered, closely jointed, greywacke with no major argillite exposed. There were virtually no steep slopes in close proximity to the foundation, with the exception of a slope to the northwest of about 20º leading down to a stream (shown in Figure 4 below).

**Figure 4:** Site 5 foundation excavation and adjacent stream.

Mapping of major defects in the excavation and detailed measurement of joints at the northwestern corner were performed. Analysis indicated two potential wedge type failures that could daylight to the northwest. Dip of the wedges (<19º) and the nature of the defects which formed this wedge (rough, zeolite filled) indicated failure was unlikely.

Rotational and translational type failure of the northwestern slopes was also considered. Computer based stability analyses were performed indicating a Factor of Safety against instability of >1.4 under extreme conditions (generally >1.3 was acceptable under extreme conditions). Erosion of the slope toe by the stream was also considered. With a sufficient setback between the steeper slope and foundation edge it was considered enough warning time would be available to implement protection measures should erosion advance up the slope.

### 8.2 SITE 9

The concrete foundation at this site was eventually poured at the second foundation hole excavated. Within the first 19m wide hole was exposed weak argillite and breccia forming approximately 50% of the excavation in a northwest-southeast orientation, displaying evidence of significant crushing. This zone was parallel with lineaments in the general area. The site is approximately 500m from the Wellington Fault.

The argillite was of low strength and although this zone could not be proved to be associated with active faulting, it was considered the risks were too high and an alternative position for the turbine should be identified. Resource consent conditions limited any alternative site near the first excavation. Shallow investigative trenching immediately north of the existing hole identified suitable rock with no obvious crush zone(s).

Excavation of the second hole revealed a clay pug filled lineament crossing the southern part of the foundation hole and continue through the platform in an east-west orientation (Figure 5). This feature appeared to be an off-shoot of the crush zone, or alternatively obscured in the crush zone exposed in the first hole.

**Figure 5:** Clay filled lineament through second foundation excavation at site 9.

Approximately 100m to the north of the site was known to be Pleistocene alluvial deposits overlying greywacke. A trench of approximately 60m in length was dug in an attempt to intercept the inferred fault and identify whether it offset the overlying alluvial deposits. The fault was not encountered and careful measurement and plotting of the fault in the cut platform indicated it curved away to the south, avoiding the practical limits of trenching.

Detailed aerial and on-ground study of lineaments was performed. This work suggested the inferred fault was inactive as it would appear this lineament had been offset by later faulting that is now considered inactive. The presence of faults and relative periods of activity were primarily established through valley locations and offsetting of streams.

The foundation was subsequently constructed at this location. Slopes to the immediate north of the foundation were noted as being affected by past instability and considered at risk of further movement. A palisade wall was recommended along the northern edge of the foundation.

### 8.3 SITE 34

Although excavated on a ridge crest, this foundation encountered inferred Holocene/Pleistocene gully alluvium and minor Tertiary sedimentary weak rock. It was evident from the excavations that an incised gully cut through the greywacke at this location. The softer alluvial deposits were undercut until very weak greywacke rock was encountered. This resulted in a relatively large undercut up to 4m+ in the northwestern corner. The undercut was backfilled in 200mm compacted layers of AP65 crushed alluvial gravel as shown in Figures 6 and 7.

**Figure 6:** Site 34 excavation and undercut concrete pad. Founding level is approximately at the mid-bench level.

**Figure 7:** Backfilling of undercut at Site 34.

## 9. CONCLUSION

Although the Tararua Windfarm development is located mainly on greywacke the surficial geology is often challenging for the founding of large turbines, further complicated by the nearby presence of active faulting.

Relatively simple, but systematic, exploration and verification testing in combination with careful observation has proved successful in addressing the geotechnical issues associated with the foundations.

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# DRAINAGE ANALYSIS OF HYDRAULICALLY PLACED MINE BACKFILLS AND MINE FILL SLOPES

**Kirralee Rankine**
*Golder Associates Pty. Ltd.*
*Cairns, QLD Australia*

## ABSTRACT

In large scale underground metallifersous mining operations, the most efficient removal of ore is often achieved by dividing the ore body into a series of prisms called *stopes*. The solid rock within each stope is blasted, and the fragments removed via drives for processing. This operation leaves massive underground voids, which are backfilled with hydraulic fill comprising a slurry of the coarser portion of the tailings generated during processing of the ore. The hydraulic fill is at moisture contents much larger than the saturation moisture content and therefore significant amounts or water must be removed via porous barricades. The barricades are constructed within the stope access drives to contain the backfilled material within the stope during filling.

Due to the large volumes of water being introduced to the stopes, the potential for considerable pressure build up on the barricades can be very high. Recent catastrophic underground barricade collapse within Australia has prompted a response within the industry towards research into hydraulic mine drainage issues.

## 1 INTRODUCTION

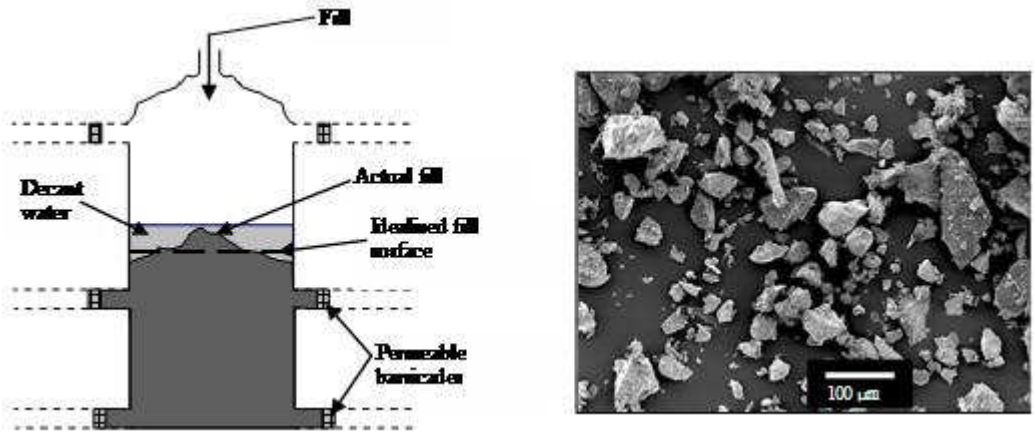
The open stoping mining method is a mining technique used in the underground ore extraction process for very large ore bodies. Excavation initiates from the base of the ore body, and the roof and/or walls are progressively dug out, so the ore falls to the base of the stope. The ore is then crushed and transported via drives, to the surface for processing. The large voids (which may extend well over 100 m in height) are generally backfilled using the by-products of the ore extraction and minerals processing.

The ore body is excavated through a very complex blasting pattern designed individually for the stopes. Access to the stopes for explosive placement is achieved via drives which are generally spaced 20-40 m vertically up the wall of the stope (Fig. 1(a)). Stopes will generally be designed with two to four drives at each access level.

To provide local and regional rock support for the subsequent removal of adjacent stopes, the voids are backfilled once ore extraction is complete. The fill material must have sufficient strength (often achieved through the addition of cement to the fill mix) to avoid instability when adjacent stopes are sequentially removed throughout the mining cycle. To complete the extraction of an ore body, many stopes are required. At any given stage of an underground mine operation, the excavation of several stopes will be under way at one time, and similarly several stopes will be in the backfilling stage.

This paper focuses on the drainage characteristics and associated properties of a particular type of mine fill material known as ‘hydraulic fill’, and some of the issues related to using this fill material in underground mining operations.

The manner in which the stope is backfilled depends on the type and characteristics of the fill, but the fill material is generally introduced to the highest point of the stope either via pipeline (for the case of paste or hydraulic fill) or by conveyor (in the case of aggregate or rock fill). When material enters the stope all drives to the stope must be completely sealed to prevent the fill from entering the drives. The fill is usually in a slurry form when placed in the stope, and without restraint, would easily flow into other parts of the mine. In hydraulic fill mining operations, permeable barricades are constructed within the drives to contain the fill slurry as it is being placed into the stope (Fig. 1(a)).




(a) (b)

Figure 1: (a)- Cross-sectional diagram of stope during filling; (b)- Electromicrograph of hydraulic fill sample

Typically underground barricades take the form of walls constructed from specially designed permeable barricade bricks. If the pressure build up behind the barricades (typically considered a result of the pore pressure build-up, development of the lateral load from the hydraulic fill, and any dynamic loads) exceeds the strength of the barricade then failure will occur. As a consequence of barricade failure, hydraulic fill may surge from the stope into the mine. Such a surge is hazardous to mining operations, and has in recent cases resulted in tragedies.

## 2. CHARACTERISATION OF TYPICAL AUSTRALIAN HYDRAULIC FILLS

Hydraulic fills are simply sandy silts, or silty sands with negligible clay fraction. They are the result of fill being passed through hydroclones in a process known as desliming. As shown in Figure 1(b) hydraulic fills typically have a highly angular grain shape (Nicholson and Wayment, 1964, Pettibone and Kealy, 1971), and therefore higher friction angles than common granular soils. Based on a thorough investigation of over twenty separate hydraulic fill samples from several of the larger underground metalliferous mines across Australia, many properties (in particular the permeability of typical Australian hydraulic fills) have been determined and were used as input for the numerical modelling presented later in this paper. Some important properties characterised are reported in this section.

It is widely understood within the mining industry that the fine grains have the most significant influence on permeability of the fill. Thomas and Holtham (1989) observed that 10 μm is generally selected as the arbitrary division between coarse and fine grains. Therefore, the rule-of-thumb adopted across Australia is to aim to have less than 10% particles passing 10 μm. Herget and De Korompay (1978), quote 35 μm as the typical $D_{10}$ value for hydraulic fills, and many other researchers with extensive experience have quoted hydraulic fill $D_{10}$ values in excess of 10 μm as typical (Kuganathan 2002; Cowling et al. 1988; Brady and Brown 2002; Bloss 1992). Grain size distribution tests were performed on 22 different hydraulic fill samples from several mines across Australia, and the grain size distribution curves for typical Australian hydraulic fills, fall within a very narrow band with $D_{10}$ values ranging from approximately 8.5 μm to 42.9 μm.

There is a wide range for fill specific gravity values for the Australian hydraulic fills tested. These values ranged from approximately 2.79 to 4.35. It is common for fills to be pumped back underground as a certain percentage solids by weight (e.g., 72% solids). For this reason the specific gravity of the fill being placed will significantly influence the quantity of water that is also placed underground. At a specific solid content, larger values of specific gravity would imply larger mass of solids to fill in the stope and thus larger quantity of water in the slurry. This can equate to a difference of tens of thousands of tonnes of water in a single stope, and therefore specific gravity for a particular fill should be tested and treated on an individual basis.

Constant and falling head permeability tests were performed on several hydraulic fill samples from various Australian mines. The test results are summarised in Figure 2, and show very good agreement between the two testing methods. In the mining industry, mm/hr is the preferred unit for permeability (1 cm/s = 36000 mm/hr). Average Darcy's coefficient of permeability fell between 10.2 mm/hr and 53.7 mm/hr for these samples tested.

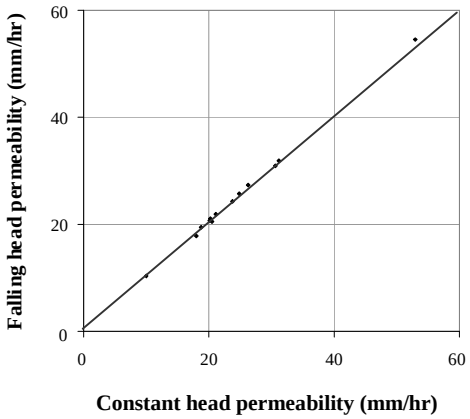
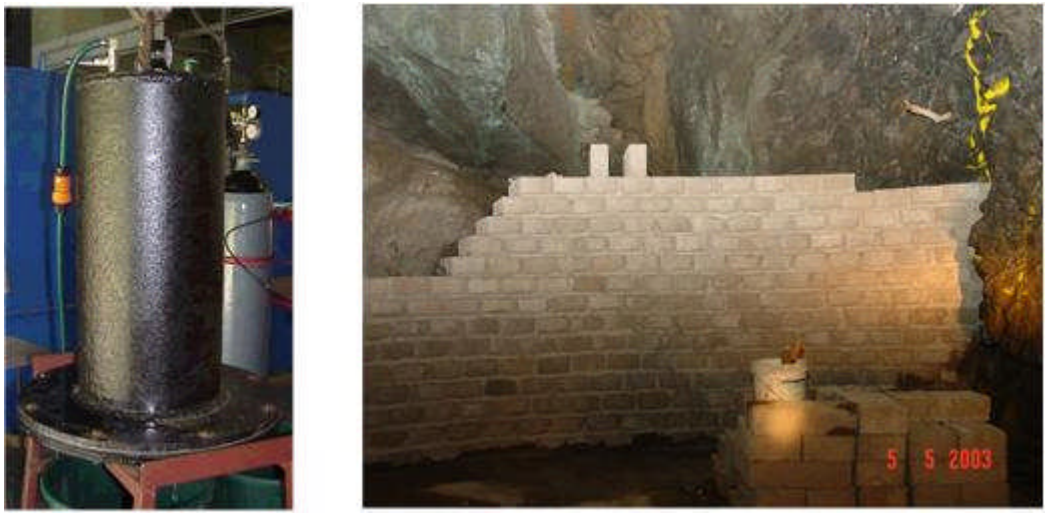



Figure 2: Fill constant head and falling head permeability values

## 3. PERMEABILITY OF POROUS BARRICADE BRICKS

The cylindrical pressure chamber shown in Figure 3(a) was constructed at James Cook University (Townsville), to undertake constant and falling head permeability tests as well as flow variation with pressure tests on permeable barricade bricks. The pressure chamber provides an innovative and effective means by which the permeability of various materials may be obtained through three different permeability testing methods, and also allows for the analysis of the flow behaviour under applied pressure.




(a) (b)

Figure 3: (a)- Photograph of pressure testing chamber; (b)- Photograph of in place porous brick barricade

All bricks tested had significantly larger permeability than that of any of the hydraulic fill samples tested. Hydraulic fill permeability values average approximately 2 $x10^{-4}$ cm/s to 35 $x10^{-4}$ cm/s whereas the range for the permeable bricks used to contain the fill were tested to range between 1.2 $x10^{-2}$ cm/s and 3.1 $x10^{-1}$ cm/s, which generally places them 2-3 orders of magnitude more permeable than the fill. This has a significant impact on the understanding of the overall stope drainage system. This sizeable difference indicates that provided the barricades are built from the bricks in such a way that the construction does not impede the drainage performance, it may be assumed that for modelling purposes, the barricade does not contribute to the pore pressure development within the fill, and hence the drainage of the system does not depend on the permeability of these bricks. Furthermore, it implies that a coupled analysis between the brick drainage and the backfill drainage is not required.

# 4. 2-DIMENSIONAL AND 3-DIMENSIONAL SIMULATIONS OF STOPE FILLING AND DRAINAGE

## 4.1 VERIFICATION OF NUMERICAL MODELS

There is very limited material published specifically relating to the numerical modelling of the drainage of hydraulic fill stopes. The potential use of numerical modelling as a prediction tool in the drainage of underground cemented and uncemented hydraulic fill mines, could probably be attributed to the pioneering work undertaken for Mount Isa Mines Ltd. (MIM) by L.T. Isaacs and J.P. Carter (Isaacs and Carter, 1983). The work resulted in the development of a 2-dimensional model intended to provide a basic understanding of the concepts of the drainage of hydraulic fills in underground stopes, and produced many significant outcomes. The finite difference program, written in FORTRAN developed as a component of this work by Isaacs and Carter (1983), is still used as the predominant numerical model on which hydraulic fill drainage predictions are based within the Australian mining industry (Cowling, 2002). It is for this reason, along with availability of verification data, that the Isaacs and Carter program was utilized as the reference for the hydraulic fill drainage modelling undertaken in this research.

The explicit finite difference program *FLAC*, was used to simulate the filling and drainage of a two dimensional stope. A fictitious stope drainage problem was designed based on typical input parameters for stope dimensions and fill properties, such that an identical data set could be used as input to provide a direct comparison between the 2-dimensional, finite difference flow-only code developed by L.T. Isaacs and J.P. Carter at the University of Queensland in 1983, and a similar simulation developed by the author in *FLAC*. This verification exercise was designed to compare the water and tailings levels, and discharge during a specific filling schedule. The maximum pore pressures predicted were found to match those cited by Isaacs and Carter (1983). Maximum. The *FLAC* code incorporated capabilities to monitor these values throughout the drainage of the stope for the entire duration, since filling started and until the free water had completely drained from the stope, leaving the fill at residual water content.

The 2-dimensional program coded in *FLAC* was used as a preliminary step, and the overall problem investigated to determine the key objectives for which the more time consuming 3-dimensional extension of the program (modelled in *FLAC*$^{3D}$) would investigate.

The *FLAC* and *FLAC*$^{3D}$ results for fill and water levels, and discharge rates throughout the filling of the verification simulation were compared to results obtained from Isaacs and Carter for an identical simulation. Results compared very well between all three models. Verification was also undertaken to ensure the three programs produced similar results for simulations in which the water level fell below the fill height.

## 4.2 THREE DIMENSIONAL DESIGN CHARTS TO PREDICT RELATIVE EFFECTIVENESS OF ADDITIONAL BASE DRAINS

Discharge rates and pore pressure developments within a stope are governed mainly by the geometry of the stope and drain, the permeability and specific gravity of the hydraulic fill, and the solids content and residual water content of the slurry. The following section will show how the three dimensional program can be used to determine the relative effect additional base level drains have on the simple single centrally located drain arrangement commonly used as a control case. Figure 4(a) illustrates the plan view of each of the base drain arrangements investigated (the stars indicate the maximum stope pore pressure locations). For stopes which have drains located on several vertical levels, discharge was found to primarily exit through the base drain/drains. Thus, additional level drains are not included in this analysis. Figure 4(b) illustrates the geometrical variables for the Case 1 (single centrally located drain) arrangement.

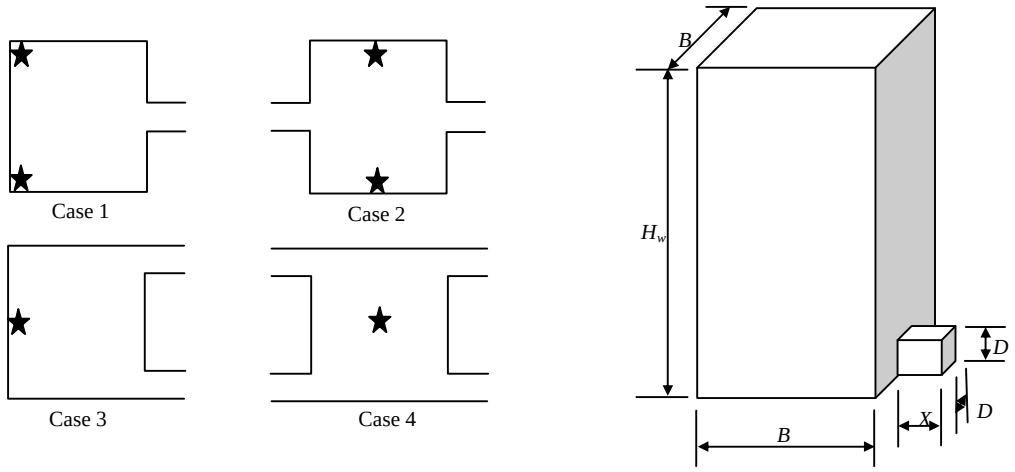



Figure 4: (a)- Maximum pore pressure locations shown on plan view for 3-dimensional stopes; (b)-Geometrical variables for 3-dimensional Case 1 stope

The value of drain placement and numbers, relative to Case 1 may be studied by dividing the discharge and maximum pore pressures for each of the other three cases by those measured for Case 1. Figure 5 shows the relative discharge efficiencies for the various stope arrangements relative to Case 1, for a D/B ratio of 0.2. Non-dimensionalized charts for discharge rates such as the one shown in Figure 5, and also maximum pore pressure predictions for various $H_w$/B, D/B and X/D ratios can be used to assist in determining the performance of subsequent drains on removing water from a stope and reducing pore pressure build up within the stope.

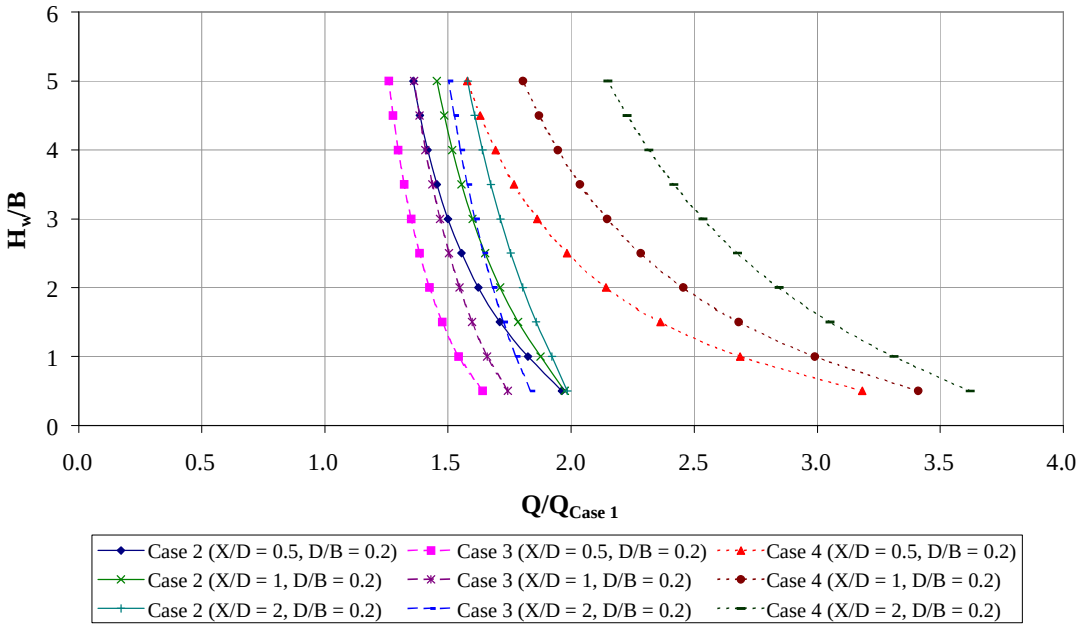



Figure 5: Case 2, 3 and 4 discharge rates /Case 1 discharge rates for D/B = 0.2

Several trends are observed through these charts and may be used during stope design to maximise drainage and minimise pore pressure. Obviously, Case 4 (with the most drains) has the highest discharge rates and lowest maximum pore pressure values for all geometrical aspect ratios studied, and Case 1 (with the fewest drains) has the lowest discharge rates and highest maximum pore pressure values. Case 2 is marginally more effective in water removal and has consistently lower peak pore pressure values than Case 3 (which also has 2 drains). The difference between the cases decreases with drain length and with increased drain width (the measurements were all recorded for drains with a square cross-section).

The efficiency of drain performance relative to Case 1 plateaus to a maximum for pore pressure and a minimum for discharge at large $H_w/B$ values. This means that as the water level and the pore pressures increase, the benefit in extra base drains on reducing the pore water pressure increases despite the effectiveness in the extra drains on removing the water (relative to the Case 1 arrangement) decreasing. Discharge efficiency of subsequent drains increases with drain length.

## 5. SUMMARY

A good understanding of the pore water pressure developments within hydraulic fill stopes and the discharge through the drain is essential for improving the designs of stopes, hydraulic fills and barricades and subsequently improving the safety in the mines. Hydraulic fill and porous barricade brick drainage properties were determined through thorough laboratory testing and a numerical model developed to simulate the filling and draining of a single stope such that discharge rates and pore pressures within a stope can be predicted. It has been shown how this program can be used as a tool to better understand the drainage behaviours in stopes and the information obtained used in optimising drainage in stope design.

## 6. ACKNOWLEDGEMENTS

This research was undertaken under the supervision of Dr. Nagaratnam Sivakugan at James Cook University in Townsville, QLD. I would like to offer my most sincere thanks to Siva for his guidance, patience and friendship during this research. I would also like to thank the various mines who facilitated this research through their provision of testing material.

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# STABILISATION OF THE CLIFF FACE AT WITTON BLUFF

**Brendan T. Scott**
**Peter W. Mitchell**
*URS Australia Pty Ltd*
*Adelaide Office*

## ABSTRACT

The remediation works recently carried out to stabilize a section of the cliff face at Witton Bluff, South Australia are described. These remediation works were required because continuing terrestrial erosion has lead to large sections of the Witton Bluff cliff face becoming unstable. The remediation methods included the use of a geogrid reinforced earth retaining wall, used for the first time in South Australia in a coastal environment. The importance of community involvement, including working closely with the local Aboriginal people, is emphasised. Experiences gained from the project provided several lessons for future projects of a similar nature.

## 1. INTRODUCTION

Witton Bluff is an elevated headland situated near the Adelaide seaside suburbs of Christies Beach and Port Noarlunga (Figure 1). The bluff and adjacent cliffs have in the past been under active erosion both at their base by the sea (marine erosion) and at higher levels by rain, wind and non-natural agencies (terrestrial erosion). Earlier protective measures undertaken in the 1980s comprising a rock revetment around the entire cliff base and selective filling and vegetation have proven successful in the section of cliffs toward Port Noarlunga (Mitchell & Lee 1987). Since that time however, protective measures have been required on the cliffs north of the previously protected section due to ongoing terrestrial erosion.

This paper describes the remediation works that have been recently carried out to overcome the continuing terrestrial erosion at Witton Bluff. Scott & Mitchell (2004) outlined earlier observations on the protection measures. This paper outlines the behaviour over the 3-year period since construction.

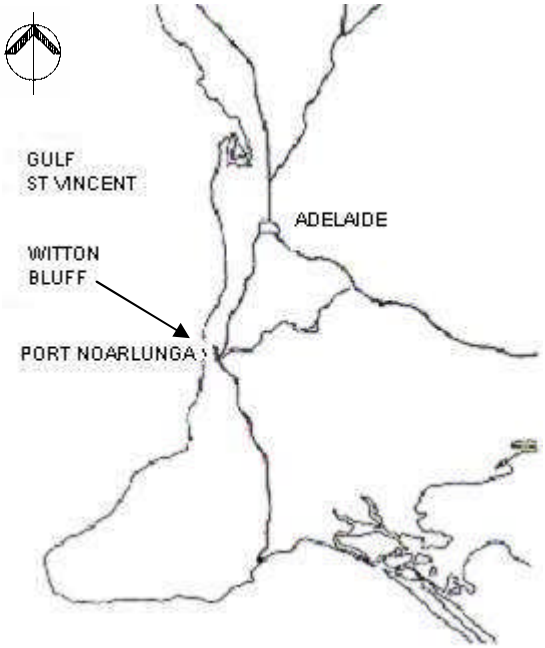


**Figure 1:** Location Plan

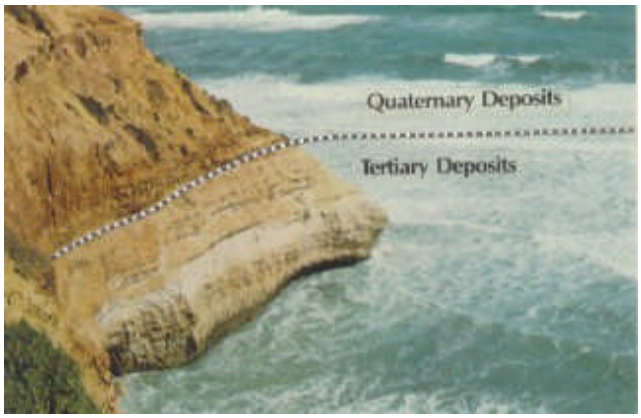


**Figure 2:** Geology of Witton Bluff
Photo taken in 1980 prior to previous protection measures (photo from Selby 1984)

## 2. GEOTECHNICAL ASPECTS

The coastal cliffs at Witton Bluff generally consist of Tertiary deposits (1.6 million to 70 million years old) overlain by Quaternary deposits (present day to 1.6 million years old), as shown in Figure 2.

The lower Tertiary deposits contain mainly the so-called Blanche Point Formation which is a weak rock comprising calcareous and siliceous claystone. These deposits are more resistant to erosion due to wind and rain, however are subject to erosion from sea wave action.

The Quaternary deposits comprise clay and sands which are easily eroded by rainfall runoff. The Quaternary deposits at Witton Bluff are readily erodible, losing strength and giving them the potential to collapse when saturated (eg. Taylor et al 1974).

Slope stability analysis of the slopes at Witton Bluff found that the slopes had a suitable factor of safety against failure when dry, however, when saturated, the slope was considered to be unstable (Mitchell & Lee 1987). Stormwater runoff and rainfall impacts have combined to create an active erosion process of the Quaternary deposits, leading to sections of the coastline to become unstable.

## 3. EXISTING CONDITIONS

The City of Onkaparinga were becoming increasingly concerned over the rate of erosion and commissioned a study of the coastline north of Witton Bluff. Three separate zones of coastline were identified as requiring remedial action, with each area having a different type and level of distress. The three zones are indicated in Figure 3 and are briefly summarised as follows:

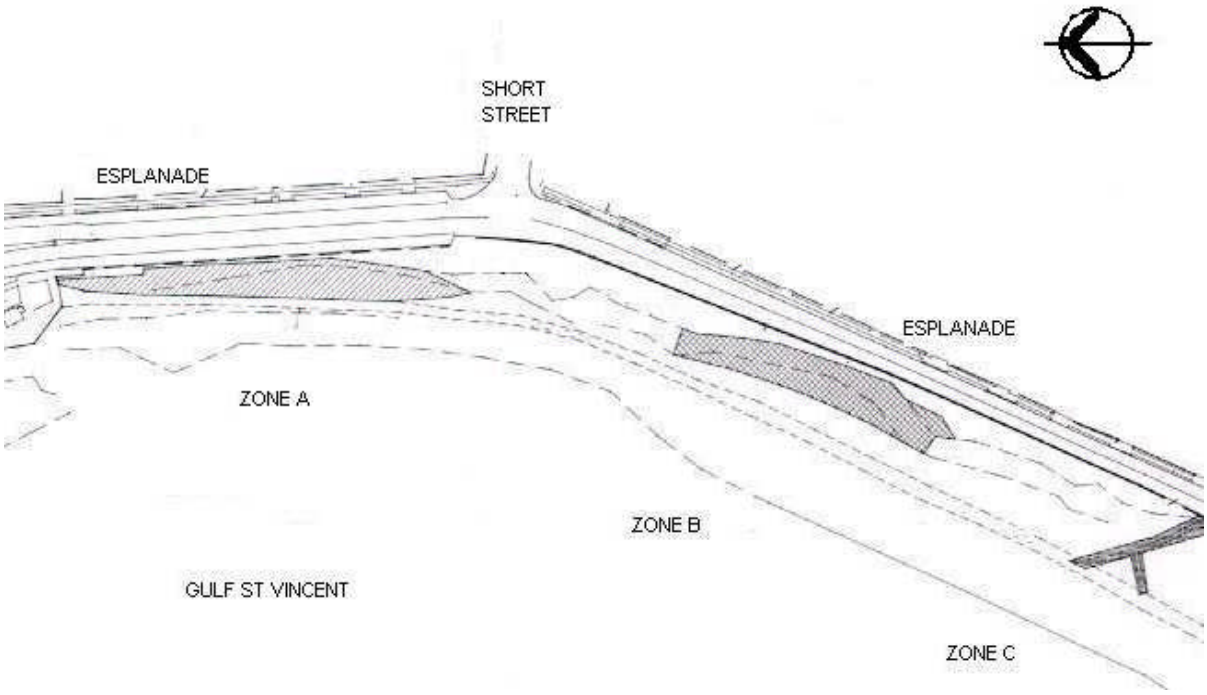


**Figure 3:** Plan view of site showing the three identified zones of distress

Zone A is a 90 metre long section of coastline featuring cliffs that are relatively low in height, but within close proximity to the road above. The steep cliff-faces were undergoing an active erosion process and there was evidence of undercutting of the slopes.

Zone B is a 60 metre section of coastline consisting of a near vertical cliff face and a narrow berm at the top of the cliffs. Erosion of the cliffs in this area was of primary concern, with fissures close to the road evident in the top of the cliffs, as observed in Figure 4. The long term stability of the road was of concern in this area. Although precise measurements were not taken, blocks of soil were being lost and the cliffs were becoming increasingly unstable.

Within Zone C, the cliffs were generally stable; however there were localised unstable areas where an informal walkway had been etched into the slope. Human activity in this area has hastened erosion, causing obvious slips and creating unstable areas.

**Figure 4:** Erosion of Cliffs Adjacent to Road

## 4. PROTECTION OPTIONS

The primary objectives of the remediation works were to:

- Make safe a section of coastline at Witton Bluff so that the risk of structural failure of the cliff and surrounding areas was eliminated; and
- Safeguard those cliffs that were not subject to immediate collapse from the worst effects of erosion and degradation from the natural effects of wind and rain and the additional effects of over-intrusion by people and animals.

Several possible protection options were investigated and costed to determine the most appropriate strategy for each section of coastline. It was considered that placing fill against the cliffs was the most suitable and cost effective method of stabilisation for Zones A and C, as filling could be placed against the cliffs without impeding access to the walking path at the base of the cliffs.

However in Zone B, site clearance issues indicated that a type of retaining wall would be required to enable the well traversed walking path at the base of cliffs to remain open. More details of these protection options are given in PPK (2000) and in the following section.

## 5. DETAILS OF PROTECTION STRATEGY ADOPTED

The method of slope stabilisation consisted of the placement of fill and vegetation against the existing cliffs.

Imported clean fill was environmentally tested before being brought to site. The fill was compacted in place and constructed on a batter of 1 vertical to 2 horizontal as shown in Figure 5.

Biodegradable erosion matting was placed in all zones over the newly placed fill. The purpose of the erosion matting was to stabilise the surface layer of soil against wind and rain until such time that vegetation becomes established.

Zone B provided the most challenging part of the project, with a geotextile reinforced earth wall constructed. Figure 6 shows a typical cross section of the Zone B remedial works. Zone B featured a 6.6 metre high geogrid reinforced earth retaining wall built in front of the existing cliff to stabilise the upper section of the cliff-face. The geogrid retaining wall consists of high strength polymer grid layers that reinforce the soil, making the slope self-supporting. It also allows the face of the wall to be vegetated to provide an aesthetic appearance as well as additional stabilization.

Both primary and secondary soil reinforcing layers were used, with hessian sand bags of garden loam used to provide support at the edge of the geogrid wall. Drainage was incorporated at the base and back of the wall by gravel layers. Geotextiles were used as separating layers, as a drainage medium, and for soil reinforcement.

Figure 7 shows the base construction of the reinforced earth wall and the drainage filter. Figure 8 shows the reinforced earth wall approximately half completed. The soil bags used to help define the geometry of the face of the wall and the layers of primary geogrid reinforcement can be observed. Figure 9 shows Zone B at project completion with the geogrid wall constructed to a height of 6.6 metres above its base and then the slope battered from the top of the existing cliffs to the top of the wall. It is understood that this was the first time in South Australia that a geogrid reinforced earth wall has been used in a coastal environment.

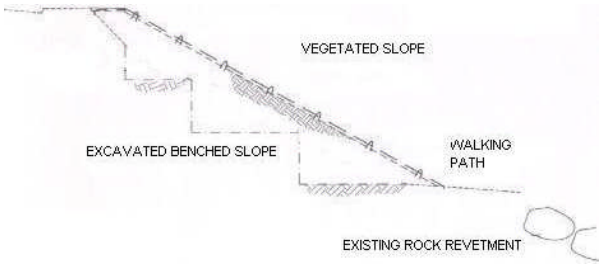


**Figure 5:** Simplified cross section of Zone A remedial method (Zone C similar)

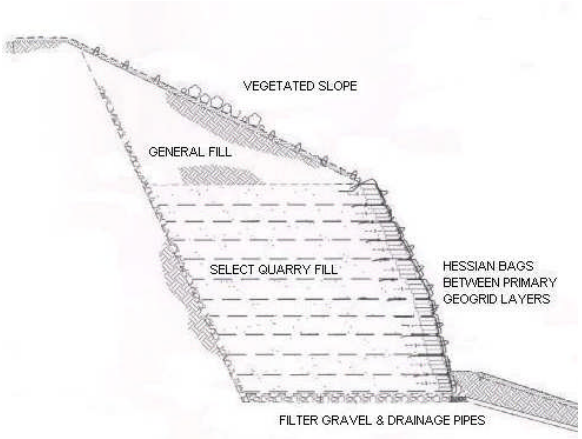


**Figure 6:** Simplified cross-section of Zone B remedial works

**Figure 7:** Base level of geogrid installed with drainage textile and filter gravel below

**Figure 8:** Zone B geogrid wall during construction

**Figure 9:** Zone B at project completion

**Figure 10:** Aboriginal Monitoring

## 6. CONSTRUCTION MONITORING

Level 1 supervision was undertaken by a Geotechnical Engineer who was on site full-time for the duration of the construction period. Monitoring of all quantities of fill brought to site was undertaken, and compared closely with the predicted fill volumes despite the presence of municipal waste being encountered at the base of Zone B which required over excavation and importing more fill than was envisaged at the time of the design. The final cost of the project was within budget and was completed on schedule, even allowing for the construction works being undertaken during the wetter months of May to July 2003.

## 7. VEGETATION

Previous protection measures at Witton Bluff included maintaining the growth of vegetation to prevent erosion. Within each of Zones A, B and C, vegetation was planted to cover the newly placed fill on the battered slopes. It was decided to use only native species that were grown locally within the southern Adelaide coastline. A total number of 11 species giving a total of 2900 plants were planted in a random mix during the project. Of these, the most commonly planted species was the ground cover *myoporum parvifolium* (creeping boobialla). This species was also the predominant species planted into the face of the reinforced earth wall in Zone B.

The long term success of the project is significantly dependent on the success of the newly planted vegetation to help prevent erosion. A three year maintenance period has just been completed to give the native shrubs and groundcovers the best possible chance of long term survival.

## 8. COMMUNITY AND ABORIGINAL ISSUES

Community Consultation was undertaken during the planning, construction and post-construction stages of the project. Various measures were undertaken to keep the community informed, including community meetings during the design and planning stages, press releases and newspaper articles, as well as letter drops to all stakeholders before and after construction.

The issues of concern to the local residents were related to the road closures during construction, pollution from the works entering the marine environment and the generation of noise and dust during the works. All of these issues were handled or restricted effectively during the construction period so that a positive community response was received.

Another issue of importance was that of Aboriginal Heritage. The local area is of significance to the local Kaurna Aboriginal people. Full-time Aboriginal monitoring was undertaken during all excavation works at the site. Figure 10 shows the Aboriginal monitoring process during excavation works in Zone B. Native title was not an issue because the site was constrained within a prescribed road reserve. Aboriginal people were also on site assisting with the planting works.

## 9. PROJECT COMPLETION AND LESSONS LEARNED

The protection of the three zones was carried out very successfully over the period May to July 2003. The visual impact on the environment has been fairly minimal as shown over 3 years after completion in Figure 11. A major reason for the success of the project was the close working relationship established between the Principal, Contractor, Geotextile supplier and the Consultant.

Experiences gained from the project provided several lessons for future projects of a similar nature, including the following:

- Being a project in a fairly high profile location and exposed to the general public for the entirety of the project, it was of obvious benefit to have the general support of the community. The community were generally appreciative of the fact that they were kept informed as to the need for the works and throughout the project duration. The time spent on community consultation was beneficial to the project outcome.
- Given the originality and uniqueness of the project, the presence of a full-time Geotechnical Engineer undertaking Level 1 supervision was of benefit to the project. As the supervising Engineer was also involved in the design process, it led itself to fast troubleshooting of problems or queries from the Contractor, leaving both the Contractor and Principal satisfied that works were not being delayed due to design or specification issues.

- Safety issues associated with the construction of the reinforced earth wall were highlighted as part of this project. Whilst the Contractor was able to work safely without incident for the duration of the project, the use of geogrid reinforcement in such applications as cliff stabilisation works highlighted a need for safer working conditions or extra safety mechanisms or barriers during construction for the safety of construction personnel. This issue is currently being investigated by the geotextile suppliers.

**Figure 11:** Photo taken September 2006

## 10. CONCLUSIONS

The cost of the project was within the original budget estimate made at the time of the design. The timing of the construction was within the expected timeframe, despite the construction period being undertaken during winter months where several days were lost due to poor weather.

A feature of the protection strategy adopted was the first application in South Australia, of a geogrid reinforced earth wall in a coastal environment for the purpose of cliff stabilisation.

The success of the project was enhanced by extensive community consultation and working with the local Aboriginal people, a knowledgeable and conscientious Contractor, a Principal who was cooperative and helped generate a team environment, and a Geotextile supplier who was always willing to ensure that products were used in their most effective manner.

## 11. ACKNOWLEDGEMENTS

The help of staff from The City of Onkaparinga, ADCIV, Geofabrics Australasia Pty Ltd, Green Environmental Consultants, Janet Gould & Associates, Research House and Parsons Brinckerhoff Australia Pty Limited who assisted with the project is greatly appreciated. Without the help from all of those involved, the work presented in this Paper would not have been possible.

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# THE HYDRAULIC CONDUCTIVITY AND CONSTRUCTION OF CLAY LINED SLUDGE LAGOONS VICTOR HARBOR WASTEWATER TREATMENT PLANT, SOUTH AUSTRALIA

**John Slade, Project Geological Engineer**
**Peter W. Mitchell, Senior Principal Geotechnical Engineer**
*URS Australia Pty Ltd*
*Adelaide, South Australia*

## ABSTRACT

The Victor Harbour Wastewater Treatment Plant (VHWWTP) is located approximately 25 km from Victor Harbor, South Australia and included the construction of eight clay lined sludge lagoons. The sludge lagoons were designed as evaporative ponds for the storage of liquid waste generated by a newly constructed effluent treatment plant. Each sludge lagoon was located upon a cut and fill platform constructed within a moderately sloping hillside. The sludge lagoons were lined with a compacted soil sourced from a local borrow area. Selection of suitable local material for the construction of the liners provided a geotechnical challenge for the project. Local materials were composed of clayey sand and were considered marginal in the materials ability to achieve a permeability of equal to or less than $1 \times 10^{-9}$ m/sec after moisture conditioning and compaction. After a phase of laboratory testing of the local materials, results were correlated with a permeability criteria developed by Benson et al (1994). Defining the type and source of local materials prior to construction ensured the cost effective selection of appropriate materials early in the construction phase thereby ensuring the construction of barrier liners were compliant with the intension of the design. The thickness of the liner was increased from the design value. During construction, visual assessment, further laboratory testing and compaction testing were undertaken to ensure each sludge lagoon was composed of compacted and moisture conditioned material in accordance with the design intention. The successful application of the approach to the selection of materials for the construction of the barrier liners within the VHWWTP sludge lagoons confirmed the benefit of Benson et al (1994) material selection criteria. In addition, the project generated management strategies and procedures to ensure the long term integrity of the clay lined sludge lagoons.

## 1. INTRODUCTION AND DESIGN REQUIREMENTS

The VHWWTP utilizes eight clay lined lagoons to dry polymer dosed waste activated sludge derived from a newly constructed effluent treatment plant. The treatment plant and adjacent sludge lagoons are located along the cut and fill platform constructed into a southwest facing hill slope. The hill slope is composed of a 1 to 2m layer of sand underlain by clayey sand and pockets of sandy clay. The hydraulic conductivity of the constructed liners was taken as the basic parameter for the design and for characterizing liner performance and reliability. The design requirement for each layer of the sludge lagoon liner was the construction and maintenance of a low permeability barrier (not exceeding $1 \times 10^{-9}$ m/sec) to the fluid waste from the VHWWTP.

In addition to the design requirement for the liner to restrict the flow of fluids through the lagoons and down the hill slope, the design had to be cost effective, durable and repairable. Each year several of the lagoons would be required to be emptied for maintenance. The design of the liner had to be durable to withstand the excavation of fines, repairable at a minimal cost and maintainable for the designed life. The remote location of the site, the nature of the sludge (low hazard fluids due to a modern treatment facility) and the marginal quality of the local soils for use as a clay liner presented a geotechnical challenge for the project. This paper details the material selection process, the design specifications and construction of the clay lined sludge lagoons, with specific reference to the hydraulic conductivity of the local materials utilized and a material selection criteria defined by Benson et al (1994).

# 2. BACKGROUND ON COMPACTED SOIL LINERS AND HYDRAULIC CONDUCTIVITY

Many guidelines for the construction of compacted soil liners are based on experience and include minimum values or acceptable ranges for material properties that describe the composition of soil (e.g. Atterberg limits and particle-size distribution) and recommendations for the selection of compaction criteria (i.e. control of water content and dry unit weight) and compaction machinery. Lamb (1954) stated that five factors had the greatest influence on hydraulic conductivity: soil composition, permeant characteristics, soil compaction, soil structure and the degree of saturation during permeation. The soil properties are discussed in the following sections.

## 2.1 SOIL COMPOSITION

Soil composition is influenced by Atterberg limits (indices of quantity and mineralogy of clay-size particles) and a soil's particle-size distribution (Benson et al 1994). Typically, higher liquid limit and plasticity index are associated with soils having a greater quantity of clay particles thus lower hydraulic conductivity at equivalent compaction and moisture content. In addition, the particle-size distribution of compacted clay affects hydraulic conductivity because the size of voids conducting flow is affected by relative proportions of large and small particle sizes. Low hydraulic conductivity is likely to be achieved when soil is well graded and the clay fraction governs the hydraulic behaviour of the matrix (Benson et al. 1994). In compacted clays, additional fines will fill pores between coarser particles, reduce the size of pores controlling flow, and consequently decrease hydraulic conductivity. Furthermore, as the percentage of fines consisting of silt and clay sized particles increase, the pore spaces conducting flow decrease further in size and thus hydraulic conductivity decreases (Benson et al. 1994).

## 2.2 SOIL STRUCTURE, COMPACTION & DEGREE OF SATURATION

Mitchell et al. (1965) noted that soil structure and resultant hydraulic conductivity were sensitive to moisture content and compactive effort used during the construction phase. They showed that as the moisture content is increased beyond optimum and the compactive effort is raised, the hydraulic conductivity decreases. Furthermore Mitchell et al. (1965) noted the lowest hydraulic conductivity occurs at moisture contents slightly (2-4%) wet of optimum moisture content. They concluded that at wet of optimum moisture content the shear strains induced during compaction were more effective in remoulding aggregates and re-orienting the clay particles into a compact and oriented arrangement leading to reduced hydraulic conductivity.

Lambe (1954) also noted the change in hydraulic conductivity that occurs when the moisture content and compactive effort is varied is a direct result of changes in the soil fabric at macro and micro-scales. At the macro-scale, increases in moisture content and compactive effort generally results in an increased ability to break down clay aggregates and to eliminate inter-aggregate pores. At the micro-scale, increases in moisture content and compactive effort results in reorientation of clay particles and reduction in the size of interparticle pores.

## 2.3 MATERIAL SELECTION CRITERIA

Benson et al. (1994) found from a database of 67 landfills in North America that hydraulic conductivity is correlated to the Atterberg Limits, the percentage of fines, clay content and activity. For cases of a well compacted liner with a moisture content wet of optimum, the minimum soil parameters to achieve a permeability less than $1 \times 10^{-9}$ m/sec were found to be liquid limit = 20, plasticity index = 7, %fines = 30, %clay = 15 and activity = 0.3. Benson et al (1994) cautioned that this correlation is not a substitute for permeability testing. However the Benson et al (1994) correlations do provide a very convenient means of assessing the suitability of a soil type for use as a liner prior to permeability testing.

## 3. SLUDGE LAGOON BARRIER LINER DESIGN SPECIFICATIONS

The sludge lagoons at VHWWTP are comprised of 8 separate lagoons in a 2 x 4 arrangement. The barrier liner was initially specified to comprise a total thickness of 300 mm constructed in two lifts of approximately equal thickness and maximum permeability of 1 x $10^{-9}$ m/s, placed in layers and compacted to at least 98% standard density at 0% to 3% wet of optimum moisture content. For each new lift, the surface of the previously compacted layer was scarified to create a roughened surface (by a scarified depth of 50 mm created by a pad foot roller as shown in Figure 1)

**Figure 1**: Compaction of barrier liner

The specification required that during placement of the barrier liner, representative samples of the blended and moisture conditioned liner material were to be tested for grading, Atterberg limits and laboratory permeability to confirm that the as-placed material meets the design intention. The material placed and compacted was tested for field density and moisture content.

Excavations for stop logs and plinths created considerable inclusions in the clay barrier liner as shown in Figure 2. During the installation of the stop logs and plinths the specification required that the re-installment of the barrier liner was to utilize blended materials from the local borrow pit compacted to the same design standard as the main liner. Reinstatement of the incision through the barrier liner included the continuation of the compacted barrier liner below and surrounding the sides of the individual stop logs or plinths so that the clay barrier liner for the lagoon was not breached at an inclusion.

**Figure 2**: Clay barrier liner incisions for plinths

## 4. SAMPLING AND MATERIAL SELECTION AT VHWWTP

A borrow pit located to the west and down-hill of the lagoon area was used to obtain a locally sourced material for bulk earthworks, and for the proposed barrier liners in each sludge lagoon. This was the only source of clayey material within a reasonable distance of the site. However it was realized at the time of the geotechnical investigation of the project that the locally sourced materials for use as a liner were marginal because of a relatively high sand content. However this soil type was a preferred material for use by the contractor because the higher sand content meant that it would be easier to work with than would be the case with heavier clay.

A series of 11 particle size distributions and Atterberg Limit tests in accordance with AS1289 tests 3.1.2, 3.2.1, 3.3.1, 3.4.1 and 2.1.1 were carried out. These tests indicated that the material would be classified as a clayey sand (SC) in accordance with the Unified Soil Classification System as incorporated in AS1726 'Geotechnical site investigations' code. The lowest liquid limit tested of the eleven samples was 33, and the lowest plasticity index was 23. Both these tests significantly exceed the Benson et al (1994) criteria for a permeability of $1 \times 10^{-9}$ m/sec outlined in Section 2.3 above.

However of the eleven tests carried out for the particle size distribution, one test gave a %fines = 29, and three others gave %fines = 33. These indicate that the sand content is either slightly in excess of, or is close to, the limit based on the Benson et al (1994) criteria of a minimum %fines = 30, implying that the material is a marginal material for use as a liner.

As the material was considered marginal, considerable care was exercised during construction to blend the excavated clayey sand from the stockpile prior to placement as a liner so that the possibility of concentrations of sand pockets in the liner was minimized.

The original specification of a 300 mm liner was changed so that the barrier liner comprising a total thickness of 500 mm constructed in three lifts of approximately equal thickness, placed in layers and compacted to at least 98% standard density at 0% to 3% wet of optimum moisture content. A zone of more permeable soil could then be surrounded and isolated by clay of significantly lower permeability which could be checked by inspections. The rate of field density testing was specified to significantly exceed that given in AS3798-1996 'Guidelines on earthworks for commercial and residential developments' for large scale (>1500 $m^2$) developments.

## 5. CONSTRUCTION

Construction of the sludge lagoons was undertaken by Adelaide Civil, under the direction of Tenix Alliance Pty Ltd. URS Australia Pty Ltd were the consultant geotechnical engineers certifying the construction of the eight sludge lagoons. The project was undertaken during a wet winter period of June to November 2005 and was commissioned in December 2005.

Compaction tests were undertaken on the blended and compacted clayey sand placed in the constructed barrier liner of the sludge lagoons. The compaction tests for the sludge lagoon barrier liners were found to be in the range of 98% to 104% standard with a mean compaction of 99.8%. The moisture content was at or above optimum moisture content. The placement of the material for the liner therefore met the required density and moisture content. Higher compaction above the required 98% standard would therefore result in the construction of a lower permeability barrier liner.

Permeability tests were carried out on 32 samples extracted from either the compacted lagoon liner, or from the blended material placed in the lagoon prior to spreading and compaction. The samples selected for testing were of the visually sandier materials. The permeability value was determined by the falling head test in accordance with AS1289 test 6.7.2 at 98% standard and at optimum moisture content. The average of four results per sample was taken. The results are shown in Table 1.

It can be seen from Table 1 that eight of the 32 samples that were tested had permeability values higher than $1.0 \times 10^{-9}$ m/sec. This would be reflective of a sand content being close to the maximum appropriate for a material of permeability less than $1 \times 10^{-9}$ m/sec, and the marginal suitability of the material when compared to the Benson et al (1994) criteria. However the departure from $1.0 \times 10^{-9}$ m/sec is not large and the mean result is $8.8 \times 10^{-10}$ m/sec which meets the required permeability.

The need for increase in thickness of the liner, the higher compaction than the specified 98% standard, the frequency of compaction testing exceeding that specified in AS 3798-1996, and careful site inspections to ensure that concentrations of the sandier materials was localised, were therefore warranted because of the marginal nature of the material.

| Results | Lagoon 1 | Lagoon 2 | Lagoon 3 | Lagoon 4 | Lagoon 5 | Lagoon 6 | Lagoon 7 | Lagoon 8 |
|---|---|---|---|---|---|---|---|---|
| 18/05/2005 | 5.90E-10 | 1.00E-09 | 6.90E-10 | 1.20E-09 | 2.90E-09 | 5.90E-10 | 3.50E-10 | 5.90E-10 |
| 18/05/2005 | 4.40E-10 | 3.80E-10 | 6.90E-10 | 2.30E-10 | 2.80E-10 | 2.20E-09 | 8.60E-10 | 4.40E-09 |
| 5/08/2005 | - | - | 1.10E-09 | 2.70E-09 | - | - | 1.70E-10 | 2.10E-09 |
| | - | - | 3.30E-10 | 2.40E-10 | - | - | | |
| 16/08/2005 | | 3.00E-10 | | | | | | |
| | | 1.60E-10 | | | | | | |
| 6/09/2005 | 2.80E-10 | | | | 4.60E-10 | 1.70E-10 | | |
| 9/09/2005 | | | | | | | 1.60E-09 | 3.10E-11 |
| 12/10/2005 | 2.00E-10 | | | | 6.60E-10 | 1.60E-10 | | |
| arithmetic mean | | | | | 8.77E-10 | | | |

Table 1: Laboratory Test Results for Permeability (values in m/sec).

## 6. SLUDGE LAGOONS MAINTENANCE STRATEGY

During the operation of the VHWWTP, a maximum of six lagoons at any one time is expected to be used. It is also expected that the lagoons will be emptied of sludge cake during late summer each year. The following precautions are being implemented to maintain the moisture content of the clay liner above the operational water line and when the lagoons are not in use:

1. The clay liner in the base of each lagoon is to be covered with 100mm of orange sand. When the lagoon is not in use the sand is kept permanently wet with either treated effluent or a layer of sludge. After the sludge is removed from each lagoon the protective sand layer on the base and topsoil on the batters will be inspected, repaired and reseeded as necessary to ensure that any over excavation or erosion does not result in deterioration of the clay liner.
2. The eastern and western (long side) batters of the lagoons are vegetated with 200mm layer of sandy topsoil and grassed with hydro-mulch (Victorian Ryegrass). The grass minimizes erosion of the embankments and a sprinkler system has been installed to maintain the moisture content.
3. The northern and southern batters of the lagoons are covered with 200mm layer of sandy topsoil, a layer of geotextile, and a top cover of 100mm shot rock. The shot rock has been designed to prevent erosion from wave action at the ends of the lagoons.
4. Diversion drains on the upside slope and along lagoon boundaries were installed to minimize water run off and potential scouring.

Figure 3 shows a completed lagoon.

**Figure 3**: Completed lagoon

## 7. CONCLUSIONS

The utilization of clayey sand from a local borrow pit has led to the cost-effective construction of eight barrier liners for the sludge lagoons at the VHWWTP. Grading and Atterberg Limit testing, laboratory testing of permeability, compaction testing and field inspections ensured that the construction was in accordance with the project specifications.

The Benson et al (1994) criteria indicated that the clayey sand proposed for the liner of the lagoons had a sand content close to the maximum for a material with a permeability of less than $1.0 \times 10^{-9}$ m/sec. This was reflected by measured permeability values in eight of the 32 tests being in excess of $1.0 \times 10^{-9}$ m/sec.

The design of each lagoon was changed so that each lagoon was constructed in three layers to 500 mm thickness rather than two layers to 300 mm thickness, and an average compaction well in excess of the specified 98% standard was achieved. The three layers were compacted with a pad foot roller under a quality control sampling, testing and inspection regime, so that zones in the liner that were slightly higher than the prescribed standard were vertically and horizontally bracketed in the field by samples that had permeability results that were less than $1 \times 10^{-9}$ m/s.

The equivalent permeability of the entire thickness of the clay liner was determined to be less than the prescribed $1 \times 10^{-9}$ m/s, so that with the thicker design, the higher than specified compaction, the intensive field density and moisture content testing and construction inspections, the intended design was met.

A maintenance strategy was established to ensure the long term integrity of the lagoon liners, particularly when the lagoons are not in operation.

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# DESIGN ISSUES FOR SEPLAS IN UNIFORM CLAY

**Zhenhe Song**

*Curtin University of Technology*
*WA, Australia*

## ABSTRACT

Plate anchors have attracted much attention in offshore deep water development. This paper studies anchor rotation during keying-in and anchor capacity during pullout when it is installed vertically in clay. Both numerical simulation of strip plate anchor and centrifuge test of square anchor in transparent "soil" were conducted. In the numerical analysis, Remeshing and Interpolation Technique with Small Strain model (RITSS) was used to simulate large movements of the anchor. In the centrifuge model tests, a transparent "soil" was used to observe anchor rotation during keying-in. It is found that the loss of embedment during keying-in is a function of eccentricity ratio $e/B$ and anchor thickness ratio $t/B$. And the anchor embedment depth where suction force at the anchor base is lost is a function of normalised soil strength ratio $s_u/\gamma' B$ or $s_u/\gamma' D$. The bearing capacity of a plate anchor can be determined by using the formulas proposed.

## 1. INTRODUCTION

In recent years oil and gas mining has moved into increasingly deeper water in search of undeveloped fields. For water depths in excess of 500 m, conventional platforms are replaced by floating facilities, anchored to the seabed using catenary or taut-wire moorings. The latter type of mooring imparts significant vertical loading to the anchor, and consequently many different types of anchoring system have been developed (Ehlers et al. 2004). The SEPLA (Suction Embedded Plate Anchor) and VDPA (Vertically Driven Plate Anchor) are two such systems, both of which comprise a plate anchor that is penetrated in a vertical orientation using a caisson (SEPLA) or mandrel (VDPA), and subsequently rotated by applying the anchoring force at some eccentricity until the plate becomes perpendicular to the applied force. This process is schematically illustrated in Figure 1 (Aubeny et al. 2001).

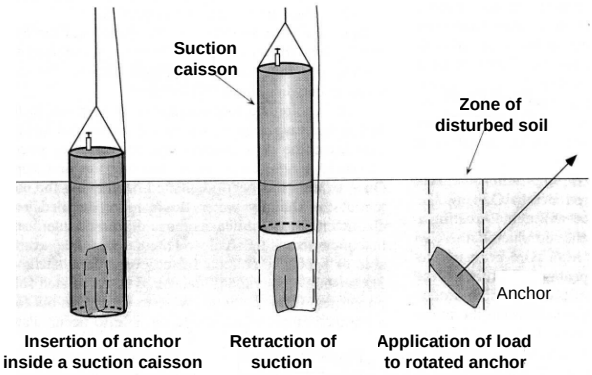



Figure 1: Suction installed plate anchor

Over the last four decades a number of researchers have proposed approximate techniques to estimate the uplift capacity of plate anchors in various types of soil ((Vesic 1971); (Meyerhof and Adams 1968); (Meyerhof 1973); (Das 1978), (Das 1980); (Das et al. 1994); (Rowe and Davis 1982); (Merifield et al. 2001), (Merifield et al. 2003); (Merifield et al. 2005)). However, most of studies above didn't consider the rotation behaviour and only took the "immediate away" case into account.

This paper aims to investigate the bearing capacity and rotational behaviour of plate anchors subject to vertical pullout in clay. Both centrifuge tests and numerical simulations were conducted.

# 2. ROTATIONAL BEHAVIOUR

## 2.1 NUMERICAL ANALYSIS

Numerical analyses were conducted using the AFENA finite element package developed by Carter and Balaam (1990) with modification to simulate the continuous rotation behaviour of vertically embedded plate anchor. H-adaptive mesh generation (Hu and Randolph 1998) has ensured that an optimal mesh is generated.

Although modelling of square anchors *in situ* is a three dimensional numerical analysis, it is complex and time consuming to simulate the large deformation associated with anchor rotation. To simply the problem, a two-dimensional strip plate anchor pre-embedded in clay was considered here in rotational analysis.

The soil was modelled as an elasto-plastic material with Tresca yield criterion. All analyses simulate undrained conditions and hence Poison's ratio $\nu = 0.49$, friction and dilation angles $\phi = \psi = 0$, and uniform stiffness ratio $E/s_u$ = 500 (where $E$ is Young's modulus and $s_u$ is undrained shear strength) are considered. The interface between soil and anchor was assumed to be rough.

The problem setup is shown in Figure 2. The plate anchor was pre-embedded vertically with $H/B = 3$ initially a uniform clay and a vertical loading force $F$ was applied to the padeye (chain attachment point on the plate anchor), which was located eccentrically at a distance $e/B = 0.17 \sim 1$ from the plane of the plate anchor. And the thickness ratio $t/B = 0.05$ 0.067, and 0.1. Thus, the moment and vertical force applied at centre of the anchor during rotation can be expressed as:

$$M = F \times e \times \cos\beta \tag{1}$$

$$F_v = F - W_s \tag{2}$$

Where $W_s$ is self weight of the plate anchor;

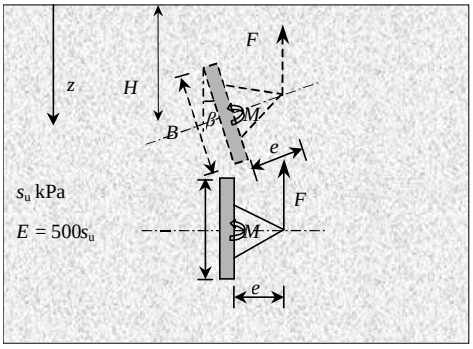
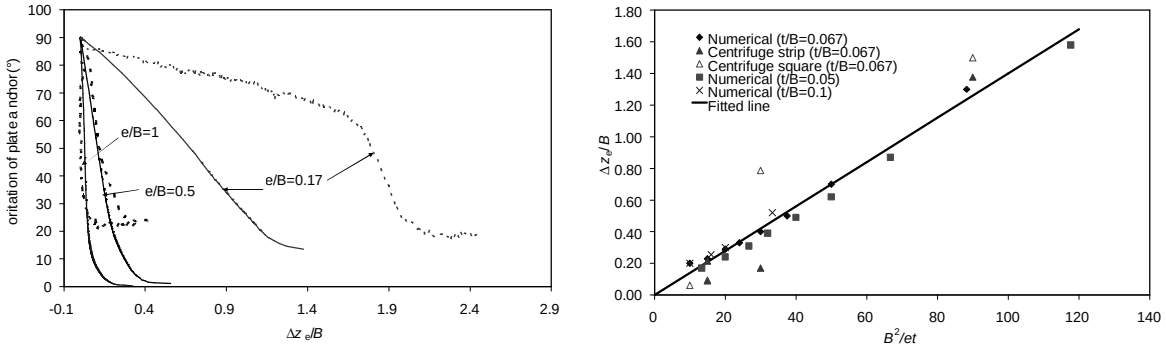



Figure 2: Numerical rotation setup




(a) (b)

Figure 3: Plate anchor rotation

Figure 3 plots the orientation of the plate angle (in degrees from the horizontal) against the loss in anchor embedment ($\Delta z_e$) normalised by the anchor breadth, $B$ (the solid curves). Evidently the loss in anchor embedment for $e/B = 0.17$ is much higher than either $e/B = 1.0$ to $e/B = 0.5$ and the lower the eccentricity ratio, the higher the loss of embedment. It is made clear in Figure 3(b), which plots the loss of embedment ratio versus eccentricity ratio. The centrifuge results from O'Loughlin et.al (2006) are also shown here (the dotted curve). It can be seen that the numerical results match the centrifuge results except $e/B = 0.17$. This is because the anchor reaching shallow embedment in the centrifuge test for

the anchor with $e/B$ = 0.17 during keying (O'Loughlin et al. 2006). It can also be observed that the final inclination fell to an orientation between 20° and 30° in the centrifuge tests while the anchors in numerical simulation were fully rotated, this is because the offset of the loading arm in the centrifuge test.

Based on Figure 3(b), a curve fitted formula can be developed to estimate the loss of anchor embedment during keying. This formula is expressed in Equation 3 below:

$$\Delta z_e/B = 0.014 \times B^2 / (e \times t) \quad (3)$$

## 2.2 TRANSPARENT SOIL TEST

In order to observe anchor rotation behaviour, tests were carried out in pre-consolidated transparent "soil" in the drum centrifuge located at the University of Western Australia (UWA). The UWA drum centrifuge has a diameter of 1.2 m and maximum acceleration level of 485 g. Technical details of the drum centrifuge can be found in Stewart et al. (1998). This transparent "soil" is made from fumed silica, paraffin and white spirit, has clay-sized particles and exhibits similar geotechnical properties to natural clay. A "strongbox" with a plexiglass window was built to allow visualisation of anchor rotation and soil flow mechanisms. A high-resolution digital camera located within the drum centrifuge channel and focused on the opaque side of the strongbox allowed digital images to be captured during the test. The setup for the transparent soil test in the drum centrifuge is shown in Figure 4. The detailed procedure for producing the transparent soil can be found in Gill (1999). The anchor was square with 4 m × 4 m in prototype with thickness $t$ = 0.2m under 100 g centrifuge test. In the drum centrifuge test soil shear strength was determined using a T-bar penetrometer (Stewart and Randolph 1994). The T-bar tests were performed at a rate of 1 mm/s so that undrained behaviour is obtained (Randolph and Hope 2004).

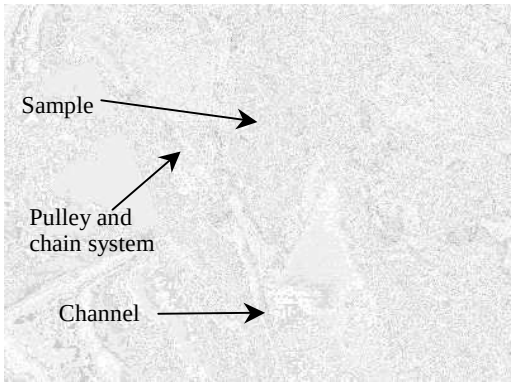



Figure 4: Transparent soil test setup

Figures 5 display the rotation of the anchor in this transparent soil test. Figure 5 (a), (b) display the chain movement during the first stage of the rotation. The arrows show the movement of points from previous positions to the current positions. Figures 5(b), (c) depict the anchor rotation after the chain has tightened. The anchor starts to rotate from Figure 5 (c). From Figure 5 (c), it can be seen the padeye of the anchor has moved vertically upward as the anchor rotates by about 42°. Tracking the movement of a row of beads that were pre-embedded in the transparent sample gives some indication of the soil movements in the vicinity of the rotating anchor. According to the trajectories of the beads, the soil close to the top edge of the anchor is pushed down and the soil close to the bottom edge of the plate is lifted, indicating soil flow around the edge of the anchor.

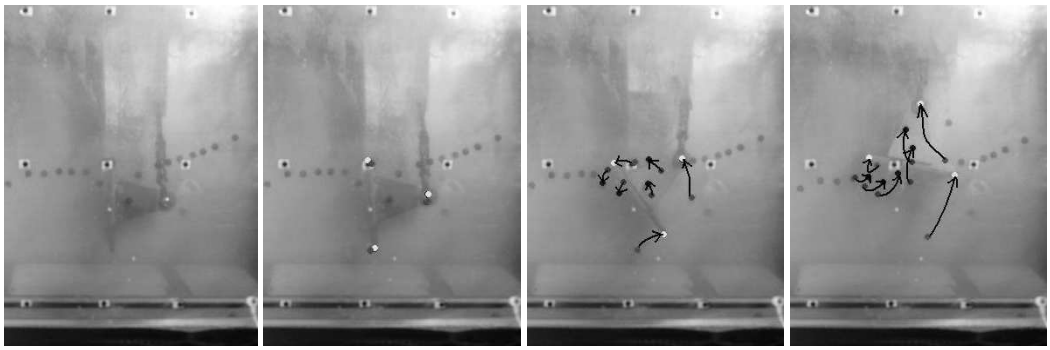


Figure 5 Anchor rotations in transparent soil test

From Figure 5 (c), the anchors continue to rotate until anchor is fully rotated to position (Figure 5(d)). After that, the anchor goes straight upwards although the pullout angle is not exactly 90˚. This is because of the set up of the test. In the set up of the test, it is impossible to set up precisely 90˚ pullout after the anchor rotation. The initial 90˚ is set up from the padeye of the plate anchor. After rotation of the anchor, due to the anchor eccentricity (2.5 m in prototype), the pullout angle is around 85˚. In terms of the loss in embedment during anchor keying, the anchor centre moves vertically upwards by 0.38 B (Figure 5(d)), which is a little bit lower than the result 0.44B predicted by equation (3). This is because the equation is a fitted trend line from all the centrifuge data and numerical simulation. The further numerical simulation for an anchor with same dimension indicates that the anchor would take 0.39 B rotate into position, which is only 0.01 B difference compared with the transparent soil data.

## 3. SEPERATION DEPTH

In separation depth analysis, both strip and circular anchors were studied. Anchors with fully attached base and vented base embedded horizontally were simulated to investigate the soil-plate separation phenomenon. Soil was uniform clay and modelled with friction and dilation angles $\phi = \psi = 0$. The undrained shear strength ratio were $s_u/\gamma' B$ ($s_u/\gamma' D$) = 0.02 ~ 0.7 for uniform clay.

Figure 6 displays the continuous pullout response of a circular plate anchor embedded in uniform soil with different initial embedment depth ($H_i$) in uniform soil with $s_u/\gamma' D$ = 0.07. The pullout capacity factor $N_c$ is calculated using Equation 4.

$$N_c = Q_u/As_u \tag{4}$$

Anchors with fully attached base and vented base are analysed with initial embedment ratios $H_i/D$ = 2, 1 and 0.5. For the anchor with fully attached base, it can sustain tension force while no tension force is allowed for the anchor with vented base. For the attached anchor with $H_i/D$ = 1, the pullout capacity factor is constant ($N_c$ = 13.7) during the initial pullout. It starts to decrease at the embedment $H_{SD}/D$ = 0.75, which marks the transition from deep embedment to shallow embedment. For the attached anchor with $H_i/D$ = 0.5, the pullout capacity factor reaches A curve instantly after the initial pullout; and then it approaches the large deformation curve for $H_i/D$ = 1.

When the anchor base is vented, the pullout capacity factor shows exactly the same response as the attached anchor during initial pullout. The soil-anchor separation occurs at the embedment depth $H_s/D$ = 0.3. The separation embedment depth ($H_s$) marks the transition from a full soil-anchor attachment to the soil breakaway at anchor base when suction is lost. The initial embedment depth has no effect on the separation depth. After the separation depth is reached, with further pullout, the $N_c$ number decreases linearly and more rapidly than that of attached anchor.

When the attached anchors are embedded initially at different depths, it is observed again that the pullout response soon joins the unique curve after the initial pullout. This unique curve for the attached plate can be expressed as below from curve fitting (A-curve in Figure 6 for a circular anchor):

$$N_c = 11.0 + 3.6(H/D), \text{ For } H/D > 0.75$$

$$N_c = 13.7, \text{ For } H/D \le 0.75 \qquad \text{5(a) for circular anchor}$$

$$N_c = 8.6 + 2.2(H/D), \text{ For } H/D > 1.40$$

$$N_c = 11.7, \text{ For } H/D \le 1.40 \qquad \text{5(b) for strip anchor}$$

The development of A-curve for strip anchor is similar to that for circular anchor, thus not shown here.

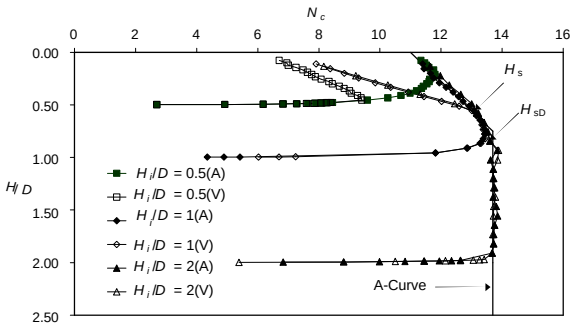



Figure 6 Breakout factors for horizontally embedded anchors

When the vented anchor is initially embedded above the separation depth, the pullout capacity of the anchor is quickly developed to the ultimate value; then the separation occurs immediately.

For the vented anchor embedded above the separation depth, the pullout capacity factor for a shallow anchor ($N_{csh}$) can be derived from the shear resistance of the shear plane above the anchor ($Q_{ushear}$) and the soil weight above the anchor ($Q_{usoil}$) based on the assumption of vertical slip planes extending up from the edge pf an assumed horizontal anchor. This can be expressed as the following:

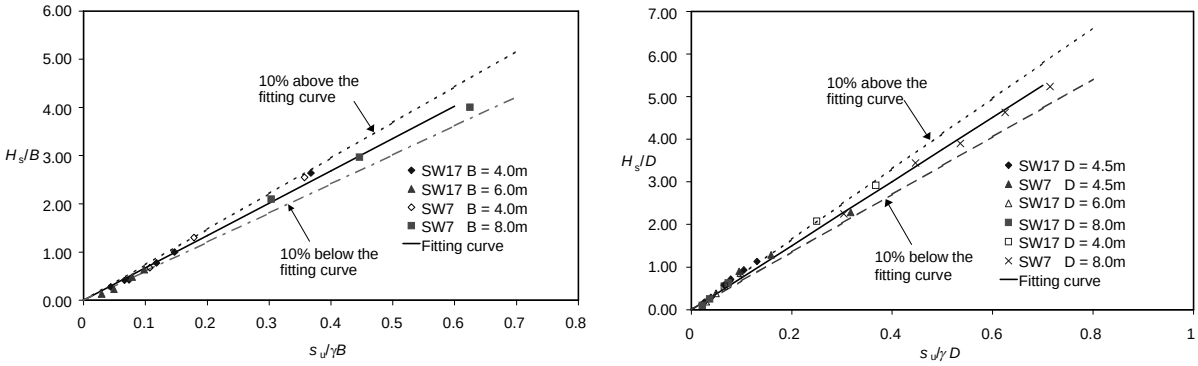
$$N_{csh} = (Q_{ushear} + Q_{usoil})/As_u = (\pi DHs_u + AH\gamma')/As_u = (4/D + \gamma'/s_u)H \qquad 6(a) \text{ for circular anchor}$$

$$N_{csh} = (Q_{ushear} + Q_{usoil})/As_u = (2Hs_u + AH\gamma')/As_u = (2/B + \gamma'/s_u)H \qquad 6(b) \text{ for strip anchor}$$

where $A$ is the anchor area and $\gamma'$ is the soil unit weight.

To examine the anchor size effect on soil-anchor separation depth ($H_s/D$ or $H_s/B$) and soil weight effect, more analyses are conducted with different anchor sizes $D(B)$, different soil weight $\gamma'$ and different $s_u$. The separation depth ratio ($H_s/D$) against a normalised shear strength ratio, $s_u/\gamma'D(s_u/\gamma'B)$, is plotted in Figure 7 for both strip plate anchors and circular plate anchors. It is apparent that the separation depth ratio ($H_s/B$ or $H_s/D$) increases linearly with increasing stability number ($s_u/\gamma'D$ or $s_u/\gamma'B$). The fitted curve can be expressed in Equation 7 below:

$$H_s/D = 7.51(s_u/\gamma'D) \text{ (Circular) or } H_s/B = 6.71(s_u/\gamma'B) \text{ (Strip)} \qquad (7)$$




(a) strip anchors (b) circular anchors

Figure 7 Separation depths versus normalised strength ratio

The correlation coefficient for both curves is $R^2 = 0.99$.

## 4. DESIGN STAGE

Design phase of plate anchors is suggested as:

1. Determine representative values of soil parameters $s_u$, $\gamma'$ from site investigation.
2. Design the anchor size $B$, eccentricity of anchor $e$ and embedment depth $H$ according to the installation procedure of VDPA or SEPLA, calculate the embedment ratio $H/B$, eccentricity ratio $e/B$ and thickness ratio $t/B$.
3. Estimate the loss of anchor embedment, $\Delta z_e/B$ using equation (3). Then the current embedment depth is determined by $H_c/B = H_i/B - \Delta z_e/B$.
4. Determine the separation depth, $H_s/B$, using equation (7).
5. If $H_c/B > H_s/B$, obtain bearing capacity factor, $N_c$, using equation (5) for attached anchor; if $H_c/B < H_s/B$, obtain bearing capacity factor, $N_c$, using equation (6) for vented anchor. If the values obtained from the two equations above don't equals to each other at same embedment, the lower value of $N_c$ should be adopted.
6. Calculate anchor holding force using Equation 4: $Q_u = N_c As_u$
7. Check if the holding force fulfills the design requirement, $Q_{required}$: If $Q_u > Q_{required}$, stop; Otherwise, design an anchor with new dimensions and go back to step 3.

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# APPLICATIONS OF SOIL THERMAL RESISTIVITY

**Chris Soutar**
*Maunsell Limited*
*Auckland, New Zealand*

## ABSTRACT

Soil thermal resistivity has a significant role in the design and laying of high voltage buried power cables. The cables generate heat which must be dissipated by the surrounding soil in order to avoid overheating and failure of the cables.

Soil thermal resistivity has been demonstrated to be complex and depends on various parameters such as soil type, particle size and compaction characteristics. This poses problems when attempting to estimate soil thermal resistivity by mathematical and empirical models.

In practise thermal resistivity is measured using laboratory and field thermal probes, and tests are used to design suitable thermal backfill materials for cables. Test results for projects in New Zealand are presented.

## 1. INTRODUCTION

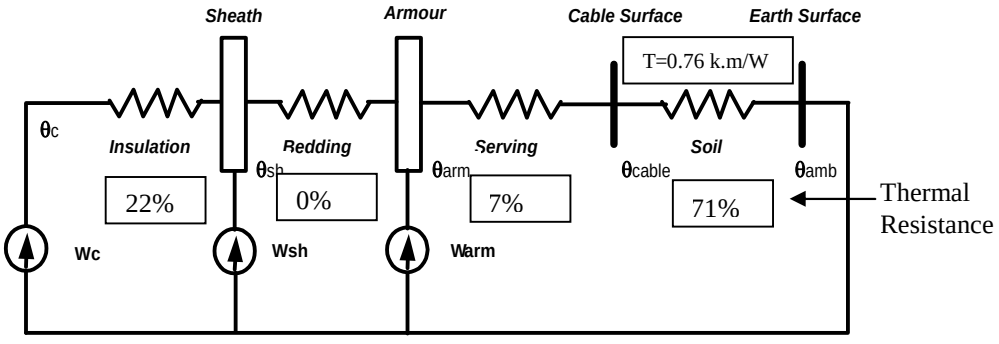
The assessment of the thermal properties of soil is problematic for many geotechnical engineers during design and laying of buried power cables. Soil Thermal Resistivity is the resistance offered by a soil mass in dissipating the heat generated by a cable, (Rao and Singh, 1999). An underground cable typically comprises a conductor encased in various layers of insulation, sheathing, bedding for the armouring and an outside sheathing. Heat is dissipated from the conductor through the various layers to the cable surface. Heat from the cable surface dissipates into the surrounding soil, typically finding its way to the ground surface. In a cable circuit diagram, soil immediately surrounding the buried power cable gives the largest thermal resistance and the greatest amount of variability with distance along the thermal path (Figure 1). A major difficulty in estimating the heat transfer into the ground from the cable sheath surface is the uncertainty involved in soil properties, especially soil thermal resistivity. Factors affecting modelling of soil thermal resistivity are soil composition, moisture content, dry density and texture. However if these factors are identified from testing and field investigations an empirical model for thermal resistivity can be adequately estimated.

Cable insulation degrades if the soil surrounding the cable is unable to conduct the heat away from the cable. It is this need to use soil as an insulating material to dissipate heat that requires an understanding of the soil thermal characteristics. This was demonstrated in 1998 when several of Auckland City's underground power cables catastrophically failed due to degradation of the cable insulation and thermal runaway.

Two methods have been developed by previous investigators to determine soil thermal resistivity. These are classified as: (1) an empirical model based on data obtained by measurement which is then analysed using numerical techniques, (2) theoretical methods based on approximate models using simplified soil structure to allow mathematical modelling.

Because of the variation in soil thermal resistivity encountered along cable routes, cable engineers are conservative in their selection of the values used to design buried power cables. Testing and analysis has determined that a good thermal resistivity for a soil is below 1.2 k.m/ W or °C.m/W. Cable routes are surveyed and tested to determine suitability and requirement for any manufactured thermal backfill such at Fluidized Thermal Backfill (FTB).

**Figure 1 (Circuit Diagram for a 33kV cable, buried in natural soil)**



# 2. FACTORS AFFECTING SOIL THERMAL RESISTIVITY

## 2.1 GENERAL

Soil thermal resistivity is primarily affected by soil composition, density, moisture content and texture. Other factors are the environmental influences and thermal stability.

## 2.2 SOIL COMPOSITION

A soil is a composed of solid mineral grains with voids between the grains. Thermal resistivity is a function of the resistivity of a soils components. Thermal Resistivity values vary greatly between different soil particles from 0.12°C.m/W for Quartz to 0.42°C.m/W for limestone and 5 °C.m/W for organic material (Parmer, 1998) (Figure 2).

The voids between soil particles are either filled with air or water or a combination of the two. Heat flows through soil by conduction from particle to particle and by conduction and convection through the moisture or air that occupies the pore space between solid particles (Parmer, 1998). If the soil is totally dry, the voids are filled with air. Air has a very high thermal resistivity and causes a dry soil to have a high thermal resistivity. Figure 3 indicates the paths that the heat takes through soil. When air occupies the voids, heat conduction paths become longer giving more resistance to the heat flow, thereby decreasing the capacity of the soil to conduct heat. This allows 'hot spots' to form around the cable where heat cannot escape through surrounding soil particles.

## 2.3 MOISTURE CONTENT

Moisture content has the greatest influence on the thermal resistivity of a given soil. When the soil mass is wet, the air (thermal resistivity approximately 4.5 °C.m/W) in the voids between the soil particles is replaced with water (thermal resistivity approximately 1.65°C.m/W), consequently lowering the thermal resistivity of the soil and improving heat dissipation.

When water replaces air in voids in the soil mass it creates thin films, and bridges around and between particles, allowing a path of less resistance for heat flow, which greatly reduces the thermal resistivity of the soil (Figure 3). As water content is increased, the effective contact area no longer increases, consequently a decrease in thermal resistivity is less evident with higher water content this is illustrated graphically in Figure 4. Heat generated by the cable tends to cause the water to migrate away (drying out); this is called thermal instability or thermal runaway and is discussed below.

**Figure 2: Soil components and soil thermal resistivity properties**

| Soil Components | |
|---|---|
| **Description** | **Thermal Resistivity Dry (°C.m/W)** |
| **Soil Grains** | |
| **Quartz** | **0.12** |
| **Granite** | **0.30** |
| **Limestone** | **0.40** |
| **Sandstone** | **0.50** |
| **Mica** | **1.70** |
| **Others** | |
| **Ice** | **4.5** |
| **Water** | **1.65** |
| **Oil** | **8.0** |
| **Organics** | **≈ 5.0** |
| **Air** | **≈ 45.00** |

**Figure 3: Conduction paths for a dry soil and a wet soil**

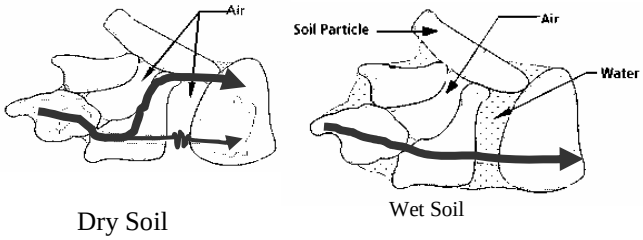
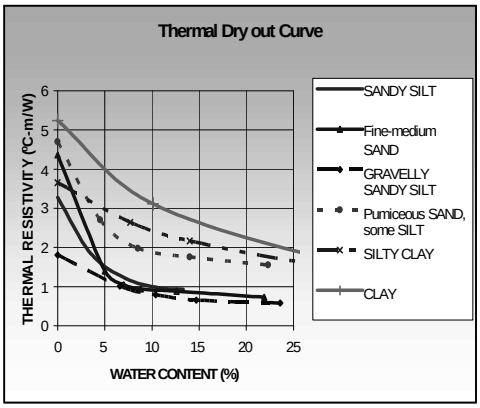



**Figure 4: Thermal Dry out Curve for a range of soils**



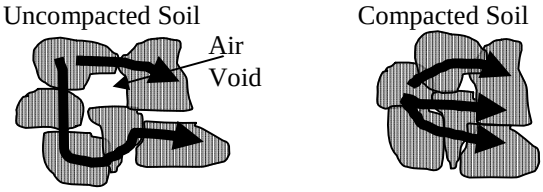
### 2.4 DRY DENSITY

Dry density is expressed as the ratio of the dry weight of the soil to the total volume of the soil mass (solid+air+water). The total volume is taken as the initial volume of the undisturbed moist soil.

Compaction closes voids improves particle contact and increases dry density, thereby lowering the thermal resistivity especially at low moisture contents. Ideally, well graded soils are used to surround cables as the higher soil density gives lower thermal resistivity.

The influence of density on thermal resistivity is shown by Figure 5. This figure indicates the effect of reducing the total area of voids to improve contacts between the soil grains, providing shorter heat paths and therefore reducing the thermal resistivity.

**Figure 5: Compacted and uncompacted soils with heat flow paths**




### 2.5 TEXTURE

Texture refers to soil grain size, shape and range of particle sizes. Since the majority of heat conduction is through particle to particle contact, the more contacts a soil has, the lower the thermal resistivity, as there are fewer voids. Soils with aggregates of a range of particle sizes and shapes will inherently maximise contact and yield lower thermal resistivities.

Figure 4 illustrates that fine grained, or cohesive, soils and peaty soils, have higher thermal resistivities than granular soils at a given moisture content. Therefore, fine grained soils and peaty soils would be expected to be less efficient at dissipating heat from buried power cables compared to granular soils

### 2.6 ENVIRONMENTAL FACTORS

Environmental factors can influence measured thermal resistivity. Insitu resistivity of a soil changes from season to season and can be due to the change in moisture content of the soil or due to a change in the water table. As moisture content can change regularly through the seasons, moisture content entered into mathematical and empirical equations when estimating TR should always be taken as the lowest possible.

### 2.7 THERMAL STABILITY

Heat generated by a cable causes migration of soil moisture away from the cable. In soils, this effect increases thermal resistivity, inducing further heating within the cable, which in turn dries the soil out even more. The eventual total drying of the soil results in excessive heating of the cable. This effect is known as thermal runaway or thermal instability.

Thermal instability is illustrated in thermal dry out curves (Figure 4). The critical moisture content is the moisture content below which the relativity flat section of the thermal dry out curve gives way to an uneven or rapid increase in thermal resistivity. Above this critical moisture, soil will resist thermal dry out by means of capillary suction; below this value thermal runaway occurs unless moisture is added externally to the system (e.g. Rain).

## 3. THERMAL RESISTIVITY MEASUREMENTS METHODS

### 3.1 GENERAL

There are a number of techniques and instrumentation to measure thermal resistivity which employ steady state or transient methods. The most common steady state method is the ‘guarded hot-plate’, where the transient heat method uses cylindrical heat source or thermal needle. The thermal needle (line source method) is accepted by IEEE Standard 442 (1981) and is used due to field and laboratory practicality.

Insitu thermal resistivity values are determined by tests using an approximately 300mm long by 6mm diameter thermal probe with the Geotherm TPA-7000 or other processor running off either a portable generator or a car battery. The probe is tapped or pushed in to the undisturbed soil and power is applied to the probe heater. Soil resistivities are

recorded by the processor as the probe heats up. A smaller thermal probe is used in the same way for tube samples and bulk samples.

Laboratory testing involves conducting resistivity tests with the soil at 'as received' moisture content and at various stages as the soil is dried to zero water content. Dying involves placing the sample in an oven. After each drying stage, the sample is allowed to cool and moisture levels allowed to equilibrate before thermal resistivity in measured. The thermal dry out curve of thermal resistivity versus water content is then plotted, (Figure 4).

### 3.2 THERMAL NEEDLE METHOD

The thermal needle method (Figure 6) also known as thermal probe is based on the measurement of the rate of temperature rise along a line heat sourced within an infinite homogenous medium. The equation to calculate thermal resistivity is (1),

$$(1)\quad \rho = 4\pi \frac{(T_2 - T_1)}{2.303q\log(\frac{t_2}{t_1})}$$

Where $\rho$= resistivity °C cm/W; $T_1$= temperature measured at some arbitrary elapse time (Degrees Celsius); $T_2$ = temperature measured at another arbitrary elapsed time (Degrees Celsius); $q$ = heat dissipated per unit length (W/cm); $t_1$= elapsed time at which a temperature measurement was recorded (min); $t_2$= elapsed time at which another temperature measurement was recorded (min). (IEEE, 1981)

The thermal needle / probe apparatus is relativity simple with ease of use both in the field and the laboratory (Photo 1). It has the advantage of calculating soil thermal resistivity in the field with little knowledge of the heat capacity.

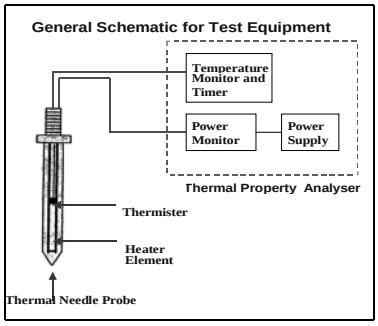



**Figure 6: General Schematic for Thermal Needle test Equipment**

**Photo 1: Test Equipment in action**

### 3.3 FACTORS AFFECTING MEASUREMENTS

Soil moisture can migrate away from the thermal needle during the test and can result in both higher and lower measurements. Moisture migration can be significant and is caused when the power input per unit area into the thermal needle is too high. Moisture migration can take place toward the end of the test when heats are the greatest resulting in apparent increases in soil thermal resistivities.

Under lab conditions, measurements of soil thermal resistivity can be affected by the relocation of moisture due to gravity, resulting in higher thermal resistivities.

Other small effects on the needle method include improper installation of the thermal needle, incorrect selection of the cable depth and power supply stability.

## 4. ELEMENTS OF A CABLE ROUTE SURVEY

### 4.1 REVIEW AND PLANNING

The first step in a cable route survey is to review all available geological and ground condition data along the proposed route. This will include information from government agencies, other utilities in the surrounding area and borehole logs for any existing structures. Using this information, test locations and distance between them can be determined. If soil conditions are uniform, test numbers can be reduced.

Borehole locations will need to be determined and marked, underground services checked and identified on site and any consents obtained. Thermal needles are installed in a number of predug holes. Site access needs to be assessed to minimise damage to the ground.

Soil thermal testing should be carried out at least every 800m depending on the length of the cable route (Williams *et al*, 1992).

As soil thermal resistivities are moisture dependant, every effort should be made to determine the soil thermal resistivity during the driest months of the year.

### 4.2 FIELD TESTING

Where a deep cable is proposed, the field investigation should use a drilling rig. Insitu thermal resistivity tests and Shelby tube sampling is carried out at appropriate depths. Water level should be recorded and the drillhole logged during drilling. For shallow cables, excavated pits can be used instead of drillholes.

At each testing location, field soil descriptions and insitu measurements of natural soil thermal resistivities and ambient temperatures (using the thermal needle) at several depths down to the proposed cable depth are made.

Undisturbed samples, generally thin wall Shelby tubes, should be taken above and below the cable depth. When a Shelby tube cannot be taken, bulk samples or standard penetration test samples can be used.

### 4.3 LABORATORY TESTING

In the laboratory, soil samples are tested to determine moisture contents, dry densities organic content, thermal dry out curves and critical moisture content. Atterberg tests and sieve analysis can also be carried out to characterise soil properties.

### 4.4 ANALYSIS

Once field and laboratory test results have been completed, comparisons and inconsistencies can be determined. Using the laboratory thermal dry out curve, the laboratory results can be compared to the field results. Further sampling may be required to resolve any inconsistencies.

From these results the lowest expected ambient soil moistures are determined and the resistivity for the natural soils is chosen from the thermal dry out curve.

## 5. CORRECTIVE THERMAL BACKFILLS

### 5.1 GENERAL

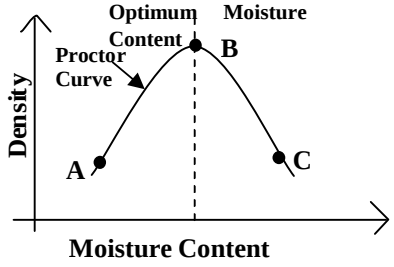
Corrective thermal backfills can significantly enhance the heat dissipation of a buried cable**.** Corrective thermal backfills have higher costs than natural or unclassified materials, but the benefits can be significant as a possible increase in cable ampacity increases revenue.

### 5.1 NATURAL SOILS

Soils do not typically make good thermal backfills because of the difficultly in re-compacting them to a suitable density. They also may have natural high thermal resistivities. In the natural state, soil may have good thermal resistivity or dryout curve, but it may not have the right texture and moisture content, or could be hard to handle, resulting in a poor thermal dryout curve on backfilling. Contamination of the natural soil can also occur with topsoil and organics, further compromising its suitability.

### 5.3 COMPACTED GRANULAR BACKFILLS

Well graded sand and gravels screening, without organic material make ideal thermal backfill. For installation and protection of the cable sheath, a maximum particle size of 7.5mm is recommended. The fines content should be between 8% and 15% (Parmer, 1998). This is to ensure that the material has good particle contacts and the porosity is low, which enhances heat transfer. In general, materials are installed at a density close to that obtained at standard Proctor compaction and at optimum moisture content (Figure 7).




A: Compacted dry of optimum moisture content. High porosity and low degree of saturation. Higher thermal Resistivity.

B: Compacted at optimum moisture content. Lowest porosity and medium to high degree of saturation. Low Thermal Resistivity.

C: Compacted wet of optimum moisture content. Higher porosity and higher degree of saturation. Medium thermal resistivity.

**Figure 7: Proctor Curve for Thermal Backfill**

### 5.4 CONCRETE

Concrete has good thermal resistivity, however it does not flow readily and costs more than Fluidized Thermal Backfill. The main disadvantage is that it can not be removed easily for access and maintenance of the cable.

### 5.5 FLUIDIZED THERMAL BACKFILL

Fluidized thermal backfill (FTB) is a material engineered to meet a specific thermal resistivity, thermal stability, strength and flowability criteria. FTB offers many construction advantages. It is a free flowing (high slump) concrete-like backfill consisting of a stone aggregate, sand, cement, and flyash (for fluidization).

FTB fills voids and hardens quickly to a uniform density, has negligible settlements and supplies support for underground services.

FTB can be supplied as a ready mix in concrete mixing trucks and can be installed using standard concrete production equipment. Unlike compacted granular thermal backfills, the cable sheath is not damaged during installation of FTB due to the absence of mechanical compactors. Its removal as it is a very low strength concrete is relativity easy for future cable repairs.

## 6. APPLICATIONS AND TEST RESULTS IN NEW ZEALAND

New Zealand had one of the more well known cable failures due to thermal instability. At the beginning of 1998 Auckland suffered a complete failure of 4 cables supplying the CBD. Two were 40 years old, oil-filled cables which were past their replacement date. Auckland was in the middle of a very dry summer. The first of the old cables failed on the 20th January and the second on 9th February. The two remaining cables, failed on the 19 and 20th February, leaving the central business district without power for over 1 month. The latter two cables failed due to increased ampacity because the central business district was being supplied via 2 cables instead of 4, and because of thermal runaway. The backfill materials around the cables were sandy and dried out during the summer. Since the Auckland power crisis many major New Zealand cities have conducted TR testing on their buried power cables to ensure similar failures will not occur. Results indicated that New Zealand has many cables buried in soil with a thermal resistivity greater than the recommended 1.2°C.m/W (Figure 8).

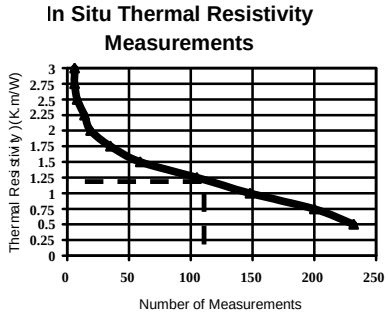



**Figure 8 Results from New Zealand Soil Thermal Resistivity Testing (Maunsell investigation results)**

## 7. CONCLUDING REMARKS

The capacity of buried power cables is being pushed to the limit by the ever increasing demand for electricity. As extra load is put onto these cables, greater amounts of heat are being generated. This heat needs to be dissipated through the surround soil, requiring a greater understanding of soil thermal resistivity. Soil thermal resistivity plays a major role in the design and laying of buried power cables, as the soil resistance of a cable circuit makes up approximately 70% of the total resistance in the cable, by far the biggest factor. The factors affecting soil thermal resistivity are moisture, dry density, texture and composition. Currently the method for measuring soil thermal resistance is the thermal probe or needle. This apparatus can be used both in the field and in the laboratory and its ease of use means it has been accepted by the IEEE. Once testing has been completed thermal backfills can be designed if required. Thermal backfills can be cement, granular or fluidized thermal backfill. In 1998 the importance of soil thermal resistivity for underground cables was highlighted when Auckland's central business district went without power for 5 weeks after the 4 cables supplying the CBD failed due to thermal instability. Specialists in the power industry need to understand soil thermal resistivity to ensure that buried cables will operate as intended.

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# SETTLEMENT OF EMBANKMENTS ON SOFT SOIL

**Sarah Stapleton**
*GIPENZ*
*Tonkin & Taylor Ltd., Wellington, NZ*

## ABSTRACT

Predicting the magnitude of settlement under an embankment is an important aspect in the design of these structures. There are a number of methods that are used to extrapolate settlement data and make predictions of the final settlement. This research focuses on three such methods: the Asaoka method, the hyperbolic method and the log(1-U) method.

The objective of the research is to compare and evaluate the methods and to determine how much settlement data needs to be acquired before accurate predictions of final settlement can be made. The methods are compared by applying them to data from the theoretical Terzaghi settlement curve and to field data gathered at a site where embankments were constructed on compressible soils.

From the research it was determined that for all three methods to be used accurately, the degree of consolidation (U) must reach about 0.7.

## 1. INTRODUCTION

Constructing embankments on compressible soils poses many challenges. Settlement of compressible soil under an embankment can continue for years. Long settlement durations are impractical for the planning and construction of roads and other structures on embankments.

Predicting the magnitude of settlement and the settlement duration is critical to the design of embankments. Estimates of final settlement can be made prior to construction from ground information and oedometer testing. Monitoring settlement as it takes place is used to confirm that settlement is ocurring as predicted. Predictions of final settlement can be made from measured settlement data as settlement progresses to confirm if the design assumptions are correct.

The objective of this research is to assess and evaluate three methods of predicting settlement under embankments from measured settlement data. The methods are assessed in terms of their accuracy at predicting final settlement and the degree of consolidation that needs to be reached before accurate predictions of final settlement are able to be made.

## 2. CONSOLIDATION THEORY & METHODS OF SETTLEMENT PREDICTION

### 2.1 TERZAGHI THEORY

The Terzaghi theory of one-dimensional consolidation is the most widely used theory of consolidation. The theory is based on the assumption that when a load is applied to a saturated soil it is first carried by the water in the soil. This leads to a build up of pore water pressure. After time the pore water pressure is gradually dissipated through drainage and the load is transferred from the water to the soil, leading to a reduction in the volume of the soil. There are many factors that influence this process including the coefficient of consolidation, $c_v$, the coefficient of compressibility, $m_v$, and the soil permeability, k. Terzaghi's theory is based on a number of assumptions including: the load is applied instantaneously to a uniform soil, consolidation only takes place in the vertical direction; permeability (k) and volume compressibility ($m_v$) remain constant and no secondary compression occurs. While these assumptions may not always be valid, the theory can be used satisfactorily in most cases.

## 2.2 METHODS

There are numerous methods for predicting settlement under embankments. There are generally two categories of methods: graphical and observational. Traditional graphical methods for estimating $c_v$, such as the log time method and the root time method involve graphical constructions to settlement time curves. Observational methods use past observations of settlement to predict future settlement. Three such observational methods are used in this research: the Asaoka method, the hyperbolic method and the log(1-U) method.

### 2.2.1 ASAOKA METHOD

The Asaoka method is an observational method of predicting settlement of clay after Asaoka (1978). The method is applied to constant loading settlement data at regular time intervals.

Each settlement data point ($s_j$) is plotted against the previous data point in the series ($s_{j-1}$). The plotted points follow a linear trend (see Figure 2.1). The equation of the best fit line through the points is of the form $s_j = \beta_0 + \beta_1 s_{j-1}$ where $\beta_0$ and $\beta_1$ are the intercepts with the $s_j$ axis and the slope of the line respectively. The final settlement, $s_f$, can be estimated by extrapolating the line until it intersects the diagonal bisectors of the axes where $s_{j-1} = s_j = s_\infty$. A value for the coefficient of consolidation $c_v$ can be calculated using the following formula where $\beta_1$ is the slope of the Asaoka plot, $\Delta t$ is the time interval of the settlement data and H is the drainage path length.

$$c_v = -\frac{1}{6}\frac{H^2}{\Delta t}\ln \beta_1 \qquad (1)$$

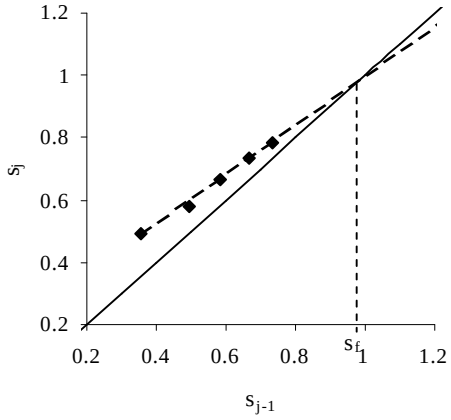


Figure 2.1 Asaoka Plot

### 2.2.2 HYPERBOLIC METHOD

The hyperbolic method is a method of predicting primary settlement based on Terzaghi's theory detailed in Sridharan et al. (1987). Like the Asoaka method, it is used for constant loading settlement data. It is based on the observation that a rectangular hyperbolic relationship exists between the degree of consolidation, U, and the dimensionless time factor, T. When T/U is plotted against T, the plot becomes linear between U=0.6 and U=0.9. It is this part of the plot that is used in the analysis. When the hyperbolic method is applied to real settlement data t/s is plotted against t, where s is settlement measured at time, t. This plot follows a linear form as shown on Figure 2.2. The coefficient of consolidation, $c_v$, can be calculated using the following formula where m' and c' are the slope and intercept of the t/s versus t plot respectively.

$$c_v = 0.297\frac{m'H^2}{c'} \qquad (2)$$

Sridharan et al. also derive an expression for the magnitude of primary settlement, $s_f$:

$$s_f = \frac{0.8208}{m'} \qquad (3)$$

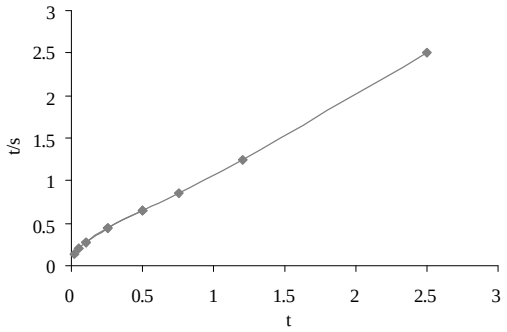


Figure 2.2 Example of Hyperbolic Plot

### 2.2.3 LOG(1-U) METHOD

The log(1-U) method is based on an approximation of the Terzaghi curve and was first proposed in Creed (1999). The Terzaghi curve is often approximated for U>0.6.as:

$$T = -0.933\log(1-U) - 0.085 \qquad (4)$$

While the approximation is used for U>0.6, it also gives a reasonable approximation of the curve for U as low as 0.2. It can be seen from the equation that log(1-U) varies linearly in relation to T. Rearranging that equation yields

$$\log(1-U) = -\frac{T}{0.933} - 0.085 \qquad (5)$$

The method can be applied to field data to find $c_v$ and the final settlement, $s_f$. Substituting $s/s_f$ for U and $c_v t/h$ for T in the above formula yields

$$\log(1-\frac{s}{s_f}) = -\frac{c_v t}{0.933h^2} - 0.085 \qquad (6)$$

The method can be used to estimate final settlement by plotting $\log(1-s/s_f)$ against t. The value of $s_f$ must be guessed as it is unknown. Different values of $s_f$ are used in a trial and error manner in order to make the $\log(1-s/s_f)$ versus t plot as linear as possible. The value of $s_f$ for which the plot is most linear is the predicted value for $s_f$. The value of $c_v$ can be calculated from the slope of the linearised $\log(1-s/s_f)$ versus t plot.

## 3. APPLYING THE METHODS

The methods of predicting final settlement are applied to two different types of data in order to determine their accuracy. Firstly the methods are applied to the theoretical Terzaghi curve to determine how they perform given ideal settlement data. The methods are then applied to field settlement data. The purpose of analysing two kinds of data (theoretical and field data) is to assess whether the methods perform in a consistent manner in both cases.

### 3.1 THOERETICAL DATA

The theoretical Terzaghi curve is a plot of the degree of consolidation, U, against a dimensionless time factor, T. The curve is generated from Terzaghi's equation. The consolidation curve rises from U=0 to U=1 as T increases, with about 50% consolidation achieved at T=0.2.

The methods are applied to different ranges of U in order to assess the accuracy of the settlement predictions and the degree of consolidation required before accurate predictions are made. The advantage of using the theoretical data is that the final degree of consolidation is known to be 1, therefore, the accuracy of each method can be easily determined by comparing the predictions it makes to 1.

Figure 3.1 shows the results from each method when applied to different ranges of U. The degree of consolidation on the x-axis represents the range of data from U=0 used to make the prediction of final consolidation. From the figure it can be seen that at low ranges of U all three methods give inaccurate predictions of final consolidation. When data up to 40% consolidation are used, the Asaoka method and the log(1-U) method underestimate final consolidation by almost 50% and the hyperbolic method by over 60%. As the degree of consolidation increases to 60%, the accuracy of the predictions of final consolidation increases. The log(1-U) method is slightly more accurate than the Asoaka method over most ranges of U. The hyperbolic method is the least accurate when applied to the theoretical curve.

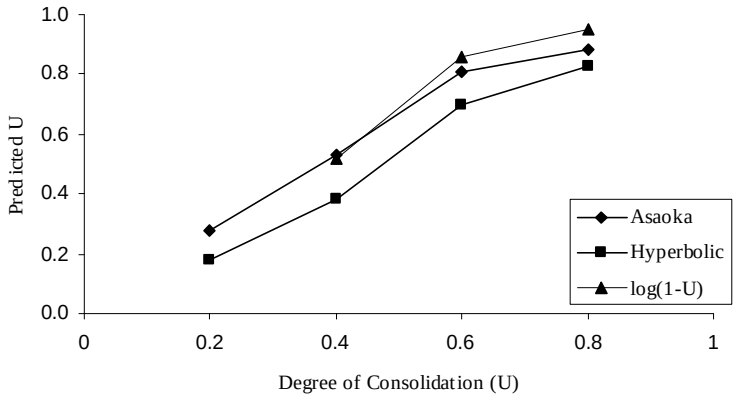


Figure 3.1 Accuracy of predictions when applied to theoretical data

### 3.2 KINSALE FIELD DATA

Field data from an embankment near Kinsale in Ireland were analysed using the three methods. The purpose of this is to predict settlement in real situations and in real conditions. The accuracy of the methods was assessed by applying them to the theoretical curve and oedometer data. However, the accuracy of the methods in practice can only be determined by applying them to field settlement data. The analysis of field data from Kinsale is summarised in the following section.

The Kinsale embankments were constructed as part of a road realignment project about 20 km south of Cork City in Ireland. The embankments were constructed in stages. The loading history and settlement curve for one of the settlement plates are shown in Figure 3.2. During the first loading stage the embankment was raised to about 5m for 575 days. The embankment was then raised to 8m for 140 days. A surcharge was put in place by raising the embankment to approximately 10.3m for 90 days. The embankment was then lowered to its finished height of 9.2m. Settlement measurements continued for almost a year following removal of the surcharge.

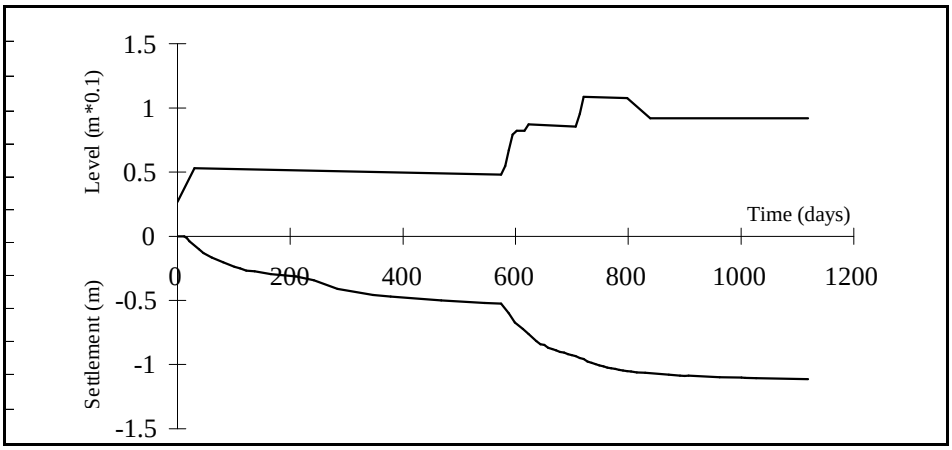


Figure 3.2 Loading history and settlement curve from Kinsale site (plate C)

The embankments are constructed on estuarine mudflats. Site investigations revealed up to 6.8m thickness of soft grey silt overlying weathered shaley bedrock. Laboratory oedometer tests on samples of the silt determined the coefficient of consolidation, $c_v$ to be 8-20 $m^2$/year and the compression index, $C_c$ to be 0.25-0.5.

The settlement data from the Kinsale embankment are analysed using the Asaoka method, the hyperbolic method and the log(1-U) method. The methods are applied to constant loading stages of the settlement time curve.

### 3.2.1 Results

The results achieved by each method for loading phase 2 are shown in Figure 3.3. As the degree of consolidation increases, the magnitude of the settlement predictions by each method and their accuracy increase. The hyperbolic method produces the largest predictions of settlement, followed by the log(1-U) method and the Asaoka method. In the analysis of the theoretical curve, the hyperbolic method produces the smallest predictions.

In order for the methods to provide accurate predictions of final settlement, the degree of consolidation reached needs to be quite high. From the analysis of the Kinsale field data, it is clear that none of the methods provide accurate forward predictions of final settlement when data from the early stages of consolidation are used. The degree of consolidation must reach approximately 0.7 before accurate predictions of final settlement are made by all three methods.

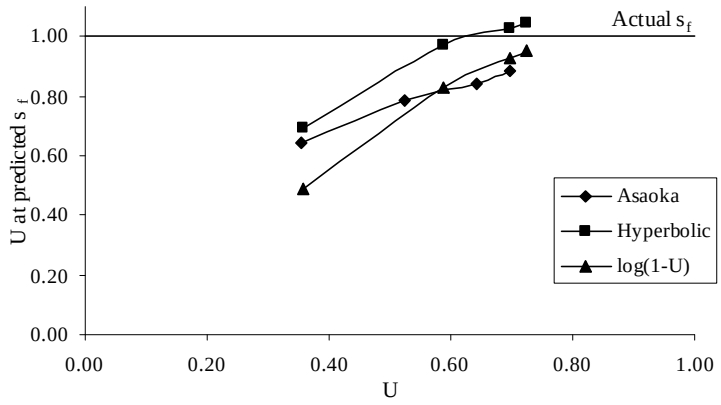


Figure 3.3 Accuracy of predictions when applied to field data (Plate C, loading phase 2)

## 4. CONCLUSIONS

The objectives of this work are to evaluate the Asaoka method, the hyperbolic method and the log(1-U) method of settlement analysis and to determine what degree of consolidation must be reached before accurate predictions of final settlement can be made.

In general the hyperbolic method produces the largest predictions of final settlement followed by the log(1-U) method and the Asaoka method. In the analysis of the theoretical curve each of the methods exhibits an increasing trend as the amount of settlement-time data used for the analysis is increased. However, when the methods are applied to field data the trends seen in the analysis of the theoretical curve are not always followed.

From the analysis of the theoretical curve it is concluded that the degree of consolidation must reach at least 0.6 for the Asaoka method and the log(1-U) method to produce accurate predictions of final settlement and in excess of 0.6 for the hyperbolic method to do the same. This conclusion is not exactly borne out when the methods are applied to field data, however. In those examples, the degree of consolidation frequently has to exceed at least 0.7 before accurate predictions of settlement are made. This means that the usefulness of the methods for estimating settlement is questionable because settlement has to be at an advanced stage before they make accurate predictions.

It is recommended that all three methods be used when making estimates of final settlement from field data. If the predictions made by all three methods are in good agreement it is a sign that the settlement data extends to a high degree of consolidation. If there are large discrepancies between the predictions from each method, the degree of consolidation reached, as reflected by the data, may not be high enough and further settlement measurements should be taken until the predictions from each method are in better agreement.

## 5. ACKNOWLEDGEMENTS

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# HORIZONTAL DISPLACEMENT OF LATERALLY LOADED PILES NEAR A SLOPE

**Laith Tapper**
*Arup Geotechnics, Perth, Western Australia*

## ABSTRACT

Masts supporting overhead electrical lines for railway lines must have sufficient stiffness to limit the lateral deflection of these lines when subject to wind loads. The lateral deflection of the overhead line arises principally from structural deflection of the mast itself and from lateral translation and rotation of the mast foundation. A special case arises when the mast foundation is constructed in the vicinity of embankment crest line since the presence of the embankment reduces the overall foundation stiffness. For this case, methods of determining horizontal displacements presented in the literature were brought together with numerical analyses to quantify the impact of the presence of an adjacent embankment slope on rotation and deflection of a short pile foundation. This in turn enabled the required depth of the foundation to be determined under serviceability load conditions.

## 1. INTRODUCTION

Arup Geotechnics was engaged by New MetroRail and RailLink Joint Venture to provide geotechnical services for the Perth to Mandurah Railway in Western Australia. These services included the design of short piles to support masts carrying overhead electrical lines. A key design criterion for the overhead lines was a limit on their lateral deflection to ensure that they remained in contact with the pantograph under serviceability wind load conditions.

Along certain sections of the railway line, mast foundations were required to be constructed in close proximity to the crest line of embankments. Hence the impact of an adjacent slope on the foundation performance was required. Methods of determining horizontal displacement presented in the literature were brought together with numerical analysis to facilitate an economical design solution.

The analysis of pile response to lateral loads has received wide attention. Subgrade reaction methods (McClelland and Focht, 1956; Broms, 1964; Reese et al., 1974) and elastic theory methods (Spillers and Stoll, 1964; Poulos, 1976) are well understood and are commonly used in the design of laterally loaded piles. However, the literature has focused primarily on the horizontal ground surface condition and relatively little has been reported on the instance where a pile is located next to a slope or embankment.

## 2. PILE FOUNDATION DESIGN DETAILS

### 2.1 DESIGN REQUIREMENTS

Serviceability requirements were found to be the dominant design consideration for determining the pile embedment depth. Furthermore, a design solution was needed that could be used for mast foundations along the 70km length of the railway. This necessitated the development of design charts which would take into account the soil condition and loading for each pile so as to determine the embedment depth which would meet the serviceability requirement. The design charts created for this purpose were based on the case of a horizontal ground surface, as this was the predominant ground condition. A modification was required that could be applied to the design charts to account for the instances in which the foundations were positioned adjacent to a slope.

### 2.2 FOUNDATION ARRANGEMENT

The foundation was to be designed as a bored concrete pile. The masts were made from precast concrete and were typically embedded into the concrete foundation. The diameter of the pile foundation for concrete masts was set at 700mm to accommodate the boring installation equipment and allow for fixing of the precast concrete mast.

The arrangement of the piled mast foundation adjacent to a slope is depicted in Figure 1. The foundation could be at the crest distance ‘x’ from the slope. The inclination of the slopes along the railway alignment was typically 1(v) in 2 (h). In analysis, the length of the piles was considered to be equal to, or less than the height of the slope.

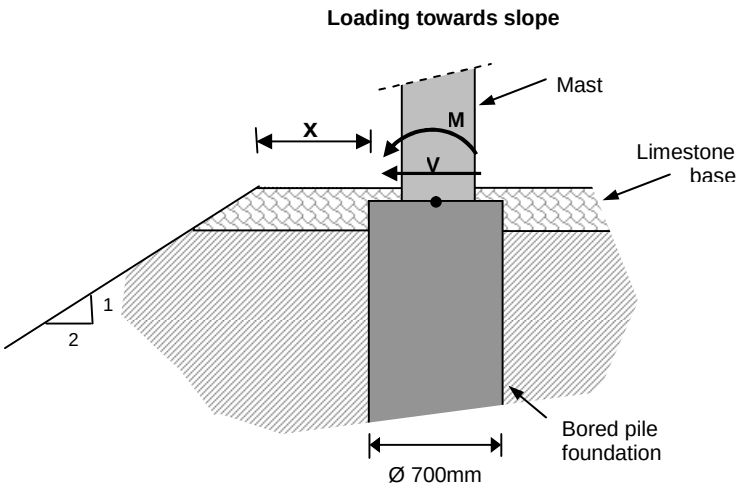


Figure 1: Arrangement of pile foundation adjacent to a slope

## 2.3 LOAD CASES / DEFLECTION CRITERIA

The serviceability loads acting at the top of the pile foundations for the concrete masts consisted of a bending moment of approximately 56kNm and a shear force of approximately 9kN. The design loads act concurrently at the top of the foundation level as shown in Figure 1. Vertical loading on the foundation was non critical in terms of serviceability or ultimate limit state bearing capacity. For the mast foundations adjacent to a slope, the loading could be either towards or away from the slope. In this paper, attention has been concentrated on the case of loading toward the slope because the overall soil resistance is less for this case due to the presence of the slope.

The total permissible lateral deflection of the overhead lines was 50mm with 60% of this to be a result of deflection from the mast (structural deflection). The geotechnical component of deflection was therefore limited to 20mm at mast contact wire level which was 5.66m above ground level.

## 2.4 GEOTECHNICAL CONDITIONS AND PARAMETERS

The in-situ ground conditions along the alignment of the railway consist predominately of sand although limestone was present at certain sections along the track. Cohesive soils were not expected to be encountered.

The bedding for the railway tracks consisted of an imported limestone base above a compacted sand sub-base. The relative density of the in-situ sand below the sub-base was found to vary with location along the rail alignment. For analysis purposes, this sand layer was considered to be either loose, medium dense or dense and to be cohesionless. The generic soil profile adopted for the mast foundation analysis is presented in Table 1.

| Depth (m) | Description | Relative Density |
|---|---|---|
| 0.0 – 0.2 | Limestone base | Dense |
| 0.2 – 0.7 | Sand sub-base | Dense |
| 0.7 onwards | In-situ sand: | |
| | Case 1 | Loose |
| | Case 2 | Medium dense |
| | Case 3 | Dense |

Table 1: Generic Soil Profile

# 3. LITERATURE REVIEW

The purpose of the literature review was to investigate how (1) slope inclination; and (2) distance of the pile foundation to the slope crest influences the horizontal displacement of a pile adjacent to a slope.

## 3.1 SLOPE INCLINATION

A number of investigations have considered the influence of slope inclination on the horizontal pile displacements caused by the applied lateral loads. Uto et al. (1985) conducted three field tests on piles, two in soft rock slopes and one in a sandy slope. Terashi et al. (1991), Bouafia and Bouguerra (1995), and Mezazigh and Levacher (1998) performed centrifuge model tests in cohesionless sand slopes whilst Schmidt (1977) performed tests on a scaled model. Zhang et al. (2004) and Chae et al. (2004) both undertook numerical studies.

Results reported from these experimental and numerical tests are given in Figure 2 for a pile located at the crest of a sand slope of a given steepness. This figure shows the relationship between slope inclination and the ratio of the displacements of the pile head in sloping ground ($S_{slope}$) and of the reference pile in level ground ($S_{level}$) subjected to the same lateral load. The displacement ratio is seen to increase markedly when the slope is steeper than 1(v):2(h) and approaches unity when the slope becomes flat. At a slope steepness of 1(v):2(h), the displacement ratio was found to be in the range of 1.6 to 1.8. Figure 2 also presents the results of numerical analysis that was conducted for this paper. These results are discussed in Section 4.

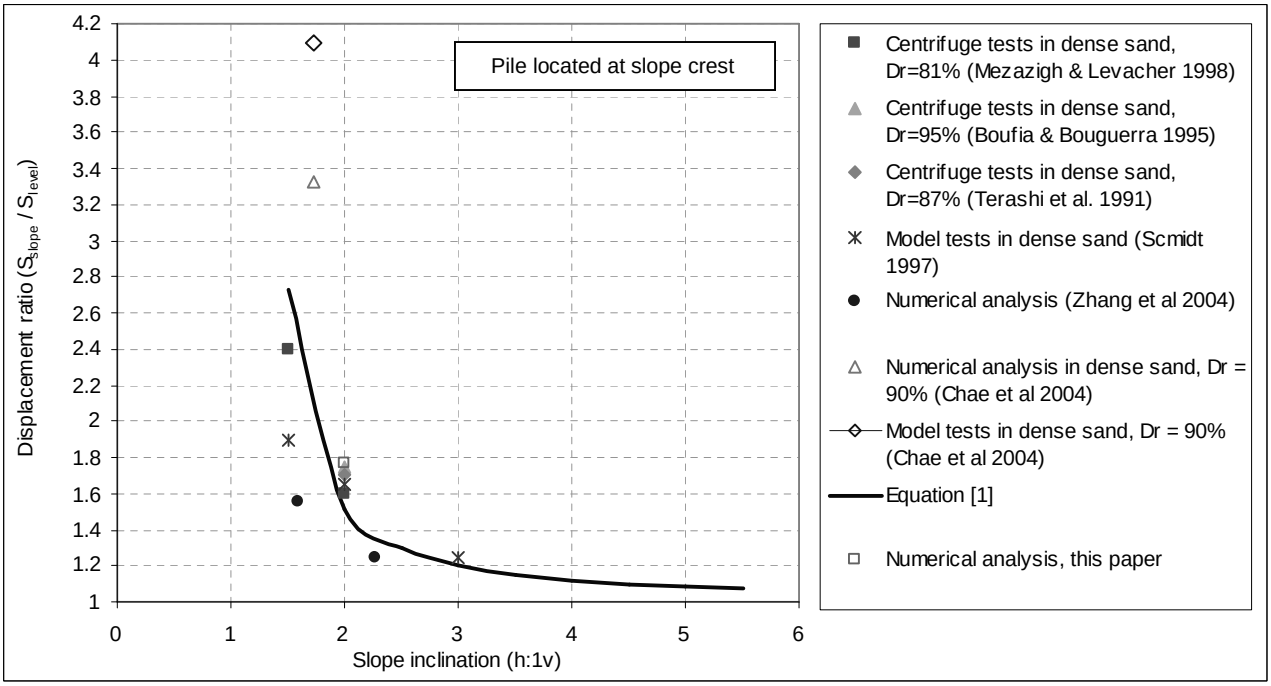


Figure 2: Effect of slope inclination on displacement at pile head

Zhang et al (2004) proposed the following empirical equation for the relationship between slope inclination and the displacement ratio:

$$\frac{S_{slope}}{S_{level}} = 1 + \frac{k_s}{\frac{\tan\phi'}{\tan\alpha} - 1} \tag{1}$$

where $\alpha$ is the slope angle, $\phi$' is the friction angle of the soil and $k_s$ is a curve fitting coefficient. This equation is plotted in Figure 2 (with $k_s$ = 0.3 and $\phi$' = 38 for a dense sand) and is seen to be provide a reasonable fit with experimental results.

### 3.2 DISTANCE OF PILE FROM SLOPE CREST

Centrifuge modelling by Mezazigh and Levacher (1998), Bouafia and Bouguerra (1995) and Terashi et al. (1991) have investigated the effect of the distance of the pile to the slope crest on the horizontal pile displacements caused by the applied lateral loads. Figure 3 presents the results from these studies on the effect of the distance of the pile to 1(v):2(h) slope on the displacement at the pile head. The deflection at the top of the pile was found to increase as the pile approaches the slope although there was some variation in the literature with regard to the degree of pile head displacement. Mezazigh and Levacher (1998) and Bouafia and Bouguerra (1995) recorded similar results in that they showed the effect of the slope becoming minimal at a pile distance of 4B away. However, Bouafia and Bouguerra (1995) still experienced a displacement ratio of approximately 1.5 at this distance. A possible explanation for this disparity was not determined.

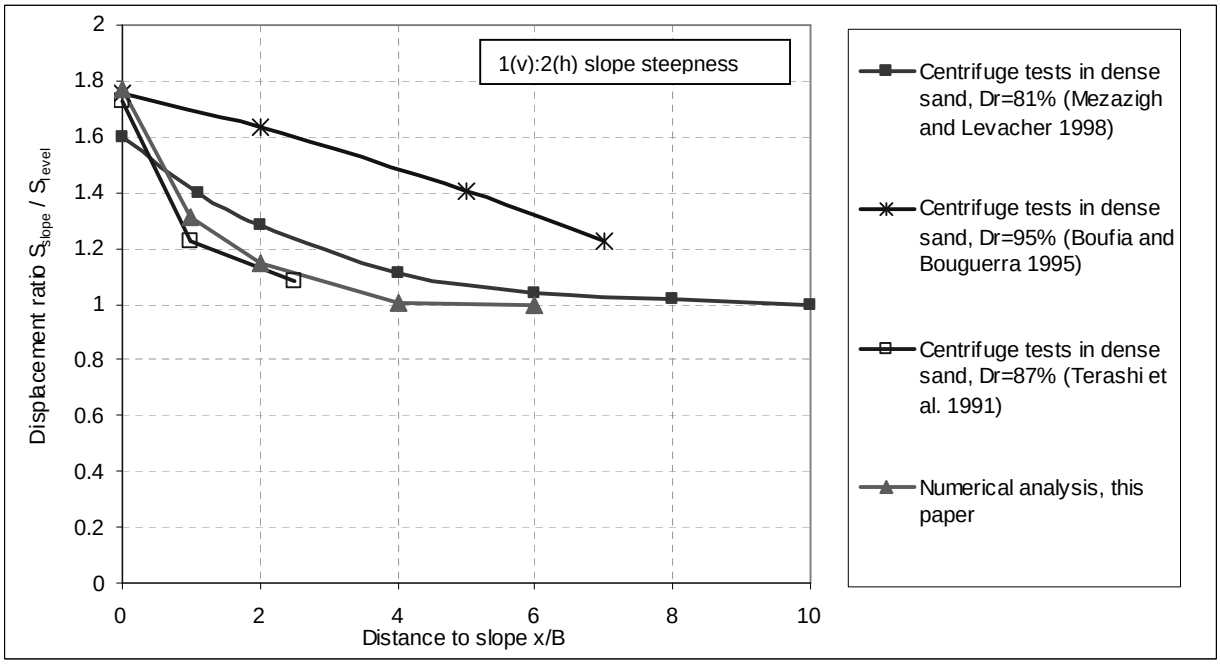


Figure 3: Effect of distance to the slope of 1:2 on displacement at pile head

## 4. NUMERICAL STUDY

A numerical study was undertaken to supplement the literature review in evaluating the effect of the slope on the pile performance. A 2-D plane strain finite difference analysis was performed using the program FLAC, Version 5 (Itasca, Minnesota, USA) to confirm the displacement ratios presented in the literature. Analysis was conducted in which the distance of the pile to the slope was varied and compared to the horizontal ground case.

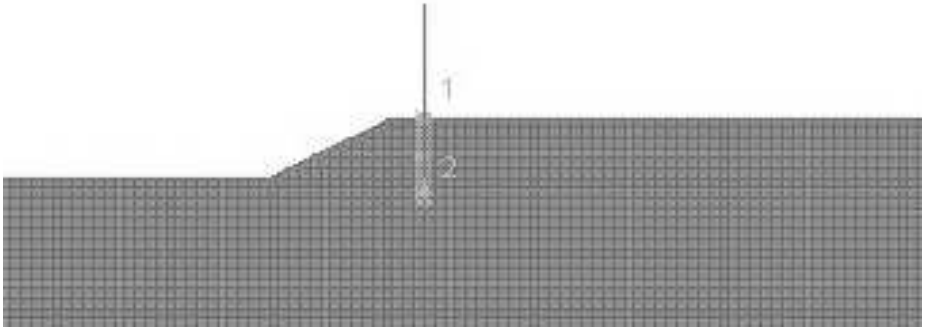


Figure 4: Finite difference grid for a pile in 1:2 sloping ground

For each case in the analysis, a slope of steepness 1(v):2(h) was modeled. The concrete piles were assumed to be 1m in diameter with a 3m embedment depth (L). The material properties of the sand consisted of a friction angle of 33°, Young's modulus of 30MPa, Poisson's ratio of 0.3 and a density of 1800kg/m$^3$. Figure 4 shows the finite difference grid for a sloping ground case.

The results of the analysis for the impact of a slope on horizontal displacement are included in Figures 2 and 3. The displacement ratio for a pile on the crest of a 1(v):2(h) slope was found to be 1.77which compared well to other results presented in the literature. The presence of a slope was found to have a negligible impact on the performance of the pile when it was located greater than 4B from the slope and no influence at a distance of 6B. This finding was similar to that of Mezazigh and Levacher (1998) and Terashi et al. (1991).

# 5. ANALYSIS AND RESULTS

The following section describes the development of design charts for laterally loaded piles in horizontal ground. These charts were suitably modified to allow for the sloping ground condition based on the findings of the literature review and numerical analysis.

## 5.1 FOUNDATION ON HORIZONTAL GROUND

Analysis using the serviceability limit state loads was used to create design charts for determining embedment depth. Foundation deflections (rotation and translation) at ground level were calculated for various foundation depths. The foundation deflections at ground level were then converted to the corresponding deflection at the contact wire height at 5.66m above ground surface.

This analysis was carried out using the commercially available computer program ALP (Oasys, London, UK) for the serviceability limit state condition. ALP is a program that predicts the soil pressures, soil horizontal movements, and shear forces and bending moments induced in a pile foundation when subjected to lateral loads. The pile foundation is modelled as a series of elastic beam elements. The soil is modelled as a series of non-interactive non-linear springs. The soil load-deflection behaviour is modelled using P-Y curves which are generated automatically by the program based on specified soil parameters for each soil layer. The initial slope of the P-Y curves, k, was determined from suggested values reported in Resse et al (1974). These values have been supported by the back analysis of full-scale lateral load test data on bored piles installed in Perth sand (Lehane, 2005).

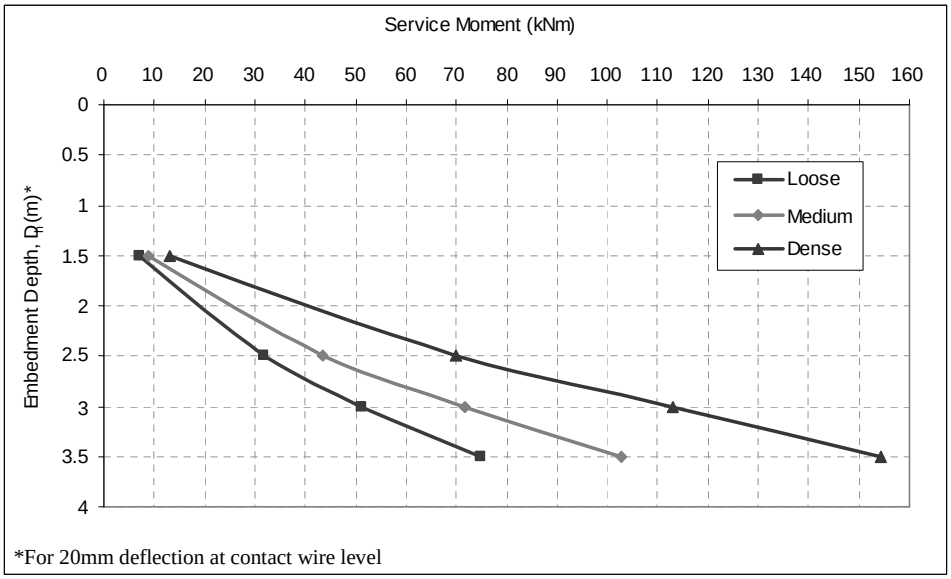


Figure 5: Concrete mast foundation design curves

The foundation embedment depth ($D_h$) versus serviceability limit state moment (M*sls) design chart for the concrete masts is given in Figure 5. These values assume that the foundation is located on a horizontal ground surface and that the groundwater is below the depth of the foundation. The required embedment depth for a particular pile in the field can be ascertained in reference to the relevant serviceability moment and insitu sand density.

### 5.2 FOUNDATION NEXT TO A SLOPE

From the results of the numerical analysis and literature survey, it has been determined that the deflection ratio for piles at the slope crest would be typically 1.8. This meant that for piles with a horizontal ground surface, the pile depth should be chosen such that pile head rotation and deflection caused a lateral overhead deflection of no more than 12mm. If such a pile was then constructed at the slope crest line, the allowable overhead line deflection of 20mm would not be exceeded. The analysis carried out to create the data presented in Figure 5 was then re-run limiting the line deflection to 12mm. For the design serviceability loads, the increased pile penetration was calculated and expressed as a ratio. The average depth ratio was calculated to be of the order of 1.4. On this basis, and assuming that the deflection ratio is unity at a distance from the slope of 4B, the following equation was derived to calculate the required depth of penetration ($D_s$) of the pile:

$$D_s = F_s \times D_h \quad \text{where} \quad F_s = 1 + 0.4\left(1 - \frac{x}{4B}\right) \text{ if } x \le 4B, \; F_s = 1 \text{ if } x > 4B \qquad (2)$$

## 6. CONCLUSIONS

This paper has described work undertaken by Arup Geotechnics (Perth) on the design of short piles to support masts carrying overhead lines on the Perth to Mandurah railway. A number of these laterally loaded pile foundations were to be located next to a slope and a literature review and numerical analysis was carried out to investigate the effect of this on the horizontal displacement of the pile.

It was found that the horizontal displacement of a laterally loaded pile increases with the steepness of the slope and particularly so when the inclination is greater than 1(v):2(h). For a pile located at the crest of a 1(v):2(h) slope, the displacement at the head of a pile was found to be approximately 1.8 times greater than a pile subjected to the same load on horizontal ground. The influence of a slope was found to be minimal for a pile located beyond approximately 4B from the crest and to have no influence at a distance of approximately 8B. These findings were used to develop a modification to be applied to design curves for the horizontal ground condition to account for the presence of a slope.

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# FLOOD DAMAGE ENGINEERING ON A ROADING NETWORK

**Steve Temple** BSc(Geol), MSc (hons) (Eng Geol)
*Opus International Consultants Limited*
*Hamilton, New Zealand*

## ABSTRACT

In May 2005, the Bay of Plenty was hit with a 100 year storm event that resulted in substantial damage occurring to the State Highway and local authority roading network. As a result of the storm, 40 slips and erosion sites were recorded of which 11 have either been repaired or are in the process of getting repaired and the other 29 sites have been repaired by no specific engineering design e.g. removal of the debris, upgrading of the drainage system, installation of kerb and channel, etc.

The upper geology of the Bay of Plenty consists of sequences of airfall deposits sourced from a series of volcanic activities mainly from the nearby Taupo Volcanic Zone. The airfall deposits mantle the underlying base geology and vary in thickness considerably across the region. The substantial amount of rainfall (347mm in 24 hours) had resulted in the increase of the pore water pressures within the soil, which induced widespread slope failures in the area.

Repair of the slips required solutions that allowed the carriageway to be reinstated or widened while providing suitable drainage to minimise potential for future high pore water pressures. This has allowed a variety of retaining walls and slip repairs options to be used consisting of traditional methods (e.g. gabion baskets, timber pole and panel walls) and modern proprietary systems (e.g. Green Terramesh).

## 1. INTRODUCTION

On the 18 May 2005, the Bay of Plenty (see Figure 1) was affected by extreme rainfall that was equated to the 100 year rainfall event. Within a 24 hour period 347mm of rain was recorded with a maximum of 94mm with a 1 hour period (Hegan et al 2005). Earlier in May 2005 there was also a period of heavy rain, which may have also been a significant contributing factor. The rainfall resulted in 40 slips and erosions sites affecting the State Highway or local authority roads (Ferrier et al 2005), plus many others affecting other private and public assets in the Bay. This type of event occurs frequently over small watersheds in the region and landslides in rural areas are common events.

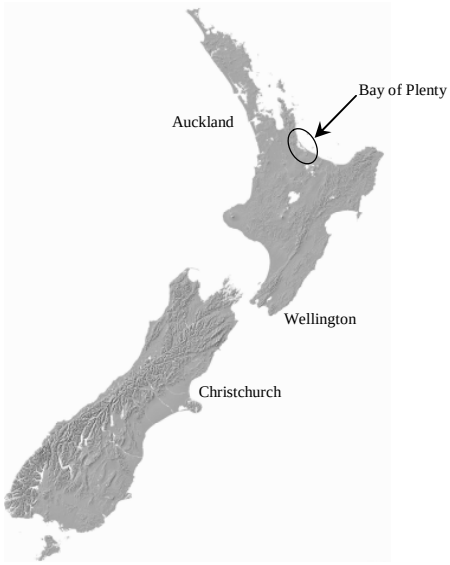



Figure 1: Location map.

Typically the slips have occurred due to the increase in water pressures in the void space between the soil particles, which has resulted from the increased infiltration of the rainfall into the soil. As the slip moved, the material within the failure generally disintegrated and mixed with water to form minor mud flows.

Reinstatement of the State Highway and local authority roads required solutions that allowed the carriageway to be reinstated to original condition or in many cases widened to meet current roading geometric design criteria. One of the key goals of the repairs was to provide suitable drainage to minimize potential for future high pore water pressures from similar weather events. The criteria has allowed for a variety of traditional and modern proprietary systems to be used to reinstate the carriageway. Of the 40 slips and erosions that have affected either the State Highway or local authority roads, 11 of them have either been repaired or are in the process of being repaired. The 11 slips and erosions that have met the criteria for funding have allowed a variety of traditional and modern proprietary systems to be considered for the reinstatement of the slips and erosions. The other 29 sites have been repaired by the road maintenance contractor with no specific engineering design e.g. removal of the debris, upgrading of the drainage system, installation of kerb and channel, etc.

This paper outlines the geotechnical issues and remediation options used to repair slips that occurred as a result of the 100 year storm within the Bay of Plenty.

## 2. GEOLOGY

The geology of the Bay of Plenty area mainly consists of both primary and secondary strata that are of volcanic origin (Briggs et el 1996). The local basement geology consists of Waiteariki Ignimbrite which overlies older andesite, rhyolite and basalt volcanics. The Waiteariki Ignimbrite has been tilted during uplift to the west. The basement geology mainly dips at approximately 5° to the northeast in the Tauranga Basin, where it is unconformably overlain by a sub-horizontal largely undeformed younger volcanic and sedimentary sequence.

The primary volcanics consist of basaltic to rhyolitic lavas, dacitic to rhyolitic ignimbrite and tephras, which form the Tauranga region or south Coromandel Volcanic Zone and the Taupo Volcanic Zone. Theses formations are intercalated with terrestrial and estuarine sedimentary deposits that have been placed due to fluvial, lacustrine and estuarine processes of erosion, transportation and redeposition of consolidated and unconsolidated volcanic debris.

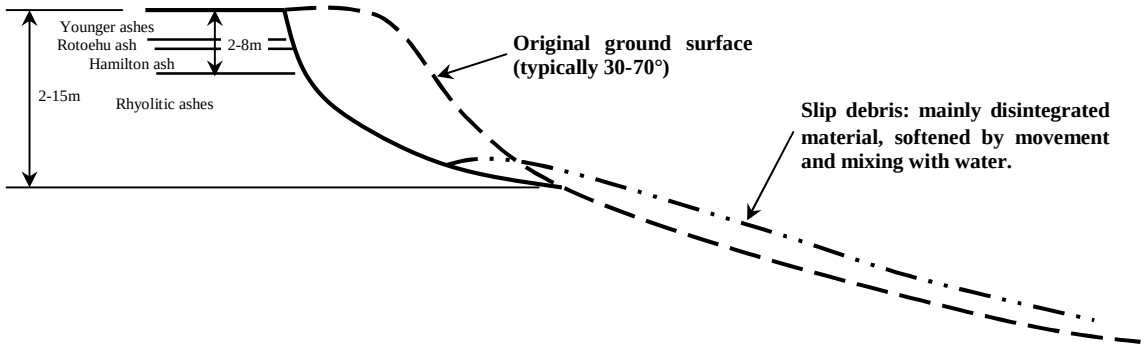
Much of the Tauranga region is covered with a think blanket of Late Pliocene and Holocene tephras which includes the Hamilton Ash, Rotoehu Ash and a sequence of rhyolitic younger tephras derived from the Taupo Volcanic Zone. Also during the late Pleistocene coastal and alluvial sedimentation has occurred that resulted in the formation of low terraces, coastal sand dune complexes, barrier islands and bars. These form the geology of the uppermost 30m of the region.

## 3. MECHANISMS OF FAILURE

Typically, the slopes in which failures have occurred consist of a sequence of volcanic ashes, and alluvial sands. The properties and mineralogy of the older ashes have been altered due to weathering. Within the upper layers, particularly the younger ashes, there are often random vertical fissures or cracks which allow water to easily enter the slope and to seep through. The effect of the rainfall was to increase the infiltration within the soils which results in a rise in the porewater pressures. Zones of confined or perched groundwater were created in some areas within the complicated stratigraphy. The infiltration of water within the soils can also act as an additional external force that would result in the tendency of the soil to slide downhill under gravity.

The failures generally extended from the top of a slope to about halfway down the slope (see Figures 2 and 3). The slip material generally spread out over the lower slopes and in many cases traveled a considerable distance beyond the original toe of the slope. As the slip moved it tended to lose all shear strength and act as a mud flow. In general the slips have occurred on slopes that are the remains of cliffs (typically 30-70°). A study undertaken for the Tauranga City Council (Bell et al 2003) in 2000 identified more than 2000 slide features that were classified as either actual or probable landslides.

Figure 2: Showing the most common form of slide.



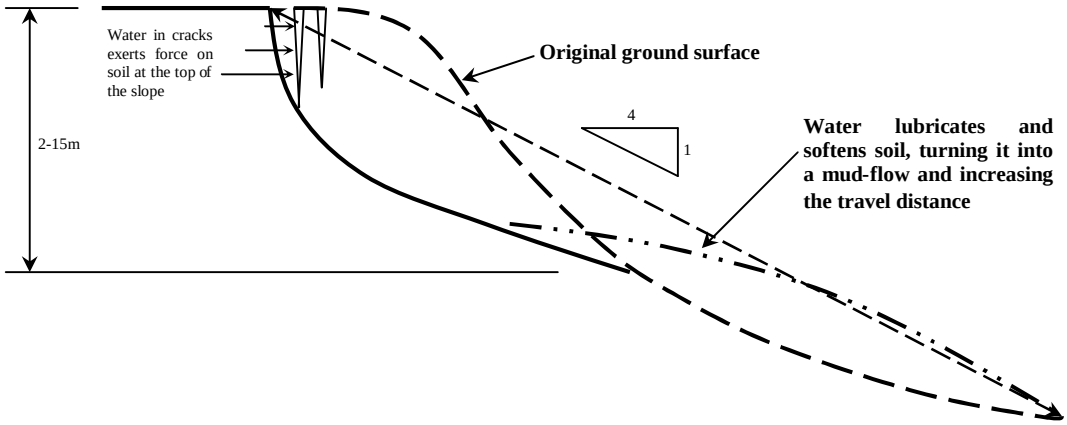



Figure 3: Shows the mechanism by which water influences the slope and the behaviour of the slip. The general run-out distance was approximately 4H:1V from the headscarp.

## 4. REPAIR OF SLIPS

A variety of different types of overslips and underslips have occurred within the State Highway and local authority roading networks, which have resulted in either damage occurring to the carriageway or debris being deposited onto the carriageway. Whenever possible, the repair work had to be carried out within the road reserve boundaries to avoid the need to get access to private land.

The design philosophy for slip repairs in the rural roading network is based on the following general principles:

1. Repair works should re-establish the pre-existing levels of service, in terms of drainage, road geometry, pavement condition, footpaths, etc. In some cases our contract required us to increase the width of the original carriageway.
2. Works may take place within areas subject to natural hazards, but shall not worsen, accelerate or create a new natural hazard.
3. Risk of additional damage should be no higher than the original situation and should be no greater than is present in the surrounding area.
4. Works are not designed to any particular earthquake loading or other unusual condition, except as required to satisfy point 3 above.

The restrictions and criteria for the remediation of the slips has meant that a variety of traditional methods and modern proprietary systems have been considered for the individual sites. During the design stage of the projects, the traditional and modern proprietary systems have been considered for all reinstatements and the advantages of both were considered in terms of the best solution to meet the criteria of the roading authority as well as the way that the solution will blend in with the environment. The remediation of the slips has allowed the following traditional and modern proprietary systems to be used:

- Earthfill slopes
- Hybrid synthetic reinforced slopes (e.g. Green Terramesh)
- Hybrid wall structures (e.g. gabion gravity walls)
- Structural walls (e.g. timber or concrete)

## 5. TRADITIONAL SYSTEMS

Traditional systems in the form of gabion baskets, earthfill embankments and timber pole and panel walls have been either proposed or used to reinstate slips to the criteria of the roading authority. All traditional systems were designed to have a minimum Factor of Safety of 1.3 and to provide a suitable level of drainage to minimize the future potential for high pore pressures.

A gabion basket retaining wall has been used to reinstate a section of a local authority road damage by an underslip that occurred due to excessive water overflow and overslip material from the opposite side of the carriageway (see Figure 4).

Figure 4: A photograph of the underslip following the storm event.

Figure 5: A photograph of the completed gabion basket retaining wall.

The design of the reinstatement for the carriageway embankment consisted of a 4m high gabion basket wall with free draining backfill. The wall is 16m long and was keyed into the existing ground (see Figure 5). The use of the gabion baskets achieved the design Factor of Safety of 1.3. The use of the gabion baskets and the free draining backfill will minimise the potential for future pore pressure build up during substantial rainfalls.

Another traditional system has been proposed for an underslip that has occurred next to a local authority road due to erosion after the rainfall exceeded the capacity of the local stormwater system (see Figure 6). The inundation of the stormwater system has resulted in the excessive flow exiting through the centre of a pre-existing failure, which resulted in the road shoulder and a portion of the carriageway being damaged.

Figure 6: A photograph of the failure that resulted from the stormwater system being overwhelmed.

The geotechnical design for the remediation of the failure is an engineered granular fill consisting of benched free draining compacted granular fill layers with rock rip-rap face around the existing culvert structure (see Figure 7). The modeled design will allow a minimum Factor of Safety of 1.3 to be achieved and the use of free draining granular fill will allow any future run-off from the carriageway to move through the repair without causing any additional damage.

## 6. MODERN PROPRIETARY SYSTEMS

An alternative to the traditional proprietary systems is the use of modern proprietary systems in the form of retaining wall systems like Maccaferri's Green Terramesh, recycled type retaining wall, etc. As an alternative to the use of either gabion baskets or mechanically stabilised earth structures, Green Terramesh systems have been used to reinstate a section of the local authority roading network. The Green Terramesh system was used to reinstate the carriageway where an underslip had occurred due to significant stormwater from an uphill water table (see Figure 8).

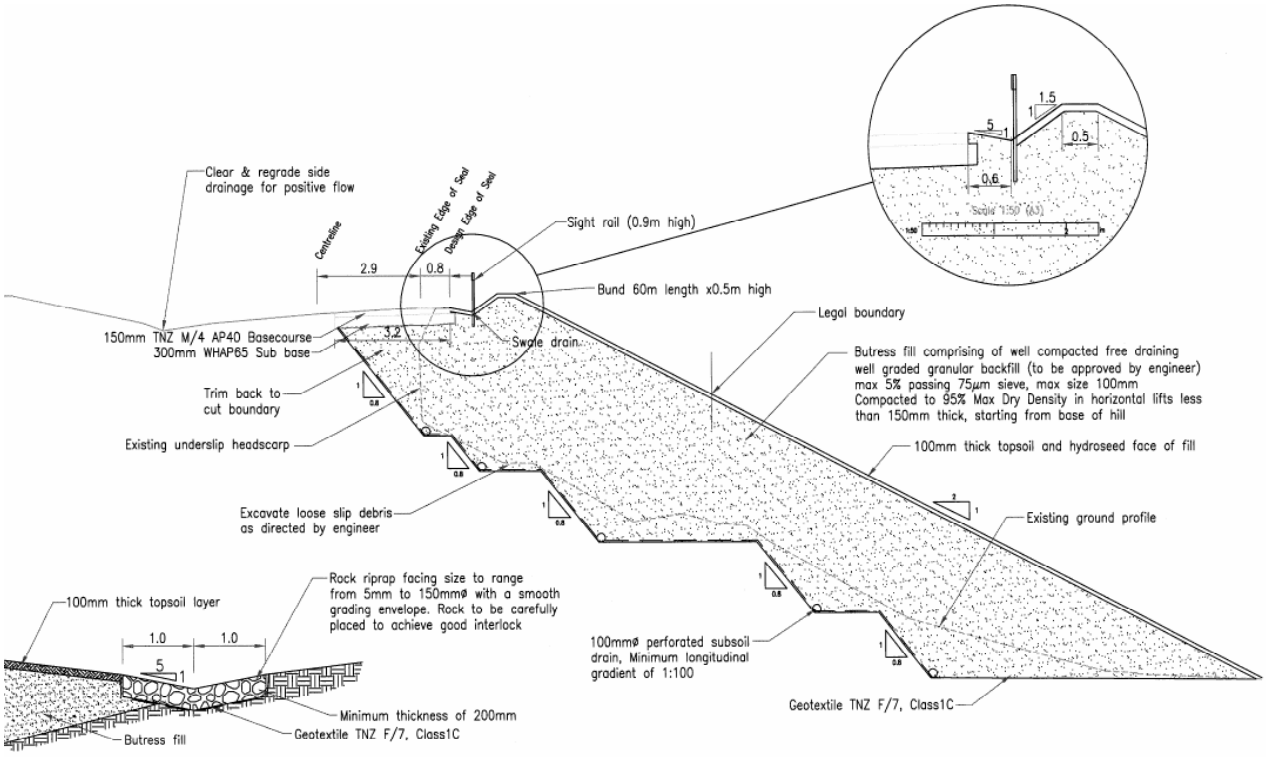



Figure 7: The design of the engineered granular fill.

The design of reinstatement of the carriageway embankment consists of a mechanically stabilised earth (MSE) structure with Green Terramesh facing unit and geogrid reinforcement within the lower section. The design consisted of a 21m long Green Terramesh structure constructed of 8 units with a face batter angle of 70° (see Figures 9 and 10). The base of the MSE structure was 3.5m with Tensar geogrid reinforcement within the lower 5 units. The upper 5 layers of the MSE fill structure utilised lightweight pumice fill material.

Figure 8: Photograph of the underslip that occurred due to significant stormwater from an uphill water table.

Figure 9: Photograph of the construction of the foundation layer.

Using Green Terramesh units instead of standard wrap-around geogrid faces has allowed the retaining solution to be constructed in less time that would normally be required (see Figure 11). The Green Terramesh units have allowed a 0.35H:1V (70°) face to be formed for the full height of the repair, avoiding the requirement to extend the works onto a relatively steep slope that extends for a substantial distance into a valley feature below.

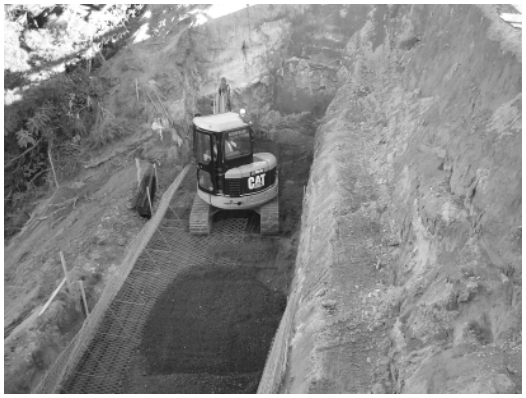



Figure 10: Photograph of the installation of the Green Terramesh units.

Figure 11: Photograph of the completed wall.

## 7. CONCLUSIONS

The 100 year storm event that affected the Bay of Plenty on the 18 May 2005 resulted in 40 slips and erosions within the State Highway and local authority road networks. Of the 40 slips and erosions, 11 have been repaired or are in the process of getting repaired, while the other 29 sites have been repaired with no specific engineering design.

The uppermost 30m of the geology of the Tauranga region is covered with a think blanket of Late Pliocene and Holocene tephras. The properties and mineralogy of the ashes have been altered due to weathering.

The mechanism of failure is due to water entering the slope via random vertical fissures or cracks, the increased infiltration within the soils results in a rise in the pore water pressures. The infiltration of water within the soils can also act as an additional external force that would result in the tendency of the soil to slide downhill under gravity. As the slip moved, the material within the failure generally disintegrated and mixed with water to form minor mud flows.

The restrictions and criteria for the remediation of the slips has meant that a variety of traditional methods and modern proprietary systems have been considered for individual sites. The remediation of the slips has allowed the following traditional and modern proprietary systems to be used:

- Earthfill slopes
- Hybrid synthetic reinforced slopes (e.g. Green Terramesh)
- Hybrid wall structures (e.g. gabion gravity walls)
- Structural walls (e.g. timber or concrete)

## 8. REFERENCES

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# COLLECTING GEOTECHNICALLY SIGNIFICANT DATA FROM DRILL CORE

**Esther Theron**
*SRK Consulting Engineers and Scientists*
*Perth, Western Australia*

## ABSTRACT

This paper is intended as a practical guide to the collection of meaningful geotechnical data from drill core, based on the author's experience. There are numerous empirical rock mass classification systems available to aid the geotechnical evaluation process. All of these systems rely on meaningful input data, which in turn relies on the sound judgment and experience of the core logger.

The paper focuses on the following subjects, with the aim of instructing the logger in the practical basics of collecting geotechnical data, regardless of which rock mass classification system is used for analysing the data:

- How to differentiate natural structures from artificial breaks induced by the drilling and handling process.
- How to identify major (primary) structures.
- How to distinguish geotechnically significant secondary structures (including rock fabric).
- How to speed up the logging process by grouping structures and collecting data per geotechnical domain.
- How to overcome drilling direction bias, even in non-oriented core.
- How to define geotechnical intervals in drillcore.

## 1. INTRODUCTION

Empirical rock mass classification systems are, for the most part, based on the number of naturally occurring structures in the material, and the surface condition of these structures. This component typically accounts for 75% or more of the total in-situ stability rating for any material. Even the most careful drilling and handling process will result in breaking the core. It is thus vital that artificial breaks are accurately identified and discounted during the logging process.

The data collected from geotechnical core logging is ultimately used as the basis for a statistical analysis of the materials. To this end, statistically representative and meaningful logging intervals are required. This is discussed under the heading of "geotechnical domaining".

## 2. IDENTIFYING STRUCTURES

The first, and most important task in the process of geotechnical logging, is to distinguish natural structures from core breaks introduced to the material by the process of drilling and handling of the core. All persons responsible for the geotechnical logging of core should spend some time observing the extraction of drillcore from the core barrel, the process of placing core into core trays, and the nature of any drilling mud used in the drilling process.

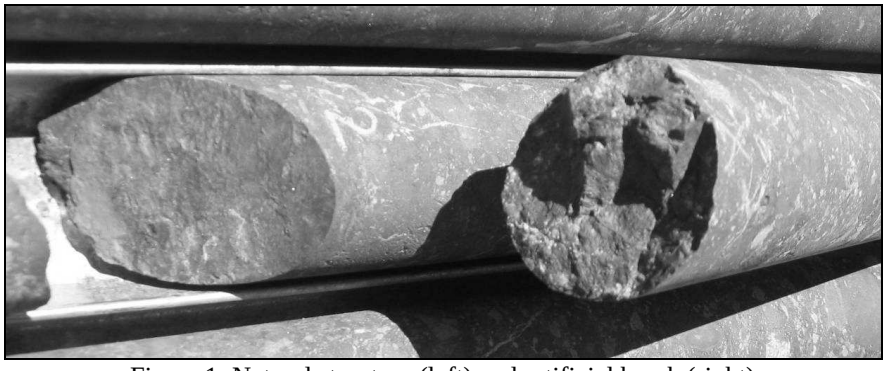

Figure 1: Natural structure (left) and artificial break (right)

## 2.1. ARTIFICIAL BREAKS

Artificial breaks are distinguished from natural structures through a subjective inspection of every "break" in the core. Artificial breaks are generally excluded from the logging process, with the exception of: core disking, which is noted in the comments field; and core loss, which may need to be logged as a separate interval.

Some typical causes of artificial breaks include:

- *Processing breaks* – resulting from core being broken to fit into core trays, and from placing core in the trays ("snapping" or dropping the core into the tray). These breaks should all be consistently marked by the handler as they happen. Such breaks are generally perpendicular to the core, unless the material breaks preferentially along an internal fabric direction (e.g. bedding or foliation). Often accompanied by "chips" off the core circumference. Note: Do not automatically assume that "end-of-tray" breaks are artificial, or that all core perpendicular breaks are artificial.
- *End of Run breaks* – the end of every drilling run should always be inspected. Both the end of run break, and breaks within 10cm of the end of the drill run are likely artificial.
- *Hammering breaks* – resulting from the extrusion of the core from the drill core barrel. These are sometimes expressed as cross cutting breaks (X or V-shaped) forming around the impact point, and usually occur while the core is hammered while inside the core barrel. Some crushing of the material can accompany these breaks. Note: Natural structures may also intersect in a similar manner.
- *Core Disking* – normally forms due to a high in-situ stress field perpendicular to the drilling direction. These features are not structures, and should be discounted. However, it is very important that this occurrence is noted in the logging sheet. The disking is caused by stress release in the core occurring after it is freed from the restraint of the surrounding rock by the coring action. There is generally also a consistent decline in the average disk thickness with increase in maximum horizontal stress.
- *Compression breaks* – resulting from compression of the core in the barrel. These are expressed in the core as a small crushed zone, with fragments often having a "flaky" appearance. The crushed zone is frequently bound by sharp "cones", pointing into the crushed zone. Occurs mainly with small diameter core.
- *Core Spin* – When two adjacent pieces of core are free to rotate (or spin) independently inside the core barrel, the result is a grinding action that often polishes the two ends involved. This is usually accompanied by some degree of core loss. It is not always possible to tell whether these features formed on natural structures, or on induced breaks. When in doubt, mark as a structure.
- *Core Loss / Gain* – There are various potential reasons for core loss, including: poor drilling technique or material condition. Slough at the top of a run that has been washed down the drill hole should be ignored.

Distinguishing between artificial breaks and natural structures is not always easy and there are no 'absolute rules' to aid the logger, but the following methodology and observations have proved useful:

- Start by *inspecting* known artificial breaks, as marked by the drillers (or by creating a fresh break when in doubt). Also inspect breaks that are likely to be natural features, such as surfaces with infill or staining and breaks at a low angle (sub-parallel) to the core. This will help to form an impression of the character of natural and artificial breaks. Inspect each material type separately, as different materials will produce natural structures and artificial breaks with different characteristics.
- M*ark* artificial breaks and natural structures on the core. Be methodical, and inspect and mark each feature.
- Physical *criteria* used for recognition of an artificial break can include:
  - No mineral coatings on break surface.
  - Freshly broken appearance of surface.
  - Surface is irregularly shaped and rough, but ends fit together well. Sharp edges are common. Bear in mind that natural structures can also have rough surfaces, and can sometimes be quite tight, as is the case with foliation or bedding structures.
  - Break surface is most likely oriented at a high angle (perpendicular) to core axis. This observation does not always hold for materials with preferred mineral alignment, or core broken by 'twisting'.
  - Break profile is often concoidal or cuspate shape.
  - Break is not continuous in quality across entire core diameter.
  - Evidence of hammer-induced breakage, such as small chip(s) off edge of break.

## 2.2. NATURAL STRUCTURES

The term "natural structures" is used here to include all naturally occurring discontinuities present in the rock mass. This term includes all discontinuities related to structural systems such as: shears and faults; all joints (open and cemented); rock fabric such as bedding, foliation or banding; and "sharp" lithological contacts.

The natural structures that are recorded for the purposes of geotechnical logging can be subdivided into three groups, which are treated differently in the logging process:

- Primary rock mass defects (major structures), e.g. faults, shear zones, intense fracturing, zones of deformable material, significant veins, etc.
- Secondary rock mass defects (rock mass fabric), e.g. open or weakly cemented: joints, bedding, foliation, schistosity, etc.
- Tertiary rock mass defects (rock fabric), e.g. moderately cemented joints and moderately cemented rock fabric.

Strongly annealed structures (e.g. thin quartz veins, silicified fault breccia) are generally treated as "intact rock", and are *not considered* while marking natural structures on the core.

### 2.2.1. Primary Structures

Primary structures are generally easy to identify when they are associated with rock mass deformation. The proper identification of these structures is essential, as they control the overall stability of excavations and are frequently conduits for groundwater movement. The data gathered will be used to characterise these features, and will ultimately aid in the 3D interpretation and domaining of the rock mass. Some of the features indicative of primary structures are listed below:

- *Fault gouge.* Includes the whole spectrum of infill from fine grained, deformable material through coarse clasts of re-cemented, brecciated material. Some care is required to distinguish true gouge from drill induced 'gouge'.
- *Localised alteration.* Fluids causing alteration of the rock mass were often channelled through the existing pathways provided by the structural network in the material.
- *Fault zones.* This term refers to corridors of multiple major structures. These zones often result in broken material in the core box, but with a little effort the individual structures can often be identified.
- *Fault splays.* These fault "branches" are structures of often similar character, but perhaps lesser intensity, to the main feature.
- *Damaged zones or fault margins.* Primary structures frequently cause damage to the surrounding rock mass outside of the main fault. These "damaged zones" can take the form of elevated joint frequencies in close proximity to the main structure.
- *Brecciation.* Faulting is often expressed as a brecciated zone. The strength and nature of the cementing material generally determines whether the cored sample remains intact or resembles a gravelly material once extracted. It is often difficult to distinguish poorly cemented brecciation from drilling induced damage in brittle materials, and special care must be taken when assessing these intervals. From a geotechnical perspective, completely healed brecciation is regarded as part of the rock mass.
- *Contacts.* Intrusive contacts and the baked margins in the surrounding materials are generally weaker zones, and should be considered separately. Sharp lithological contacts are frequently associated with major structures, whether as a feature of emplacement, or as a result of the contrast in material properties.
- *Offsets.* Major structures can often cause complete disruption of regular material fabric.
- *Significant veins.* Where veining is commonplace, structures of 15cm or greater true thickness should be considered significant. Veins generally form in pre-existing primary features, which may subsequently be re-activated. Any vein that carries an open structure should be logged as a fault. Vein infill minerals may also be significantly weaker than the surrounding rock mass.
- *Deformable material.* Zones of soft, deformable material are especially significant when material properties contrast sharply with surrounding material.
- *Slickenlines or striations.* These features are usually associated with movement on primary structures. Note that even a small amount of movement on joints, or a preferred mineral alignment can also cause an apparently similar phenomenon.

Where the continuity of the structure alone serves to distinguish primary structures from joints, it is generally impossible to differentiate these structures in drillcore.

### 2.2.2. Secondary Structures

Secondary structures refer to *open or weakly cemented* structures. These are most commonly joints, but can also be rock fabric such as bedding, foliation, schistosity, etc. Weakly cemented structures are natural discontinuities in the core that readily open up when handled.

Physical criteria used for recognition of joints can include:

- Mineral coating or infill on structure surface.
- Staining or oxidation on structure surface in weathered zone.
- Surface is of continuous quality across entire core diameter.
- Features that are sub-parallel to the core are more likely natural structures.
- Surface generally looks different from 'freshly broken' core surface.
- Surface edges and profile are commonly smoother and more rounded than for artificial breaks.

Notes:

- Where the evidence is ambiguous and differentiation is difficult, the conservative approach is recommended – treat at least half of these undefined breaks as natural.
- Where natural fractures exist at very small (acute) angles relative to the core axis, the process of drilling and handling may result in localized fragmentation of the rock. Fragments should be inspected carefully to form an idea of the actual structural orientation and true number of natural structures present.
- Some materials are prone to desiccation (due to drying out) or slaking (due to loss of confinement) or decomposition (due to chemical breakdown of material) when exposed to the elements. This feature is not necessarily related to secondary structures.

Joints can be divided into three sets, as described below, while fabric should be logged as a separate set.

### 2.2.3. Definition of Sets

Primary and secondary structures are traditionally considered critical to the engineering mine design. All of these structures are divided into groups (hereafter referred to as sets,) based on their orientation relative to the core axis (alpha angle), and numbered accordingly. Set 1 alpha angle: 0-30°; Set 2: 30-60° and Set 3: 60-90°.

The reason for this system of grouping is twofold. The first is to allow faster and more sensible data collection than the traditional approach of logging 'block to block' and describing structures individually. This improvement is possible by recording representative data for each set, by geotechnical domain (discussed under separate heading).

The second reason is to correct for the apparent frequency of structures. Statistically, structures which are parallel or near parallel to the drillhole direction are less frequently intersected than perpendicular or near-perpendicular structures of the same frequency. This concept is illustrated in *Figure 2*. While it is almost certain that structures belonging to different structural sets will thus be grouped together, it is generally not practical and unnecessary to attempt to define true sets in drillcore at this stage. (This kind of information can be gathered from detailed structural logging.)

### 2.2.4. Tertiary Structures

Moderately annealed structures are also of importance, notably in terms of blasting and secondary fragmentation. Where these structures are continuous across the core, they can be recorded as a single group or set, regardless of orientation.

Infill minerals that qualify as 'moderately annealed' typically have a Mohs hardness of 5 or less. This criterion for identification should also consider the relative difference between the cementing mineral hardness and the hardness of the host rock.

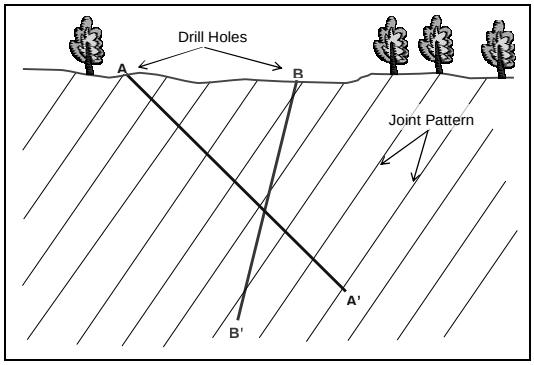


Figure 2: Apparent fracture frequency depending on drilling direction
*Note: Hole AA' is near perpendicular to the joint pattern and intersects the structure set 8 times. Hole BB' is near parallel to the same joint pattern and intersects the structures set 3 times.*

### 2.3. SUMMARY PROCEDURE

- Methodically identify all natural structures in the core, and mark them clearly.
- Primary structure planes and open or weakly cemented joints are divided into three sets, depending on their orientation to the core axis, and are labelled "1, 2 or 3".
- Open or weakly cemented material fabric is labelled as a separate set of structures (e.g. "F").
- Moderately cemented structures are labelled as a separate set (e.g. "C").

## 3. GEOTECHNICAL DOMAINING

Geotechnical domaining of the rock mass at the core box is the first step in the geotechnical design process. Once the geological logging and the identification of natural structures in the core is complete, geotechnical intervals or 'domains' can be defined. These intervals are used to build up a statistical picture of the range of rock mass conditions encountered, and it is thus essential that these intervals are correctly identified.

A geotechnical interval refers to a length of core (or rock mass) which is expected to behave in a similar manner once exposed to the mining environment. Different materials have different mineralogical make-ups and thus have different hardness, respond differently to applied stresses, have different joint frequencies and surface conditions, and have different reactions to alteration and weathering. It is therefore important to *limit geotechnical domains to one rock type*. Lithological intervals often require *sub-domains* to demarcate zones of different rock mass conditions in the same rock type.

When considering the sub-dividing of lithological intervals, it is important to reflect on the potential impact of the feature on the scale of mining. It is also necessary to reflect on the relative importance of features based on adjacent materials, or put another way: to consider the effect of the feature on the potential instability of the surrounding rock mass. The following examples illustrate this point:

- A 5cm thick zone of completely weathered material in 5m of fresh rock will be a source of potential instability, and should be sub-domained.
- A 5cm thick zone of fresh rock in 5m of completely weathered material will not be a significant source of stability in the weak zone, and does not need to be sub-domained. (Its presence should be noted in the comments, as it may act as an aquifer).
- 5cm of core loss in 5m of completely weathered material is not significant, and does not need to be sub-domained.

The main reasons for sub-domaining a lithological interval are:

- *Presence of primary structures* (faults, shears, significant veins, closely jointed, brecciation, etc.)

  - Any *single primary structure* encountered must be defined as a separate interval. This might require creating a very small interval (only a few millimetres wide).
  - A zone of *multiple primary structures* should generally be defined as a single domain.
  - Where *veining* is commonplace, intact veins of 15cm or greater true thickness should be considered significant. Where veining is not commonplace, any vein is potentially significant. Veins generally form in pre-existing primary features, which may subsequently be re-activated. Any vein, regardless of thickness, that carries an open structure should be sub-domained and indicated as a possible fault. Vein infill minerals may also be significantly weaker than the surrounding rock mass. In this case the thickness and abundance of veining will determine whether veining should be considered as 'weakly healed' structures and counted with open structures in the domain; or as 'moderately healed' structures and counted with cemented structures; or should be domained as single features.
  - Very *closely jointed zones* in materials that are normally less frequently jointed could possibly represent a major structure, and should be domained separately.
  - The presence of *brecciation*, whether intact or not, should normally be domained as a separate interval. A marked change in the clast size or cementing mineral will normally require a separate sub-domain.
- *Fracture frequency (spacing) patterns.* The basis for geotechnical sub-domaining should rest firmly on the patterns in the frequency of structures. A convenient way of assessing the patterns in the spacing of open and weakly cemented structures is by writing the number of structures identified on the end of the core tray. Review the number of structures per meter (or core tray length) first. As an example, a significant change would be a change from consistently 1 or 2 structures per meter, to say 5 or 6 in a meter. Next compare the total number of structures per box, before deciding whether to sub-divide lithological intervals.
  - As a rule of thumb: lithological intervals of 5m or less should not be subdivided based on patterns in the fracture frequency, unless there is compelling reason to do so.
  - As a rule of thumb: massive (unstructured) intervals of 10m or greater must be sub-domained.
  - As a rule of thumb: a single structure, bounded by 10m or greater of massive intervals on either side, should be domained as a 1m long interval containing that structure.
- Significant changes in *structure surface condition.* Some examples:
  - If the infill mineral on joints changes from (say) predominantly calcite to predominantly talc, a separate sub-domain is required.
  - If the surface roughness on joints changes from rough and undulating to polished and straight, a separate sub-domain is required.
- Change in *weathering condition* of material.
- Change in *geotechnically significant alteration* of material. Geotechnically significant alteration is alteration that has caused or will cause a significant difference in the stability behaviour of the material over time.
- Significant change in *intact rock strength.*
- *Fabric orientation change.* Marked changes in fabric orientation should trigger a new domain, where practical.

Notes:

- Geotechnical boundaries cannot cross 'hard' geological boundaries. The minimum number of geotechnical intervals in a borehole is thus equal to the number of true geological intervals.
- Geotechnical intervals can range from just a few millimetres through many meters in length. The quality of logging is not necessarily improved by over-zealous subdividing of the drillhole, but the statistical validity of data is definitely diminished by not domaining to the correct level of detail.
- Rock mass property data can now be collected for each successive geotechnical domain in the drillhole.
- The methodology suggested is a general one. Specific projects, such as raiseboring investigations, will require more detailed logging procedures.

## 4. CONCLUSIONS

During the geotechnical logging of drillcore, it is essential that artificial breaks are identified and disregarded. Geotechnical domain logging can speed up the logging process considerably, and eliminates the 'smearing' of input data caused by traditional logging from 'block to block'. The suggested process cannot possibly consider the unique conditions prevailing at all sites, but the essence thereof will ensure that meaningful input data for geotechnical stability assessments is collected.

# BEHAVIOUR OF OVERCONSOLIDATED CLAYS UNDER PLANE STRAIN TESTING

**Pontjo Utomo***, **Hamid R. Nikraz****
**PhD Student, Department of Civil Engineering, Curtin University of Technology, Perth, Australia*
***Associate Professor, Department of Civil Engineering, Curtin University of Technology, Perth, Australia*

## ABSTRACT

This paper is aimed to disseminate some recent testing results on overconsolidated clays which were obtained from a new proposed plane strain soil testing apparatus. Most of the laboratory testing of soil to investigate its mechanical behaviour is carried out using triaxial machines. In fact, many geotechnical issues, which may be encountered in the field such as slope failure, shallow foundations, and earth retaining structures, are truly or close to plane strain. The design parameter values which are generated by triaxial apparatus would be conservative for plane strain problems. The plane strain apparatus, which was used in this present study, is a modification of the conventional triaxial apparatus. In this machine, a pair of rigid perspex walls was used to impose zero strain in the $\varepsilon_2$-direction (out of plane), which left only $\sigma_1$ and $\sigma_3$ acting on the specimen. A rectangular specimen with the size of 72 mm x 72 mm x 36 mm was used for the tests. The novel features of this machine include its ease of setting up and the ability to use a conventional O-ring and membrane. The test results showed that the apparatus was able to generate data, which showed good agreement with known soil mechanics behaviour.

Keywords: plane strain, triaxial, overconsolidated clays

## 1. INTRODUCTION

Most of the laboratory testing of soil to obtain its mechanical properties are performed under axisymmetric (conventional triaxial) conditions, whereas in the field most geotechnical problems are truly 3D or else close to plane strain conditions. Therefore in general, data from plane strain testing would be more appropriate than data from triaxial testing. Figure 1 depicts several examples of plane strain conditions which may exist in the field.

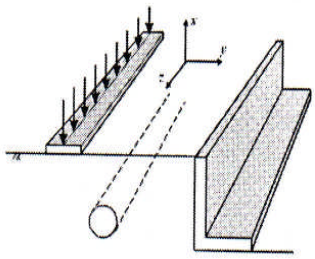

Figure 1: Examples of Plane Strain Condition (Potts et.al. 2002)

Test data on overconsolidated clays tested under plane strain conditions are relatively rare. Hence, a plane strain compression device has been developed to investigate the behaviour of overconsolidated clays under plane strain conditions.

## 2. LITERATURE REVIEW

### 2.1. HISTORY OF PLANE STRAIN TESTING APPARATUS

Wood (1958), developed a plane strain apparatus which used a long rectangular specimen for ease of controlling the plane strain condition, subsequently followed by Cornforth (1964) who also proposed a similar device, which was known as the "Bishop-Cornforth" device. However, Finn et.al (1968), Lee (1970) and Marachi et.al (1981), who

adopted this method, found that the friction force arising from the axial loading surface would cause considerable inaccuracy in the measurements of stresses and strains.

Green (1971) developed a plane strain test device using a rectangular specimen of 84 mm x 76 mm x 53 mm, after which the use of a small rectangular specimen size increased significantly. However, Bishop (1981) pointed out that Green's method, which had the $\sigma_2$ loading surface suspended by wires, could not perform as expected.

Thereafter, most of the plane strain test apparatus (Green & Reades, 1975; Mochizuki et.al., 1988; Peters et.al., 1988; Drescher et.al., 1990; Han & Vardoulakis, 1991; Viggiani et.al., 1994) had the common feature of using rigid walls and tie-rods to impose a zero strain boundary condition in the direction of one of the principal axes. This method was found to be satisfactory, and friction between the rigid walls and the specimen could be satisfactorily mitigated using silicone lubricant. However, the rubber membrane for all these devices had to be customized. This resulted in slackness and the formation of air pockets. Some researchers have resorted to applying suction, although detrimental; to remove air pockets, while it not known what other researchers do about the problem. The application of suction for removing air pockets may cause swelling and desaturation of the sample, which can lead to sample disturbance.

In terms of testing results, Mochizuki et.al (1993), carried out a series of soil tests under plane strain conditions. In general, the results for plane strain indicated that the compressive strengths were higher and also had lower axial strain. The use of mechanical properties, for medium to dense sands that are derived from axisymmetric triaxial tests would lead to more conservative designs in most cases. In plane strain testing, shear bands more readily occur compared to triaxial testing (Peters et.al, 1988). Plane strain testing of clay has been undertaken only recently (Drescher et.al, 1990; Viggiani et.al, 1994), but published data for these tests are not available. The reason for this is most probably due to the lack of reliability of testing.

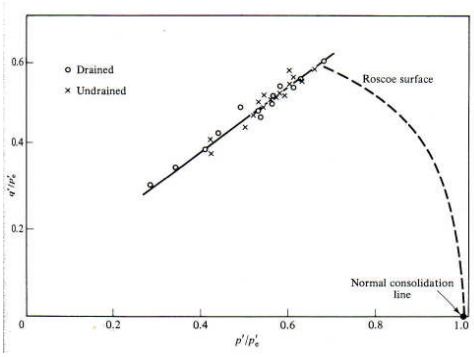


Figure 2: OC-clays at failure state fall on the Hvorslev surface (Atkinson and Bransby, 1982)

### 2.2. BEHAVIOUR OF OVERCONSOLIDATED CLAY

Overconsolidated clay (oc-clay) has different behaviour to normally consolidated clay (nc-clay). Both types of clay have different engineering characteristics due to their stress histories that they underwent in the past. The state of nc-clay lies on the "wet" side of the critical state line while the state of oc-clay is on the "dry" side of critical state line. Figure 2 shows the failure states of overconsolidated clays which fall on the Hvorslev surface. The detail discussion of oc-clay behaviour is discussed in Atkinson and Bransby (1982).

## 3. APPARATUS USED AND EXPERIMENTAL METHOD

### 3.1. PROPOSED PLANE STRAIN APPARATUS

A plane strain apparatus is proposed in this present paper. The specimen size is 72 mm high x 36 mm wide x 72 mm thick, so that the aspect ratio (height to width) is 2. This is accordance with the recommendations given by Taylor (1941), Rowe & Barden (1964), Lee & Seed (1964) and Bishop & Green (1965), they found that if the aspect ratio was greater than 2, the effect of loading platen friction and the restraint of the loading frame would be negligible and the specimen would be in constant strength.

The ordinary cylindrical rubber membrane measured 100 mm in diameter and 200 mm length was used. This would allow some flexibility in the height of specimen to be used. The rubber membrane remains taut after setting up, thus preventing the formation of air pockets in the vicinity of the specimen (Figure 3). The rubber membrane is secured to the rectangular loading platens by attaching O-rings and O-ring clamps. The water tightness of the system has been tested to a very high pressure (1000 kPa) and found to be satisfactory. Plane strain conditions are maintained using acrylic rigid walls and tie rods, while the friction between the rigid walls and the specimen is reduced by applying silicone lubricant. The volume change was measured by a volume-change transducer.

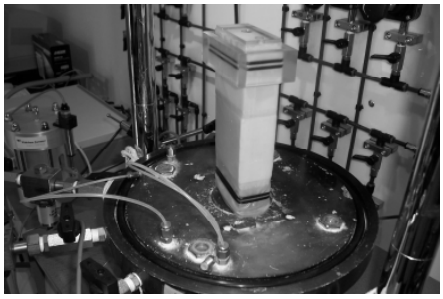

Figure 3: Part of the plane strain test arrangement

The O-ring used has a diameter of 45 mm and a wall thickness of 3 mm. Sintered bronze porous stones measuring 36 mm wide by 72 mm long and 5 mm thick were used, which had a porosity similar to that of the standard porous stone for the triaxial apparatus. The biaxial arrangement is then placed in the cell, with a height of 300 mm, 200 mm internal diameter and 30 mm wall thickness (Figure 4). The cell is filled up with water to enable the specimen to be pressurized laterally using the GDS pressure generator for applying confining pressure. A Wykeham Farrance displacement-controlled compression machine is used to apply the axial load to the test specimens.

For the purpose of measuring the axial load that is applied to the test specimen, a Wykeham Farrance WF 17109 submersible load cell is used, which has a capacity of 5 kN. The axial displacement of the test specimen is measured by attaching an LVDT (Linear Vertical Displacement Transducer) with the capacity of traveling up to 35 mm on top of the top cover of the cell. Three pore pressure transducers are used to measure cell pressure, back pressure and pore pressure in the test specimen. These pore pressure transducers are mounted to the base plate of the cell. A volume change monitoring apparatus is also connected to the back pressure line to monitor the global volume change of the water-saturated soil specimen.

A data acquisition system is prepared to record the measured data, consisting of an MPX 3000 data logger, a product of VJ Technology (UK) and a set of Pentium IV-based personal computer. A software called WINHOST 2.0 (VJ Technology product) is used to manage data acquisition. This WINHOST 2.0 transfers digital bits data from ADU (Analogue Digital Units) form to engineering units based on the calibration of the relevant measuring device, which is undertaken before running the plane strain test. These data acquisition arrangements are used to record the axial displacements, axial loads, pressures and volume changes.

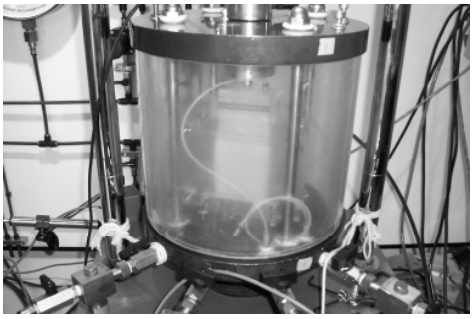

Figure 4: Assembly of plane strain apparatus

### 3.2. SPECIMEN PREPARATION AND TESTING METHOD

The kaolin clay which was used for this study is a product of UNIMIN PTY LTD, Australia, with the commercial name of Kaolin RF. Before the clay powder was mixed with water, a standard test to obtain its engineering properties was carried out.

Firstly, a slurry of kaolin clay was made. This slurry was obtained by mixing 8 kg of kaolin powder with 6 kg of water, so that the water content of the slurry is around 1.5 times its liquid limit. Kaolin powder and water were mixed in an electric mixer for about 2 hours to ensure the uniformity of the slurry. Before pouring the kaolin slurry to the consolidation unit, the inner wall of the cylinder mould was greased to ease the extrusion of the sample from the mould at the end of consolidation stage. The slurry was poured in a steel cylindrical tube with a height of 600 mm and 150 mm in diameter (See Figure 5). On the top and the bottom of this slurry, a layer of sand and geotextile were attached to accelerate the consolidation process. At the both ends of the slurry in the mould, two circular Perspexes discs were placed to apply the pressure evenly to the slurry. The soft slurry was allowed to consolidate under its own weight and some small pressures were applied to prevent the slurry from being squeezed out between the circular Perspex and the cylindrical mould. Once the slurry gained sufficient strength to withstand higher loading, a higher vertical pressure was applied. A dial gauge was used to monitor the compression of the slurry. If there was no further settlement change, the next loading stage was then applied. A maximum of 300 kPa was applied at the end of the consolidation process.

Once the consolidation process was completed, specimen formers were pushed on the top of the consolidated kaolin in the tube to fill them with the specimens. Following this, a layer of wax was placed on both ends of the formers and sealed by a plastic film to keep the water content in the specimens constant. Before undertaking plane strain compression tests on the specimens, the specimens in the formers were placed in a dehumidifier for at least 2 days to obtain an even pore water pressure distribution in the specimens.

Figure 5: Consolidation of kaolin slurry in cylindrical mould

Before the plane strain compression test was carried out, the standard procedure either for consolidated drained (CD) or consolidated undrained (CU) triaxial testing was applied (Head, 1986). The specimen was firstly saturated by applying a back pressure and cell pressure. Once the B-value of the test specimen reached a value of 0.95-0.98, then the consolidation stages were started, a back pressure of 300 kPa was applied, with a maximum of 700 kPa effective consolidation pressure applied for all test specimens, followed by an unloading process to set the condition of the specimen into an overconsolidated state. If the desired overconsolidated ratio (OCR) is 4, the isotropic swelling process was carried out in 3 steps of unloading, which are 700 kPa, 350 kPa and 175 kPa. The test specimen for CD testing is loaded axially by applying the loading through the loading machine, with the drainage line opened. The drained test required 3 days to complete and data were read every 5 minutes. For CD tests, a loading rate of 0.004 mm/min was applied. This loading rate was deduced based on the condition of the adopted kaolin clay (Bishop and Henkel, 1962). The CD test was terminated at an axial strain of about 20% or sooner. Once the test specimen had failed, it was taken out immediately for the purpose of moisture content determination. Table 1 shows the test arrangements discussed in this paper.

Table 1. Testing information for oc-clays

| Series | $v_o$ (initial specific volume) | *B*-value | Test type & OCR |
|---|---|---|---|
| PSCU4 | 1.60 | 0.97 | Undrained, 4 |
| PSCU16 | 1.64 | 0.96 | Undrained, 16 |
| PSCD8 | 1. 61 | 0.98 | Drained, 8 |
| PSCD20 | 1.65 | 0.97 | Drained, 20 |

The initial treatment that was given to test specimens for CD testing, also applied to the test specimens for CU tests). The test specimen was saturated, consolidated and unloaded. The only difference was the treatment on how the test specimen was sheared up to failure. The loading rate that applied for undrained test was 0.008 mm/min. During the application of load, drainage line was closed and pore water pressure was measured and monitored. The data were recorded at intervals of 3 minutes. The time required to complete each undrained test was about 2 days.

## 4. TESTING RESULTS AND DISCUSSION

A series of soil tests for the overconsolidated clay were undertaken, some basic properties are as in Table 1.

Table 2. Properties of Adopted Kaolin RF clay

| Parameters | Values |
|---|---|
| $\lambda$, virgin compression line slope | 0.103 |
| $\kappa$, swelling line slope | 0.04 |
| $N$, specific volume at $p'$=1 kPa (NCL) | 2.10 |
| $\Gamma$, specific volume at $p'$= 1 kPa (CSL) | 2.04 |
| $M$, critical state line slope | 0.93 |
| $m_H$, Hvorslev surface slope | 0.85 |
| $G_s$, specific gravity | 2.6 |
| *LL (%)* | 53.5 |
| *PL (%)* | 30.8 |
| *PI (%)* | 22.7 |

Table 3. Results of Plane Strain Compression Tests on OC-Clays

| Series | $q_f'$ (kPa) | $p_f'$ (kPa) | $p_o'$ (kPa) | $v_o$ | $v_f$ | Confining pressure, $\sigma_3$ (kPa) | Back Pressure (kPa) |
|---|---|---|---|---|---|---|---|
| PSCU4 | 238 | 294 | 175 | 1.60 | 1.60 | 475 | 300 |
| PSCU16 | 105 | 127 | 44 | 1.64 | 1.64 | 344 | 300 |
| PSCD8 | 188 | 182 | 88 | 1. 61 | 1.62 | 388 | 300 |
| PSCD20 | 84 | 96 | 35 | 1.65 | 1.67 | 335 | 300 |

The results of the tests are summarized in Table 3 and Figure 6 and 7. In general, it can be seen from the stress-strain curves on Figure 6 that, the curves initially went up until they reach peak stresses, followed by strain softening behaviour to reach the critical state. After this, strain increased without stress increase. This is the typical behaviour of overconsolidated clays when they are sheared. Among the specimens listed in Table 1, the highest failure stress of 294 kPa was reached by the specimen with the OCR of 4, which was tested under undrained conditions with the highest confining pressure of 475 kPa, while the lowest results were obtained from the specimen with the OCR of 20, which had failure stress and confining pressure of 84 kPa and 335 kPa respectively. Even though the specimen with the OCR=20 has the highest OCR value compared to the other specimens, it did not result the highest failure stress because it was sheared under the lowest confining pressure (CP = 335 kPa), which affects its strength. It was also clear that for drained tests (see Table 3), the volume of the specimens expanded during the application of axial loads, which is the reflection of the dilatant behaviour of overconsolidated soil.

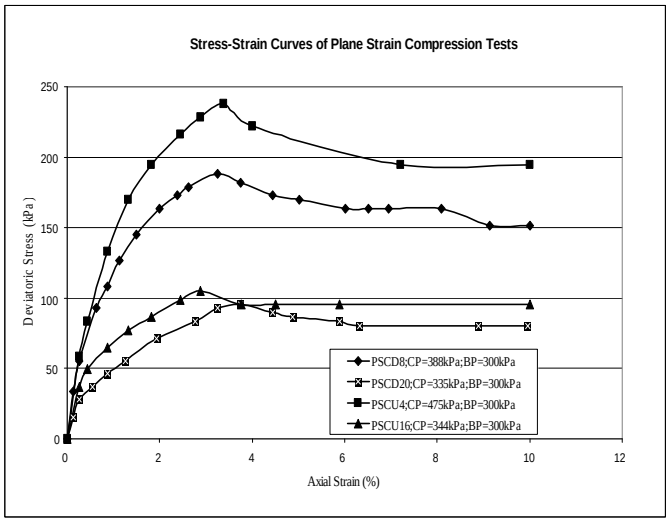


Figure 6: Plane Strain Compression Test Results on OC-clays

Figure 7 shows the stress paths of the plane strain drained compression tests. The failure stress of the specimens fell on the Hvorslev surface while critical states of the specimens fell on the critical state line. This is in a good agreement with known soil mechanics concept which was found by Hvorslev in 1937(Atkinson and Bransby, 1982) and re-investigated by Parry in 1960 (Atkinson and Bransby, 1982). The Hvorslev surface is the locus of failure state of heavily overconsolidated clays while the critical state line is the locus of the critical state of the specimens.

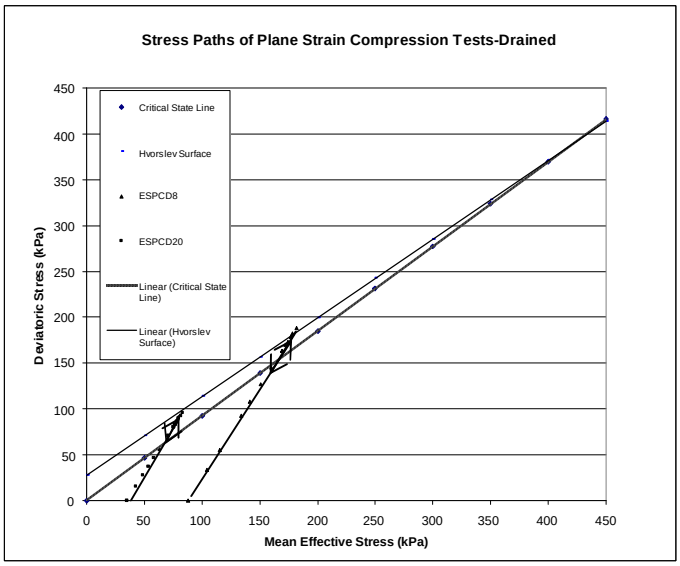


Figure 7. Stress Paths of Plane Strain Tests on Kaolin RF Clays (Drained Tests)

## 5. CONCLUSION

1. The apparatus has been able to produce test results which are in reasonably good agreement with known soil mechanics concepts.
2. Failure of oc-clays falls on the Hvorslev surface.

## 6. ACKNOWLEDGMENTS

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# GEOTECHNICAL BLOCKINESS INDEX

***A Systematic Geotechnical Mapping Method for Underground and Open Pit Mines***

**Diane Walker**
**Ian de Bruyn**
*SRK Consulting*

## ABSTRACT

Window mapping and scan line surveys are standard geotechnical rock face mapping methods at most mines. These methods allow for the collection of detailed geotechnical data at specific locations, and it is not practical to map entire mine workings using these systems. Geotechnical blockiness index (GBI) mapping was developed as a rapid mapping system for recording general geotechnical conditions throughout entire mine workings. The results from GBI mapping show the distribution of varying rockmass properties across a mine. The most important factors in the identification of GBI are the number of effective block forming joint sets and the joint spacing. The GBI system is an eight class system that describes the nature of rockmasses from massive through blocky to friable. It was originally developed within a relatively massive rockmass, but can easily be modified so as to be suitable for data collection in a variety of different rockmass types. GBI mapping data can be used directly for mine design, such as the assessment of stable stope spans and for support recommendations. In addition this data can be used in the construction of geotechnical domain models or hazard maps.

## 1. INTRODUCTION

Routine geotechnical data collection is usually achieved by logging of drillcore and/or mapping of rock exposures. The quality of data acquired by mapping is often superior in its detail, although inferior in density, to that collected from geotechnical drillcore logging. This is because a better understanding of structural fabric can be achieved by mapping than by drillcore logging. The overall characteristics of individual joint sets can be assessed, in particular their spacing, continuity (persistence), and inter-relationship.

Geotechnical mapping procedures include window mapping and scan line surveys which are conducted at the rock face in underground and open pit excavations or at natural surface exposures. Important data commonly collected in routine geotechnical mapping includes (but is certainly not limited to): rock type and strength; number of joint sets present; orientation of joint sets; joint spacing within each set; joint infill, roughness and alteration of each set; and joint set continuity.

The rockmass at each mapping locality is thereby characterised and then may be classified according to one or more of a number of industry-standard rockmass rating systems. These include the Q system (Barton et al, 1974) or modified Q' (Hutchison and Diedricks, 1996), the rock mass rating (RMR) (Bieniawski, 1989) and Geological Strength Index (GSI) (Hoek, 1994) and the mining rock mass rating (MRMR) (Laubscher, 1990). For ease of manipulation, the data is often collected using the precise descriptors or parameters required for the classification system that will be used.

These mapping methods allow for the provision of detailed geotechnical data at specific chosen localities. However it is very seldom practical to map entire extensive mine workings using these systems. The rockmass classification data collected is essentially point data, and it is often not at sufficient density to provide a means of modelling the extent and spatial distribution of areas of differing geotechnical conditions.

Geotechnical Blockiness Index (GBI) mapping was devised as a mapping method to provide a rapid data collection system for continuous description of geotechnical conditions. It provides an infill of descriptive geotechnical information between localities of detailed geotechnical data, and allows this detailed data to be placed within an overall geotechnical framework. GBI mapping is intended to supplement existing mapping methods, and is not intended to replace the detailed geotechnical mapping for rockmass characterisation and classification.

# 2. THE GBI MAPPING METHOD

The Geotechnical Blockiness Index provides a description of the rockmass in terms of the definition and nature of individual blocks of rock that are formed within the structural fabric present. The degree of blockiness refers to the effectiveness of the joint sets, fabrics or faults (primary structures) in dividing the rockmass into discrete 3-dimensional blocks, with the average sizes of these blocks being of particular importance. The main factors which contribute to the description of the rockmass in terms its blockiness, and therefore which must be evaluated in the assessment of GBI are:

- The number of effective block-forming joint sets present
- The average size of the individual blocks of rock that could be formed by these sets, which is dependant largely on the characteristic spacing of the joint sets
- The persistence of the discontinuities

These properties all have a direct bearing on the geotechnical behaviour of the rockmass, the potential for failure and the likely nature of that failure. GBI therefore provides an indication of how a rockmass will perform geotechnically in a variety of settings, based primarily on its localised structural properties.

GBI mapping is advantageous in providing a relatively rapid method for geotechnical data collection, and may be carried out on foot at a slow walking pace. GBI mapping of entire mine areas can be conducted, thereby rapidly extending the most basic, useful information concerning geotechnical conditions over the entire accessible development of a mine.

## 2.1 GBI FOR MASSIVE ROCKMASSES

The GBI descriptive index system was originally developed within a massive rockmass that was without a particularly strong or pervasive characteristic fabric. The system was developed with eight recognisably distinct conditions of blockiness that may be discerned within a typically massive rockmass. The actual detailed definition of each of the GBI indices can be modified for different mines to best accommodate the overall geotechnical environment in which mapping is to take place. The indices must be defined so as to include the full range of geotechnical conditions to be encountered. This will provide a suitable level of detail on variation in conditions within a mine.

This GBI index for massive rockmasses ($GBI_m$) consists of three primary descriptors: massive, blocky and friable. *Massive* rockmasses are those with less than three effective block forming joint sets. Distinct individual blocks of rock are seldom formed, and breakage must most often occur through intact rock for failure of the rockmass to occur. Massive rockmasses generally have greater spacing of the joints within the sets, and the continuity of individual joints is most often low (but could possibly be high for random individual joints). The block size that may be expected to be generated in such rockmasses is large to extremely large. Massive rockmasses may be subdivided into three categories (GBI 1, 2 and 3) according to the number of effective block forming joint sets that are present. These are described in Table 1.

Where there are three or more effective block forming joint sets, distinct individual blocks within the rockmass commonly occur, and little or no failure through intact rock is required for failure and break-up of the rockmass to occur. The rockmass in this case is defined as *blocky*. Blocky rockmasses may be subdivided into 4 categories (GBI 4 to 7), depending on the size of the most commonly occurring blocks (which in turn is dependant largely on the spacing of the effective block forming sets), the continuity of the joints, and the effect of primary structures on block definition.

GBI 4 is used to describe a rockmass where many primary structures (faults) intersect to divide the rockmass into large blocks that have a particularly high risk of failure as a result of the poor surface conditions on these structures. Often this class is identified where a number of faults with slickensided and/or highly altered surfaces have intersected and given rise to noticeable or severe overbreak from the backs and sometimes sidewalls of development drives. Indices 5, 6 and 7 describe rockmasses characterised by large, medium and small blocky ground respectively.

*Friable* rock (index 8) is generally found to occur in localised zones: where shearing has occurred along discrete planes; where the rock is very highly altered, in rock types of low strength which show rapid deterioration on exposure to the atmosphere; or where fracturing is particularly intense. GBI 8 is therefore often associated with highly foliated rock, dykes and fault planes.

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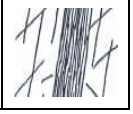
| $GBI_m$ | Suggested colour | Rockmass Description | Rockmass Fabric Description | | | Rockmass Fragmentation | | Typical RQD (%) | Figure |
|---|---|---|---|---|---|---|---|---|---|
| | | | Joint Sets | Joint Persistence | Joint Spacing | Potential for Rockmass Failure | Relative Block Size | | |
| 1 | Dark blue | Practically unjointed | Nil, only random | <2m | >5m | Along primary structures and through intact rock | Extremely Large (>20m) | 100 | |
| 2 | Light blue | Massive | 1 or 1+random | 2-4m | 3-5m | Along primary structures, jointing and through intact rock | Very Large (8-15m) | 100 | |
| 3 | Green | Massive to Blocky | 2 or 2+random | >3m | 1-3m | Along joint sets and primary structures and intact rock | Large (2-8m) | 90-100 | |
| 4 | Red | Blocky to Slabby | Dominated by primary structures. | | | Along primary structures and along joint sets | Very Large to Large (2-12m) | 80-100 | |
| 5 | Yellow | Large blocky | 2+random, 3 or 3+random | >3m | 1-3m | Along joint sets and along primary structures | Large to Medium (1-5m) | 80-100 | |
| 6 | Orange | Medium blocky | 3-5 | 2-4m | <1m | Along joint sets, may occur in structural corridors | Medium (0.5-1m) | 60-90 | |
| 7 | Pink | Small blocky | 3-6 | <2m | <0.5m | Along joint sets | Small (<0.5m) | 50-85 | |
| 8 | Maroon | Friable | Friable due to low strength, alteration, deterioration, intense fracturing, typically associated with dykes and weak fault planes | | | Defined by dyke or fault plane | Very Small (<0.10m) | <50 |  |

Table 1: Geotechnical Blockiness Index for Massive Rockmasses

## 3. DATA COLLECTION PROCESS

Mapping is carried out on foot at a slow walking pace on 1:1000 scale mine plans. GBI classes are visually identified from the appearance of the rockmass: for example blocky, massive, dominated by faults or extremely weak and friable rock. This is confirmed by a closer inspection of the number of effective black forming joint sets and joint spacing. Each GBI class is progressively coloured-in on the plan with the appropriate index colours. An example of a field mapping sheet is provided in **Figure 1**. Data in digital format is show in Figure 2.

Figure 1: A plan of underground development showing GBI mapping and collection of Q' data parameters. Primary structures mapped and interpreted are shown as lines. Diamonds indicate positions of window mapping.

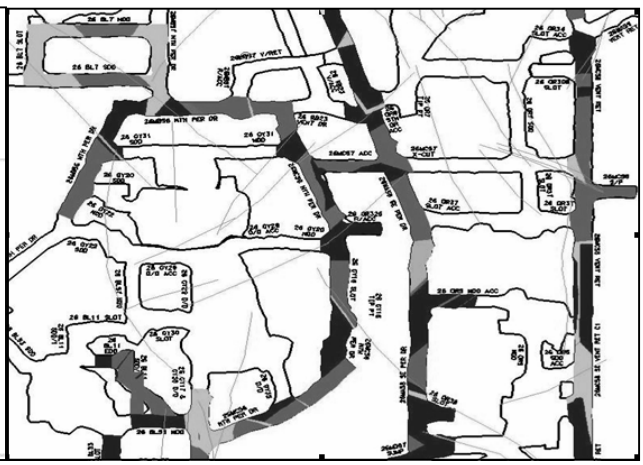


Figure 2: GBI mapping presented in digital format, also showing positions of mapped and interpreted faults. There is good correlation between faults and areas of GBI 4 and 6.

The GBI mapping method can be related to the Q' classification system (Hutchinson and Diederichs, 1996), especially the parameters RQD and Jn. RQD gives an indication of the joint spacing, while Jn describes the number of effective block forming joint sets present. During the mapping process it is also possible to collect joint roughness and infill parameters for each zone of a common geotechnical conditions (i.e. constant GBI index). From this it is possible to calculate the Q' for each GDI class mapped.

$$Q' = RQD/Jn \times Jr/Jc \quad (1)$$

Where: RQD = Rock Quality Designation
Jn = Number of effective block forming joint sets
Jr = Joint roughness
Jc = Joint infill

If the joint roughness and infill are collected during the GBI mapping, a preliminary estimate of the GSI can also be made from standard tables (Hoek, 1994, Hoek et al, 1996) by comparing the GBI rating with the structure, and joint infill and roughness with the surface condition column

It is important to understand that the aim of GBI mapping is to provide a detailed overview of the properties and quality of the rockmass in terms if its local or small-scale structural characteristics. Other factors, such as intact rock strength and the condition of the discontinuities themselves may also have a marked effect on the behaviour and stability of the rockmass observed. It can thus be explained that the values of RMR or Q' obtained from evaluation of window mapping data may have some variability even within the same GBI index. It is important that standard geotechnical mapping is carried out for detailed rockmass characterisation.

## 4. MODIFICATION FOR SHEARED OR BEDDED ROCKMASSES

Although the GBI system described was originally developed for a massive orebody, experience has shown that the technique can be modified for use in sheared or intensely bedded rockmasses or orebodies. Using the same system of rockmass descriptors, but placing more importance on the *3-D shape* of natural rockmass fragmentation, the modified GBI system (called $GBI_s$) as shown in **Table 2** can be adapted for use in rockmasses of poorer quality.

| $GBI_s$ | Rockmass Description | Discontinuity Sets | Discontinuity Spacing | Typical Potential for rockmass failure | Typical RQD (%) |
|---|---|---|---|---|---|
| 1 | Relatively Massive | 0 to 2 | >1.5m | Along joint sets and primary structures and intact rock | 90 - 100 |
| 2 | Wide slabby | 1 to 2 | >1m | Along joint sets, primary structures and intact rock | 90 - 100 |
| 3 | Wide columnar | 2 to 2+R | >1m | | 80 - 100 |
| 4 | Large blocky | 2+R to 3+R | >1m | Along joint sets and primary structures | 80 - 100 |
| 5 | Slabby | 1 to 2 | 0.1 - 1m | Along joint sets, and through intact rock | 50 - 90 |
| 6 | Columnar | 2 to 2+R | 0.1 - 1m | | 50 - 80 |
| 7 | Small blocky | 3 to 6 | 0.1 - 1m | Along joint sets | 30 - 80 |
| 8 | Fissile | 1 to 6 With pervasive set | <0.1m | Along pervasive fabric and joint sets | 0 - 30 |

Table 2: Geotechnical Blockiness Index for Rockmasses including Sheared or Bedded Materials

## 5. APPLICATION OF GBI MAPPING RESULTS

With the application of local experience, GBI mapping data can be used directly in design and in the assessment of operational requirements. Examples of these are:

- Use in underground stope stability assessments (along with proximal Q' data)
- Confirmation of ground support requirements relevant to ground conditions
- Assessment for blast design (linking blast hole layout and powder factors to expected rock conditions)

GBI mapping is of particular use for the interpretation and generation of large scale plans or models for prediction of ground conditions. These are used to assist in advance planning design of excavations and for prediction of difficulties, and may include:

- Geotechnical domain models
- Geotechnical hazard maps

### 5.1 AN EXAMPLE OF GBI MAPPING FOR GEOTECHNICAL DOMAIN INTERPRETATION

An example of an application of the GBI for rockmass characterisation and geotechnical domain modelling at an Australian underground mine is detailed here.

The rockmass at the mine is massive with generally widely spaced joints. The majority of the rockmass has an RQD greater than 90%, so this parameter is not sensitive enough for geotechnical domaining. Geotechnical conditions are not linked solely to the rock type, which is highly variable across the mine area. General trends in the rockmass include more blocky and fault-dominated zones at the centre of the orebody. The rockmass also becomes more competent with depth. Although these trends were known prior to the introduction of the GBI mapping system, this was the first time the description and specific distribution of ground condition patterns were able to be well documented.

GBI mapping was carried out at 1:1000 scale in order to collect continuous mapping data and to tie this in with the positions of faults and shears which individually greatly influence ground stability. In this way the GBI mapping was correlated with the structural geology interpretation of the mine.

Major structures are expressed in different ways within the various materials comprising the rockmass. In places these are expressed as corridors of closely jointed, highly blocky material. In other places the major structures are expressed as zones of very poor, unstable ground with multiple interacting fault planes each with weak infill or poor surface conditions. Major structures were also observed simply as discrete planes with infill ranging from rock gouge infill to thin chlorite infill. The GBI mapping helped to characterise the rockmass in the vicinity of structures of varying orientations (i.e. often of different phases of deformation) within each of a variety of material types.

Because of the variability of geotechnical conditions over short distances within the mine, the detailed GBI mapping was used for the interpretation of four geotechnical domain conditions. Each of the four domain conditions were made up of a combination of ground conditions. Domain A consisted of massive ground (GBI 1, 2 and 3). Domain B consisted of large blocky ground, which in places was massive (GBI 3 and 5). Domain C consisted of blocky ground with some faults (GBI 3, 5, 6 with occasional 4). Domain D was dominated by primary structures, with the rock between the primary structures varying from massive to blocky (GBI 4, 5, 6, 7, 8 and 3). These are summarised in Table 3. The distribution of these domain conditions was expressed as a wireframe model and then subsequently block modelled. An example of part of the wireframe model is presented in Figure 3.

| Domain | Colour | Ground Conditions | Dominant GBI | Primary Structures | Likelihood of Ground Instability |
|---|---|---|---|---|---|
| **A** | Blue | Massive rock with few primary structures | 1, 2, 3 | Very widely spaced primary structures | Low |
| **B** | Green | Massive to blocky rock with widely spaced primary structures | 3, 5 | Widely spaced primary structures | Low to medium |
| **C** | Yellow | Blocky rock with a moderate number of primary structures | 5, 3, 6, 4 | Individual and/ or interacting primary structures | Medium to high |
| **D** | Red | Blocky rock with numerous interacting primary structures | 4, 6, 7, 8, 3, 5 | Numerous primary structures | High |

Table 3: Domain Index Conditions

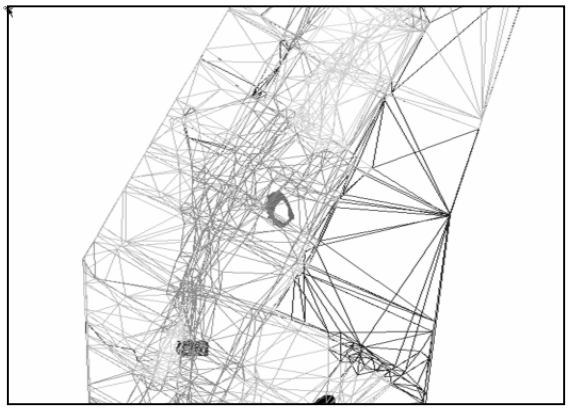


Figure 3: Wireframe model of the lateral distribution of the four geotechnical domain conditions

## 6. CONCLUSION

It can be seen that the use of GBI mapping as part of routine or targeted geotechnical investigation may provide the following advantages:

- An additional tool for geotechnical mapping, especially for recording the detailed distribution of different geotechnical conditions.
- A rapid mapping technique that is easy to learn, and to apply.
- Provision of increased coverage of geotechnical data (when compared to window mapping or scan lines survey).
- Provision of geotechnical data that is descriptive of the natural 3-D fragmentation of the rockmass, and that can be related to and correlated with the structural, lithology and alteration models.
- Rapid assessment and collection of Q' data parameters for each zone of common GBI. RQD and Jn are continually assessed in the assignment of the GBI, and roughness and infill data can be quickly recorded in addition.
- Adaptation for collection of appropriate data in a variety of different rockmass/orebody types.

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