Australian Geomechanics Society

Sydney Chapter

MINI-SYMPOSIUM

GEOTECHNICAL INSTRUMENTATION AND CONSTRUCTION WORKS COMPLIANCE TESTING

WEDNESDAY, 13 AUGUST 2003

Auditorium, Eagle House, Milsons Point, Sydney

Scholey, G.K. and Walker, A.B, Organising Committee

ISBN 0858257416

Published and printed 2003 by The Sydney Chapter of the Australian
Geomechanics Society, 118 Alfred Street, Milsons Point, NSW 2061

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Australian Geomechanics Society

Sydney Chapter

Mini-Symposium – Wednesday, 13 August 2003
Geotechnical Instrumentation and Construction Works Compliance Testing

Dear Colleague

Welcome to today's Mini-Symposium on *Geotechnical Instrumentation and Construction Works Compliance Testing.* This Symposium is the seventh of the Sydney Chapter's annual symposiums designed to keep the engineering profession aware of recent developments in particular aspects of geotechnical engineering practices. We hope that you find the presentations to be both interesting and informative. We are pleased you have been able to set aside time from your busy schedule to attend and participate in this monthly meeting of the Australian Geomechanics Society.

ACKNOWLEDGEMENTS – The AGS acknowledges with thanks the financial assistance and support of **Geotechnical Systems** (Platinum Sponsor); **SMEC Australia, Pells Sullivan Meynink, Golder Associates and Geosystems** (Gold Sponsors); and **Slope Indicator Company** (Silver Sponsor).

REGISTRATION DESK – The AGS organising committee at the registration desk are happy to deal with any queries you may have.

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QUESTIONS – You are encouraged to ask questions at the end of the presentations, if time permits, as well as in the "Questions and Answers" session.

CONFERENCE EVALUATION – You will have been provided with an evaluation form when you registered today. We would appreciate your help in completing this, as it is an important source of information on how we can improve our future symposiums. We suggest that you fill this out progressively throughout the conference, while your memory is still fresh for each presentation. At the completion of the conference, please leave the evaluation with a committee member or in the box at the registration desk.

AGS MEMBERSHIP – Information regarding membership can be found in *Australian Geomechanics* published by the Australian Geomechanics Society, from any AGS committee member, or from the AGS web site: www.australiangeomechanics.org

CONTINUING PROFESSIONAL DEVELOPMENT – Full-time attendance at today's mini-symposium entitles you to 4.5 hours of CPD.

Please do not hesitate to contact us, or one of the AGS committee members, if you have any further questions or problems.

Graham Scholey and Tony Walker
For the Symposium Organising Committee
Australian Geomechanics Society, Sydney Chapter

Australian Geomechanics Society
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FEEDBACK FORM

Mini-Symposium – Wednesday, 13 August 2003
Geotechnical Instrumentation and Construction Works Compliance Testing

Auditorium, Eagle House, Milsons Point

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Job Title
Company
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Dear Colleague

Thank you for attending our 2003 Mini-Symposium. To help us evaluate this year's Mini-Symposium and plan future functions, please give us your honest feedback.

Thank you for your time.

What attracted you to this event?

- ☐ Agenda: Which Talks:
- ☐ Speakers: Any in Particular:
- ☐ Networking Opportunities:
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What business related issues are of most concern to you now and for the future?

Do you visit any work related web sites regularly? If so give address below:

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Jeff Hsi

	Excellent	Very good	Good	Average	Poor
Content					
Delivery					
Comments					

Stephen Jones

	Excellent	Very good	Good	Average	Poor
Content					
Delivery					
Comments					

Colin Viska

	Excellent	Very good	Good	Average	Poor
Content					
Delivery					
Comments					

Doug Goad

	Excellent	Very good	Good	Average	Poor
Content					
Delivery					
Comments					

Philip Pells

	Excellent	Very good	Good	Average	Poor
Content					
Delivery					
Comments					

David Airey

	Excellent	Very good	Good	Average	Poor
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Delivery					
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Julian Seidel

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Content					
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In summing up this event in one or two sentences what would you say? For example was there anything about the event you found particularly enjoyable/interesting? Any problems or suggestions?

We are currently planning our 2003 Mini-Symposium.
Are there any subjects on which you or your company would be interested in speaking?

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TABLE OF CONTENTS

PREFACE

PROGRAMME

TECHNICAL PRESENTATIONS

- "Geotechnical Instrumentation for the Yelgun to Chinderah Freeway" by Dr Jeff Hsi, SMEC Australia
- "Instrumented Preload at Kooragang Island" by Stephen Jones, Douglas Partners
- "The Application and Use of In-Place Inclinometers" by Colin Viska, Slope Indicator Company
- "Settlement Behaviour of Deep Fill for Housing Development, Niddrie, VIC " by Doug Goad, Golder Associates
- "A Note on Real Time Monitoring of Tunnel Invert Slabs During Stressing" by Dr Philip Pells, PSM
- "Using Bender Elements to Measure Soil Stiffness at Small Strains in Laboratory Samples" by Dr David Airey, University of Sydney
- "Compliance Testing and Instrumentation of Piles" by Dr Julian Seidel

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Australian Geomechanics Society

Sydney Chapter

Preface

The Sydney Group of the Australian Geomechanics Society, a body jointly sponsored by the Institution of Engineers, Australia and The Australian Institute of Mining and Metallurgy, has selected the theme of geotechnical instrumentation and construction works compliance testing for its 7th annual mini-symposium. The objective of these annual mini-symposia has been to inform practising Geotechnical Engineers and Engineering Geologists of recent developments in different disciplines of ground engineering, to include experiences and practise which are specific to our region. Previous topics have included landslide risk management, excavation retention, pavements, piles and computer methods.

The Organising Committee, on behalf of the Sydney Chapter of the Australian Geomechanics Society, approached local practitioners to contribute papers and case histories involving geotechnical instrumentation and construction works compliance testing. The expectation had been that papers would be submitted that addressed the use of different geotechnical instrumentation types and systems to monitor compliance at the construction stage. Whilst most of the papers are in that vein, contributions also include papers which address specific instrumentation types, both field and laboratory.

The case histories include discussions of geotechnical design and construction practise. Case histories include the use of geotechnical instrumentation for monitoring the settlement of soft ground and thick engineered fill during construction and for monitoring remedial and repair works of a tunnel. The focus on instrumentation types includes laboratory methods, field instrumentation of piles, and the real time monitoring of inclinometers.

This symposium is a cooperative effort of many authors. The Editors wish to thank the authors, who have so generously contributed their time to prepare the various papers, and the employers of the authors, who have assisted with time, secretarial, drafting and photocopying facilities.

The Editors hope that the information presented will prove to be a valuable source of information for geotechnical engineers and engineering geologists working in the Sydney region.

Graham Scholey

Tony Walker

(Editors)

Australian Geomechanics Society

AGS SYDNEY CHAPTER MINI-SYMPOSIUM, 13 AUGUST 2003

HARRICKS AUDITORIUM, ENGINEERS AUSTRALIA
EAGLE HOUSE, MILSONS POINT, SYDNEY

Geotechnical Instrumentation and Construction Works Compliance Testing

PROGRAMME

2:00 *Registration*

2:30 Opening Address
Graham Scholey, Golder Associates

2:40 Sponsor Presentation
John Lakeland, Geotechnical Systems

Session 1

2:50 Geotechnical Instrumentation for the Yelgun to Chinderah Freeway
Jeff Hsi, SMEC

3:20 Instrumented Preload at Kooragang Island
Stephen Jones, Douglas Partners

3:50 Session 1 – Questions

Session 2

4:00 Sponsor Presentation
Santhira Sekaran, Geosystems

4:10 The Application and Use of In-Place Inclinometers
Colin Viska, SINCO

4:30 Settlement Behaviour of Deep Fill for Housing Development, Niddrie, VIC
Doug Goad, Golder Associates

5:00 A Note on Real Time Monitoring of Tunnel Invert Slabs during Stressing
Philip Pells, PSM

5:30 Session 2 – Questions

5:40 *Supper*

Session 3

6:10 Using Bender Elements to Measure Soil Stiffness at Small Strains in Laboratory Samples
David Airey, University of Sydney

6:30 Compliance Testing and Instrumentation of Piles
Julian Seidel, Foundation QA

7:10 Session 3 – Questions

7:20 Closing Address

7:30 *Close*

This programme is subject to amendment at any time prior to and during the event.

Full-time attendance qualifies for 4.5 hours of CPD.

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Geotechnical Instrumentation for the Yelgun to Chinderah Freeway

Dr Jeff Hsi
SMEC Australia

Mini-Symposium
Geotechnical Instrumentation and
Construction Works Compliance Testing

Presented at:

Eagle House, Milsons Point
Sydney, Australia
August 2003

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GEOTECHNICAL INSTRUMENTATION AND MONITORING FOR YELGUN TO CHINDERAH FREEWAY

Jeff Hsi
Manager, Geotechnical Services, SMEC Australia Pty Ltd

ABSTRACT

A large portion of the Yelgun to Chinderah Freeway was located in areas underlain by soft soils. Construction of the road embankments in these areas presented significant constraints and risks to the project, involving short term instability and long term settlement. Due diligence was carried out to ensure development of effective solutions, and control and reduction of risks. Geotechnical instrumentation and monitoring formed a crucial part of the soft ground treatment. The design and construction were constantly reviewed and modified as required in response to the field performance. This process allowed flexibility in design and construction and was proved to be successful and cost effective.

1. INTRODUCTION

Sections of the recently completed Yelgun to Chinderah Freeway in the northern region of New South Wales traversed flood plains and marshy areas. The subsurface consisted of mainly very soft and loose alluvial soils, imposing significant geotechnical constraints on the construction of the road embankments. Developing robust solutions and managing the risks were the most technical challenging features of the project.

Typical problems of constructing road embankments over soft ground include embankment instability, excessive ground settlement, prolonged construction time and large post-construction settlement. Extensive soft ground treatments are generally required to maintain embankment stability during construction and minimize ground settlement after project completion. The success of such treatments depends on the understanding of the soft ground behaviour and the accurate prediction of its performance. Failure to do so will render the work unserviceable and incur considerable cost overrun.

Soft ground related works generally involve substantial uncertainties, e.g. variation in soft soil profile and property, consolidation behaviour, soft ground treatment performance, etc. To reduce risks these uncertainties need to be eliminated as far as possible. Measures taken to control and minimize the risks for the Yelgun to Chinderah Freeway consisted of extensive geotechnical investigations, field trials, and instrumentation and monitoring. An integrated approach involving design, construction, and field trials and monitoring was implemented to ensure effectiveness of risk control. The information gathered from the site was used for calibration of design and modification of construction, as appropriate.

2. PROJECT DESCRIPTION

The Yelgun to Chinderah section of the Pacific Highway Upgrade provides a safe new four lane divided freeway bypassing the notorious Burringbar Ranges in the north of NSW. The 28.5 km long $348 million (AUD) motorway cuts through a series of very steep sided ridges and traverses a series of valleys before crossing the Tweed Valley flood plains.

The freeway commences just south of Yelgun Creek and rejoins the existing Pacific Highway west of Chinderah. The project route plan is shown in Figure 1. Key project features include:

- 4 lanes of median divided carriageway
- 4 major fauna movement structures including 2 cut and cover tunnels and 2 large arch underpasses
- 9 overbridges
- 39 freeway bridges over creeks and waterways
- High quality, concrete pavement carriageway with 110 km/hr design speed
- 3 grade separated interchanges located at Cudgera Creek Road, Clothiers Creek Road and Oak Avenue
- 6 million cubic metres of earthworks
- Over 50 fauna mitigation structures along the project

- 10 km of embankments constructed over soft ground with approximately 1.4 million lineal metres of wick drains used
- 134 m long twin tunnels under Cudgen Road, designed to avoid cane land and reduce rural impact

The Yelgun to Chinderah Freeway was awarded to Abigroup Contractors as a design, construct and maintain (DCM) project by the Roads and Traffic Authority of NSW, with SMEC Australia being the Principal Designer. The project commenced on 8th December 1999 with a contract completion date of 15th December 2002 and a contract maintenance period of 10 years after completion of construction. Substantial completion was achieved by June 2002 and the road was opened to traffic on 6th August 2002, 4 months ahead of the scheduled completion date.

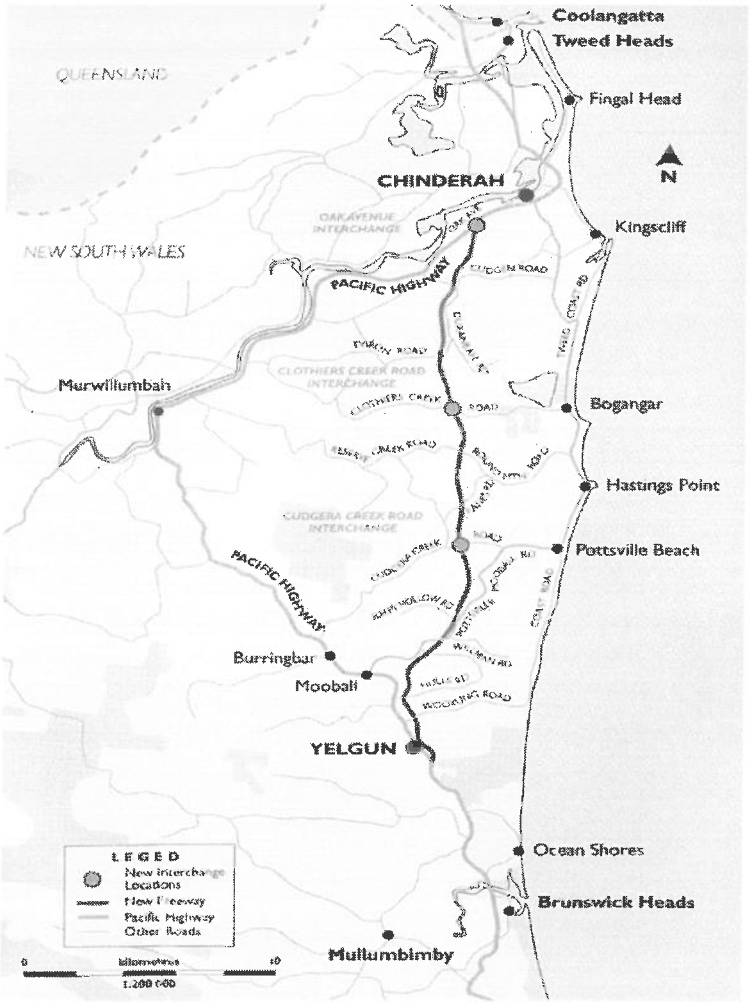

Figure 1: Project route plan

3. PROBLEMS AND RISKS

The soft soils underlying a large proportion of the project site comprised dark grey to black clays deposited under estuarine or marine conditions. The deposition of sediment in these areas was dominated by silts and clays transported in flood waters and deposited in thin layers on the flood plains and lagoons. Organic material mixed with the soft clays formed a soft, highly compressible material with a high water content. The soft soils of the project site extended up to 15 m depth and had the following characteristics:

- Low shear strength
- High compressibility
- Low permeability

It was recognized that the soft soils of the project site imposed major geotechnical risks on the project. These are described below:

- Embankment filling over soft ground. Associated problems involved excessive settlement of the embankment during construction causing an increase in fill quantity, long term total and differential settlement affecting the serviceability of the road and instability of the embankment during embankment construction. The slow consolidation nature of the soft soils would also have a major impact on the construction time and process.
- Approach embankments to bridge abutments and other structures. Differential settlement was a major concern in areas where the embankments were constructed over soft ground adjacent to bridge abutments and structures supported on piles. Settlement of the embankment would not only affect the pavement performance but also impose additional loads on the piles supporting the structure.

The problems of the project in relation to soft soils involved both short term and long term performance. The constructability of the embankment during construction and the serviceability of the road after the project completion were of major concerns. Photos 1 and 2 show large ground movement associated with construction of the embankment.

4. SOFT SOIL TREATMENTS

4.1 EMBANKMENTS

The embankments located over soft ground were designed in such a way that stability of the embankment was maintained during construction and the long term settlement of the embankment complied with the criteria.

To meet the time constraints of the project the embankment needed to be constructed as quickly as practically possible to allow for a maximum duration of preloading in order to achieve the required degree of consolidation during construction. However, speedy construction of the embankment might be followed by an embankment failure.

Measures considered to improve the embankment stability included the use of high strength geotextile placed at the base of the embankment and wick drains installed through the soft clays. The geotextile provided a restoring force against potential slip failure of the embankment, whilst the wick drains helped the ground to gain strength via the consolidation process. This system relied on the combined effects of the geotextile and the wick drains connected to a drainage blanket placed at the base of the embankment to achieve the required stability of the embankment. The ground water squeezed out via the wick drains and the drainage blanket was collected and treated before being discharged back to the ground. Photos 3 and 4 show the placement of geotextile below the base of the embankment, and Photos 5 and 6 show the installation of wick drains through the drainage blanket.

To satisfy the performance of the pavement as well as the drainage and flood requirements, the residual settlement of the embankment constructed over soft ground needed to be controlled to within a specified limit after completion of the project. Surcharging of the embankment with wick drains installed in the soft ground was used to generate early and speedy settlement in the ground. This approach allowed most settlement to occur during the period of construction and over-consolidated the soft soils to minimize the residual settlement after completion of the project.

4.2 BRIDGE APPROACHES

At bridge approaches where the embankments were constructed over soft ground the ground would settle during the course of construction and continue to settle after completion of the work. This embankment settlement would have an adverse impact on the adjacent piles supporting the bridge abutment and would also cause settlement of the pavement connecting to the bridge abutment resulting in loss of its serviceability.

To eliminate the impact of embankment settlement on the abutment piles a nest of driven piles consisting of timber piles and precast concrete piles was installed in the area adjacent to the abutment. A series of pile caps overlain by a layer of geotextile reinforced mattress was placed over the piles to form an effective bridging layer to transfer the embankment loads on to the piles. These piles then carried the full embankment loading and hence no ground settlement would occur. This method allowed for earlier construction of the abutment piles and hence earlier completion of the bridge to enable haulage and construction traffic through the alignment. Photos 7 and 8 show the installation of timber piles and the constructed pile caps.

There was a potential for large differential settlement at the interface between the piled and non-piled embankment. Heavy surcharge and closely spaced wick drains were used to over-consolidate the ground next to the piled embankment. This allowed for accelerated consolidation and reduction in long term creep settlement of the non-piled embankment. In addition, a layer of geotextile reinforced mattress together with reinforced concrete pavement was constructed across the piled and non-piled embankment to allow for smooth transitions of the pavement during its design life. Figure 2 shows the arrangement of the bridge approach treatment.

This was an integrated solution combining treatments of the foundation soil, the embankment and the pavement. The method overcame the differential settlement at the bridge abutment and allowed for early construction of the bridge. It was proved to be cost effective and satisfied relevant design criteria for bridge approaches.

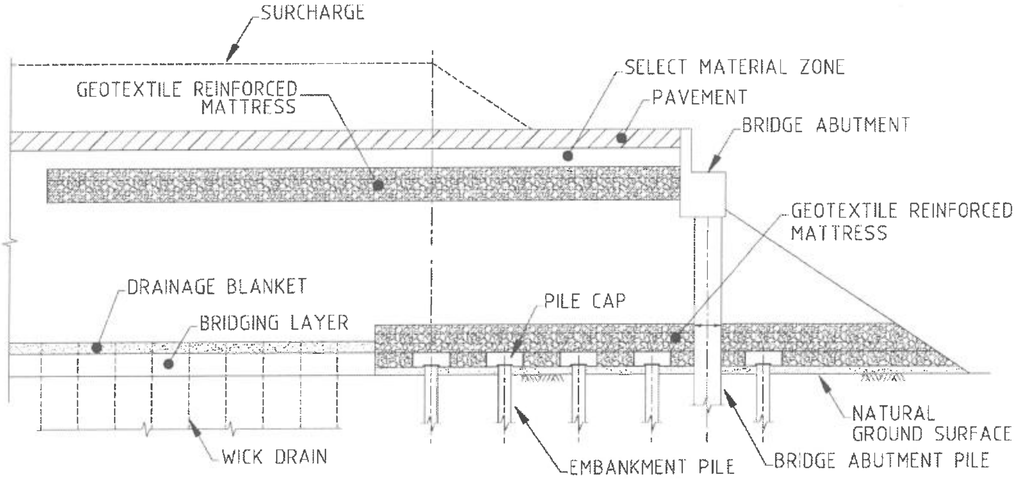

Figure 2: Bridge approach treatment

5. FIELD TESTING AND INSTRUMENTATION

5.1 GENERAL

Substantial uncertainties are generally present when constructing a road embankment over soft ground. These include variation of subsurface conditions, consolidation characteristics of the soft soil, long term creep settlement, wick drain performance, effectiveness of the drainage blanket, etc. To reduce the uncertainties and hence the risks, the following measures were adopted:

- Geotechnical investigations – to detect the subsurface conditions and the material properties
- Trial embankments – to assess the soft soil characteristics and effectiveness of the foundation treatment
- Instrumentation and monitoring – to confirm design assumptions and embankment performance

The investigation results and the field measurements were used to verify the ground conditions and confirm the design assumptions. When necessary the geotechnical models assumed in the design were further calibrated to reflect the actual ground behaviour. The calibrated geotechnical models were then used for the prediction of the long term performance of the embankment.

5.2 GEOTECHNICAL INVESTIGATIONS

Extensive geotechnical investigations were carried out for the project, including:

- Over 360 boreholes
- Over 120 electric friction tests (CPT)
- Over 70 piezocone tests (CPTU) with pore pressure dissipation tests (PPDT)
- Extensive test pitting
- Extensive field and laboratory tests

The investigations provided information on subsurface conditions and material characteristics of the site.

5.3 TRIAL EMBANKMENTS

The design of the embankment was initially based on the interpretation of limited geotechnical information available. Due to the complexity of the geology of the site, the actual ground conditions were expected to vary from the assumptions adopted in the design. Trial embankments were constructed at the early stage of the project to provide useful information for further calibration of the design assumptions and modification of the design.

The trial embankments were extensively instrumented with different arrangements of the soft ground treatments tested. These arrangements included:

- Different types of wick drains
- Different spacings of wick drains
- Different rates of embankment construction
- Different strengths of geofabrics

The arrangement of one of the trial embankments is shown in Figure 3. The performance of the trial embankments demonstrated the effectiveness of the ground treatment measures, as well as assisted in the determination of the construction program and method.

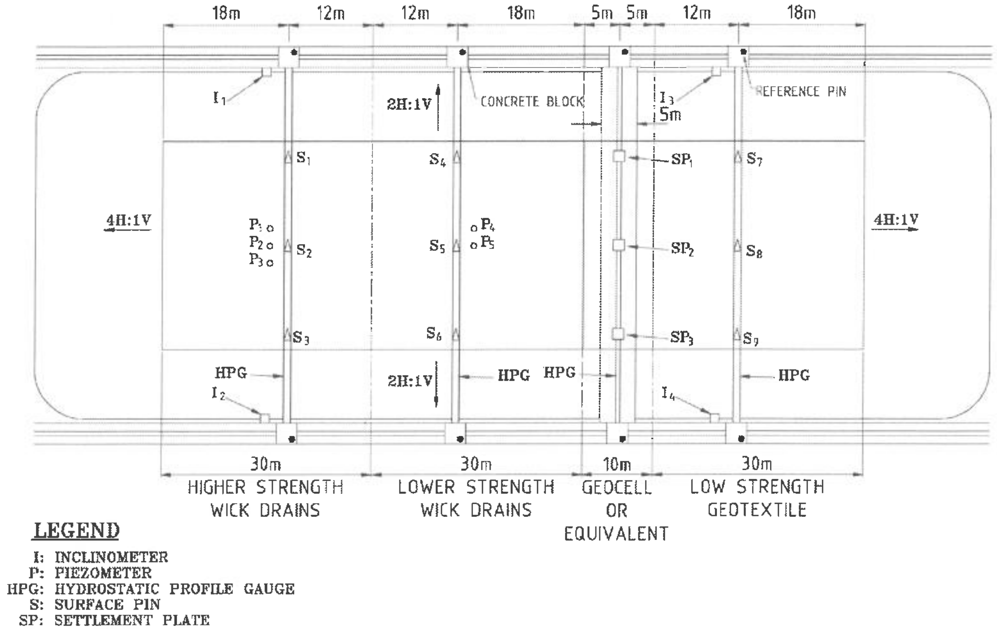

Figure 3: Arrangement of trial embankment

5.4 INSTRUMENTATION AND MONITORING

Extensive field instrumentation was implemented for the embankments located in the soft ground areas. A typical section of the instrumented embankment is shown in Figure 4. This included:

- Hydrostatic profile gauges – installed at the base of the embankments to measure the settlement profiles of the natural ground surface across the embankment
- Piezometers – installed at different depths in the foundation to measure pore water pressures in response to embankment construction
- Inclinometers – installed below the toe of the embankment to measure lateral soil movements as a result of embankment settlement
- Settlement plates – installed at the base of the embankment to measure settlements of the natural ground in relation to embankment construction
- Surface pins – installed at the surface of the preload embankment to measure the rate of settlement along and across the embankment

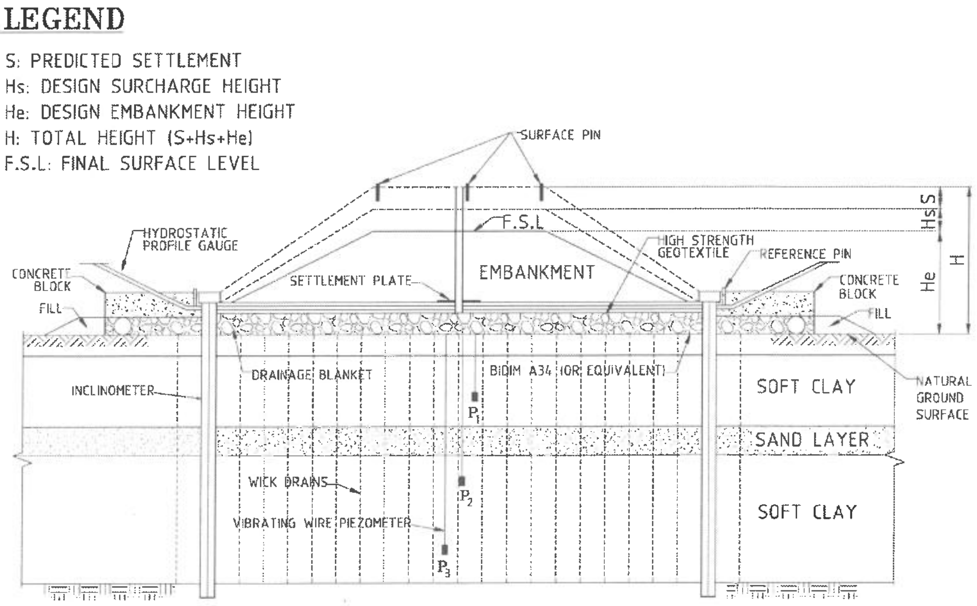

Figure 4: Instrumented embankment

A comprehensive computer data system was developed to manage and process the vast amount of monitoring data. This system was located on a designated network server which could be accessed by both the site staff and the designer. The information was periodically updated on site and was frequently monitored by the designer to assess the preload performance. Real time monitoring and review were possible through this system when the situation became critical.

The monitoring data were used by the designer for performing back analysis to match the predictions with the field measurements. The modified geotechnical models were then used for the forecast of the long term embankment performance, as well as for the determination of surcharge removal which was a crucial component of the construction program of the project.

6. FIELD PERFORMANCE

The performance of embankments during construction was assessed based on the extensive field instrumentation and monitoring. Adjustment of the construction method and the ground treatment measure was carried out accordingly to ensure that the design requirements were met.

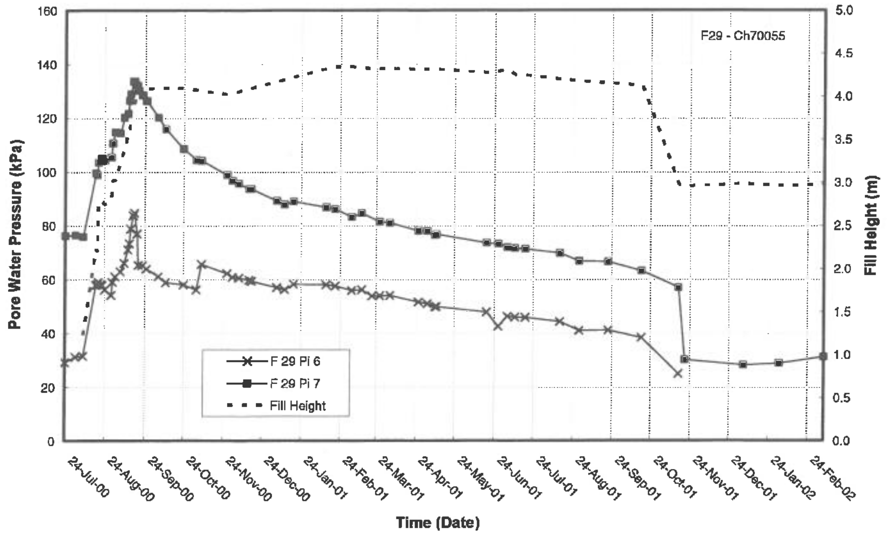

Figure 5: Pore pressure measurements

Performance of the soft ground treatment was measured by:

- Dissipation of pore water pressures showing the rate of consolidation. A typical example of measured pore water pressures in relation to the constructed embankment height is shown in Figure 5.
- Settlement of the embankment showing the rate and magnitude of consolidation. An example of measured settlement profiles below the base of the embankment is shown in Figure 6.
- Lateral deformation of soil showing the level of stability during construction. A typical example of measured lateral deflection profiles below the toe of the embankment is given in Figure 7.

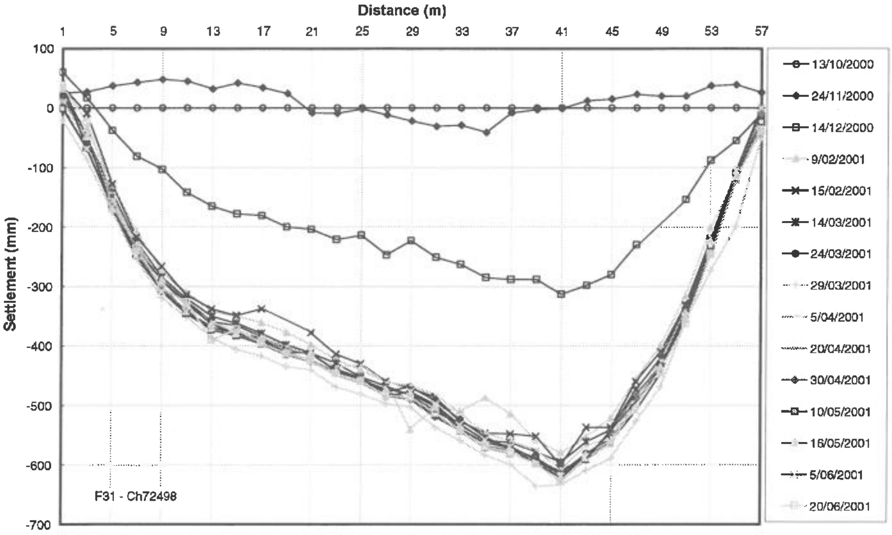

Figure 6: Measured settlement profiles

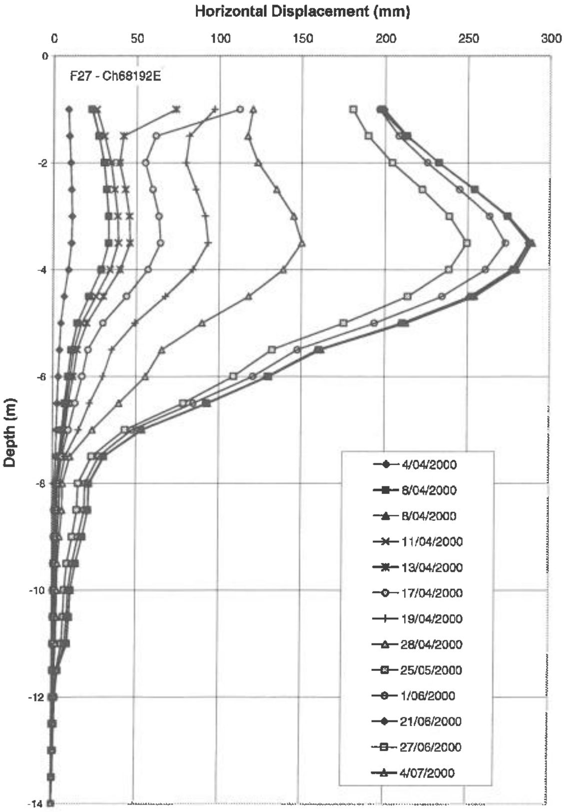

Figure 7: Measured lateral deflections

The settlement data from the settlement plates and pins were further back analysed to predict the long term total and differential settlements, as shown in Figures 8 and 9 respectively. Prior to the removal of surcharge and construction of the pavement, the predictions indicated that the long term settlement criteria, i.e., a residual settlement of 100 mm or 160 mm for specified locations in 40 years and a differential settlement of within 0.3% change in grade, were met.

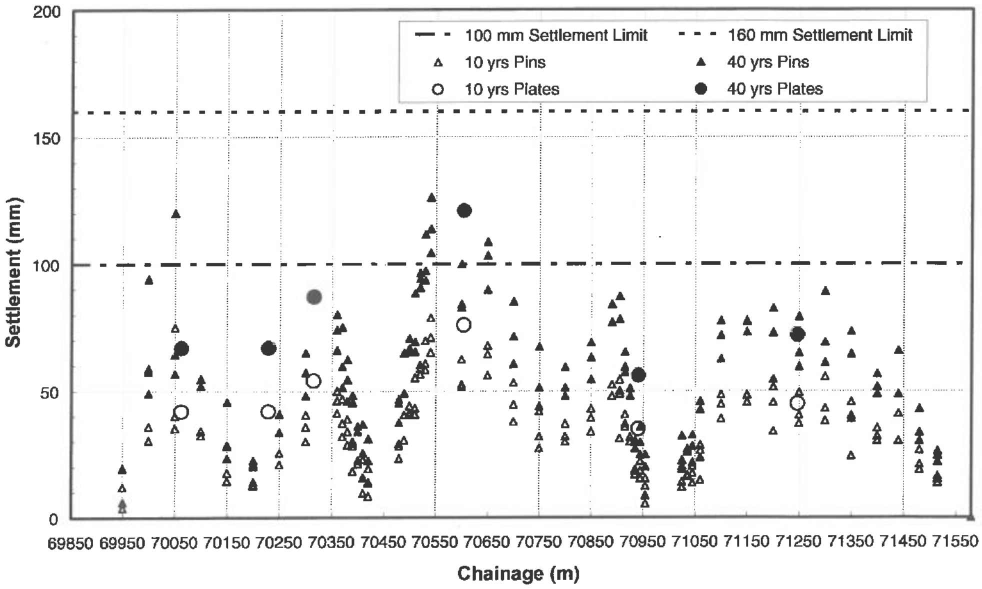

Figure 8: Predicted residual settlements

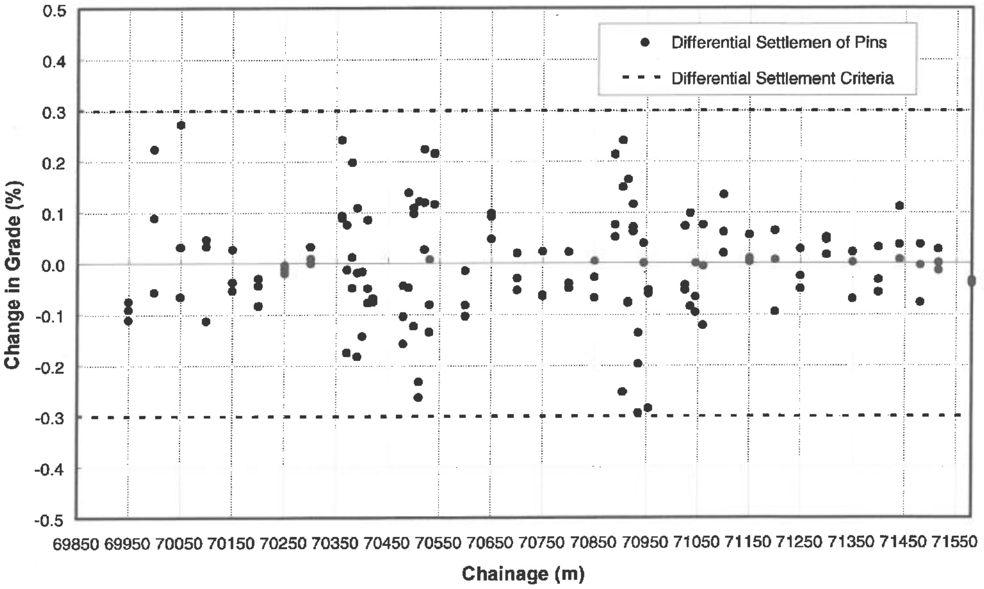

Figure 9: Predicted differential settlements

Further settlement measurements were taken after completion of the work. This information showed reduced rate of creep settlement which closely matched the predictions made prior to the removal of surcharge, as shown in Figure 10.

7. CONCLUSIONS

The 28.5 km long Yelgun to Chinderah Freeway was recently constructed and open to traffic, 4 months ahead of the contract completion date. The major challenges of the project were to construct the approximately 10 km long road embankments over soft soils. Specific soft ground treatment solutions were developed to ensure speedy construction and long term performance. Extensive geotechnical instrumentation was implemented to monitor the performance of the work. The field monitoring data were used to confirm the design assumptions and to modify the construction method as appropriate, as well as to reduce the uncertainties and risks. A computer data process system was set up on a network server which the contractor and the designer could access at all times allowing real time data update and review. This approach optimized the construction program and contributed to the early completion of the project.

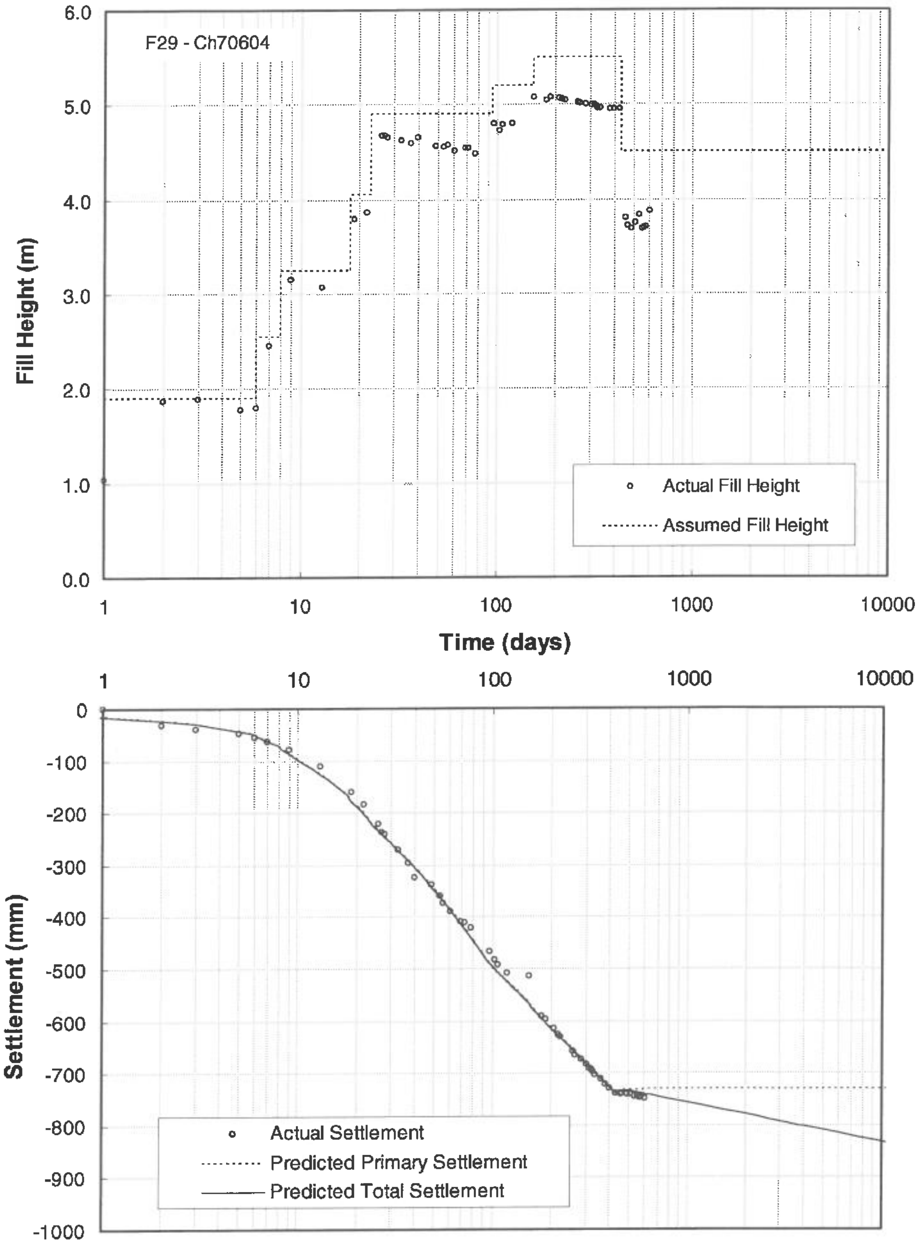

Figure 10: Measured and predicted settlements

8. ACKNOWLEDGEMENT

The author would like to thank the Roads and Traffic Authority of NSW and Abigroup Contractors Pty Ltd for permission to publish this paper.

Photo 1: Lateral soil movement adjacent to toe of embankment

Figure 2: Ground heaving adjacent to toe of embankment

Photo 3: Placement of geotextile over drainage blanket

Photo 4: Placement of fill over geotextile

Photo 5: Installation of wick drains

Photo 6: Close-up view of wick drainage rig

Photo 7: Installation of timber piles

Photo 8: Constructed pile caps

Australian Geomechanics Society

Sydney Chapter

Instrumented Preload at Kooragang Island

Stephen Jones
Douglas Partners

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INSTRUMENTED PRELOAD AT KOORAGANG ISLAND

Stephen R Jones
Douglas Partners Pty Ltd, Newcastle, Australia

ABSTRACT

The ''Stage 3 Expansion'' of the Kooragang Coal Terminal was a major industrial project which boosted the coal exporting capacity of Newcastle, making the terminal the largest coal handling facility in the world. Port Waratah Coal Services appointed Bechtel Australia in July 1999 to design and construct the expansion, which was completed early 2002.

Douglas Partners Pty Ltd provided geotechnical, environmental and earthworks testing services for design and construction purposes. The site is underlain by soft estuarine sediments, which are susceptible to settlement via consolidation and creep. The investigation included extensive use of cone penetration tests (including standard friction cone, piezocone and seismic cone) to profile the subsurface conditions and provide data for settlement estimates.

A feature of the geotechnical design was the use of a two-stage 9 metre high preload to consolidate and improve ground conditions prior to construction of the 1.2 km long coal pad and reclaimer berm. The preload was extensively monitored and the geotechnical instrumentation included settlement monitoring plates, vibrating wire piezometers, inclinometers, earth pressure cells and one extensometer installed to the bedrock at approximately 50 metres depth.

This paper describes the geotechnical investigation, preload design, settlement predictions and the monitoring results, comparing actual to predicted performance.

1 INTRODUCTION

The Stage 3 Expansion of the Kooragang Coal Terminal was a $330 million project which boosted the coal exporting capacity of the terminal to 89 million tonnes per year. The Kooragang Coal Terminal, operated by Port Waratah Coal Services Limited, is located at Kooragang Island, in the port of Newcastle, NSW.

Kooragang Island was formed through the extensive reclamation of small islands, shallows and channels and contains deep estuarine sediments which are susceptible to settlement via consolidation and creep.

The site contains the world's largest coal handling operation. Coal is transported primarily by rail from Hunter Valley mines, emptied in the receival (dump) station and conveyed to rail mounted stackers which place coal in designated stockpile areas. Coal is then reclaimed from the stockpiles by bucket wheel reclaimers and transported via conveyors to the shiploaders for export.

Port Waratah Coal Services engaged Bechtel Australia Pty Ltd to undertake engineering, procurement and construction management for the Stage 3 Expansion, which included a third stacking and receival conveying stream, rail receival (dump) station, stockpile pad and reclaimer, shiploading conveying stream and new shiploader.

Due to the presence of deep, soft estuarine sediments, ground improvement by way of staged preloading was undertaken to reduce post construction settlements prior to the construction of the coal pad and reclaimer berm.

2 GEOTECHNICAL INVESTIGATION

An extensive geotechnical investigation program was undertaken to assess site conditions (Douglas Partners 2000). The site was particularly suited to cone penetration testing (CPT), due to the presence of soft estuarine sediments. The investigation comprised a combination of CPT, piezocone tests, pore pressure dissipation tests, seismic CPT, conventional boreholes and jetted bores, shear vane tests and test pits. A range of laboratory tests including oedometer consolidation, triaxial, permeability, Atterberg limits, sieve analysis and hydrometer tests were also undertaken to characterize soil conditions.

The interpreted geotechnical soil profile is shown in Table 1 in a simplified form:

Table 1: Simplified Geotechnical Profile

Layer	Description	Thickness
FILL	Mainly loose to medium dense sand	1 m to 5 m
SILTY CLAY (upper clay)	Soft alluvial silty clay (s_u = 15 to 20 kPa)	1 m to 4 m
SAND	Dense to very dense alluvial sand	23 m to 28 m
CLAY (deep clay)	Stiff to very stiff estuarine clay / sandy clay	7 m to 12 m
BEDROCK	Siltstone / Sandstone bedrock – very low to low strength	-

The groundwater regime included two aquifers: an upper unconfined aquifer in the sand fill, and a lower semi-confined aquifer beneath the upper silty clay.

The main geotechnical issue at the site was the presence of the upper clay layer with respect to consolidation and creep. The presence of the dual aquifer system also had implications for deep excavation works.

Engineering parameters estimated from CPT data were processed together with the results of laboratory testing to produce continuous profiles of parameters such as shear strength, over consolidation ratio, coefficient of volume change, drained modulus and friction angle. The data was entered into a purpose written spreadsheet which provided the projected time-settlement behaviour at each CPT location, based on one-dimensional consolidation theory. The spreadsheet was first "calibrated" against alternative estimates based on laboratory tests (compression indices) and a two-dimensional FLAC analysis.

Settlement analysis indicated that settlements of up to 700mm were likely over the nominated design period unless ground improvement was carried out at the proposed berm and coal stockpile site. Considerable costs are incurred when re-levelling of stacker and reclaimer rails is required as a result of excessive settlements. The nominated design period was 17 years, allowing for three re-levelling episodes over a design life of 50 years.

The objective of geotechnical design was to design a ground improvement system to limit post construction settlements over the 17 year period to 200mm beneath the reclaimer berm, and 300mm beneath the coal pad. At the same time, the existing berm and coal Stacker, immediately adjacent to the new stockyard, had to remain in service during the entire construction process without disruption.

3 PRELOAD DESIGN

A number of options for ground improvement were considered to limit post construction settlements. Preloading was assessed to be the most feasible and economic option to meet the performance requirements. Settlement analysis indicated that a total preload height of about 9 m (about 165 kPa applied pressure) would be required to achieve the design objectives over a preload period of less than 12 months. The height of preload was also influenced by the availability of preload material, and the staging of construction works.

Stability analysis indicated that slope failure would occur if the full height of preload was placed in a single stage due to the upper soft to firm clays. A two stage preload system was therefore designed to address both settlement and stability issues during construction. It was estimated that each stage would take between 1 month and 6 months to complete 90% primary consolidation, depending on the thickness of the upper clay. The design included a 10 m wide bench ('stability berm') between the lower and upper stage of preload to address stability issues.

The 4 m high Stage 1 preload was initially placed and monitored to achieve sufficient strength gain in the upper clays to allow the placement of Stage 2 (an additional 5 m) with a low risk of slope failure. As the preload was a temporary slope, the design factor of safety generally adopted was 1.2, although one section was designed at about 1.1 in order to avoid widening the stability berm and increasing the amount of preload material required (a risk accepted by the client and controlled by close monitoring).

The requirements for a two stage preload increased the need for accurate and timely monitoring of the preload progress, in order to ensure that the construction program remained on schedule. A comprehensive array of geotechnical instruments was installed following initial levelling of the site and prior to placing the first stage of preload. The types and numbers of instruments installed are shown in Table 2.

Table 2: Geotechnical Instruments used to Monitor Preload

Type	**Number Installed**	**Purpose**
Settlement Monitoring Plate	45	Measure total settlement of ground surface beneath the preload
Inclinometer	34	Detect lateral movement in soil layers at the edges of the preload
Piezometer	44	Record excess pore water pressure in clay strata to indicate degree of consolidation
Extensometer	1	Measure settlement at several depths to indicate individual soil layer settlements
Earth Pressure Cell	4	Measure total load applied by preload and confirm average unit weight of preload material

A diagram representing the preload process from the initial ground conditions to the operating coal stockyard, is shown in Figure 1. The instrument layout is schematic only, and they were actually installed on a regular grid

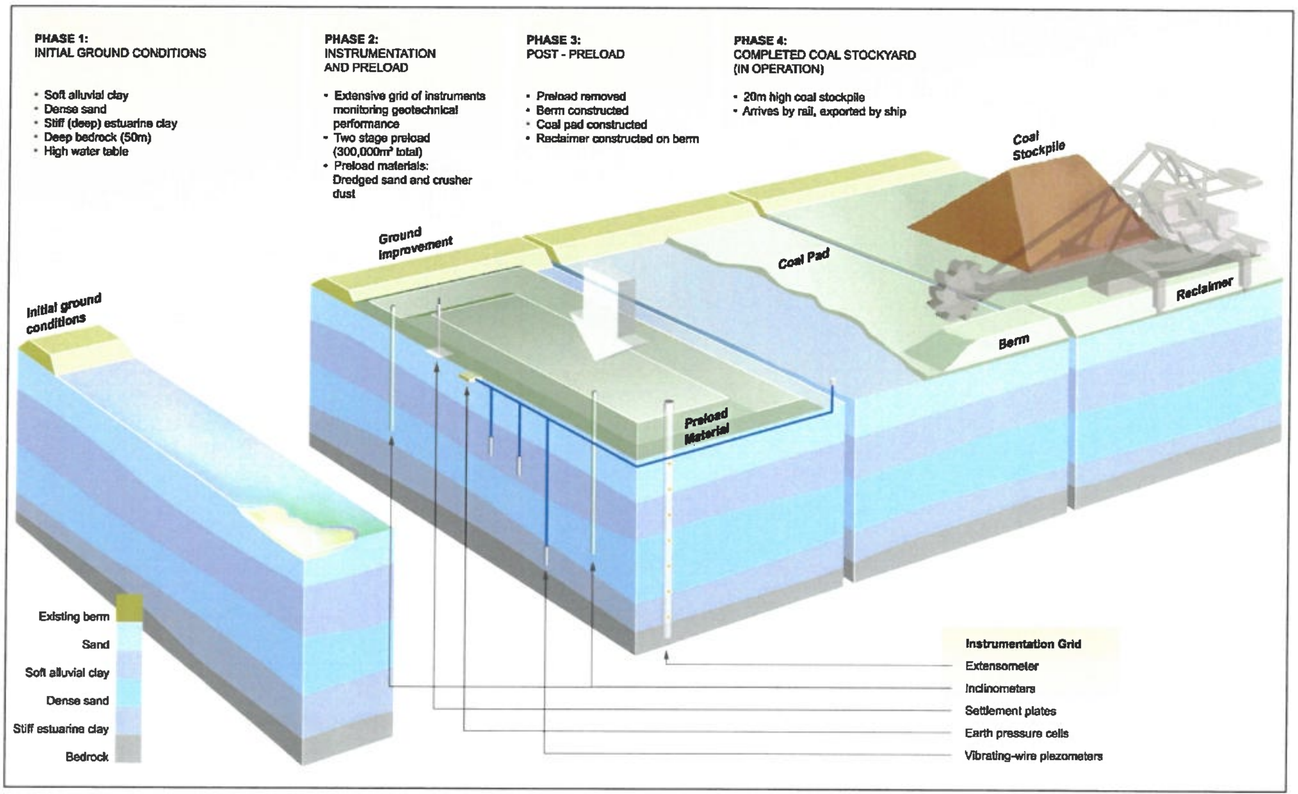

Figure 1: Diagrammatic Illustration of Preload Process

4 MONITORING RESULTS

4.1 GENERAL

The preload monitoring program comprised weekly readings of all instruments for the initial phase of consolidation, reducing to fortnightly in the areas of thicker clay (where consolidation would take a number of months). When the monitoring indicated that primary consolidation of the clay was substantially complete beneath the Stage 1 preload, a further series of CPTs were carried out to assess the strength gain in the clays. Following confirmation of the strength gain, Stage 2 preload was added (making a total preload height of 9 m).

The above process was repeated for Stage 2 preload, including CPTs to confirm the final strength gain. The preload material could then be removed and the Reclaimer berm constructed. The preload was actually carried out in sections of about 300 m to 400 m length in order to re-use preload material and minimise the excess material at completion. Berm construction proceeded in the completed sections while the next section was under preload. The 2000 t Reclaimer machine was then constructed on the first completed section of berm.

Descriptions each type of monitoring instrument and examples of the results obtained are presented in the following sections.

4.2 SETTLEMENT MONITORING PLATES

The settlement monitoring plates (SMP) comprised a 500mm square steel base plate with 32mm diameter steel risers, added in sections as the height of preload was increased. The risers were encased within a protective PVC pipe.

Survey levels were taken by the project surveyors on the base plates prior to any preload being placed, and then on the top of the steel risers at regular time intervals during and following placement of the preload. Great care was required on the part of the earthmoving contractor to avoid disturbing the SMP risers, and a low attrition rate was achieved (inevitably however some risers were damaged). A typical SMP plot is presented in Figure 2.

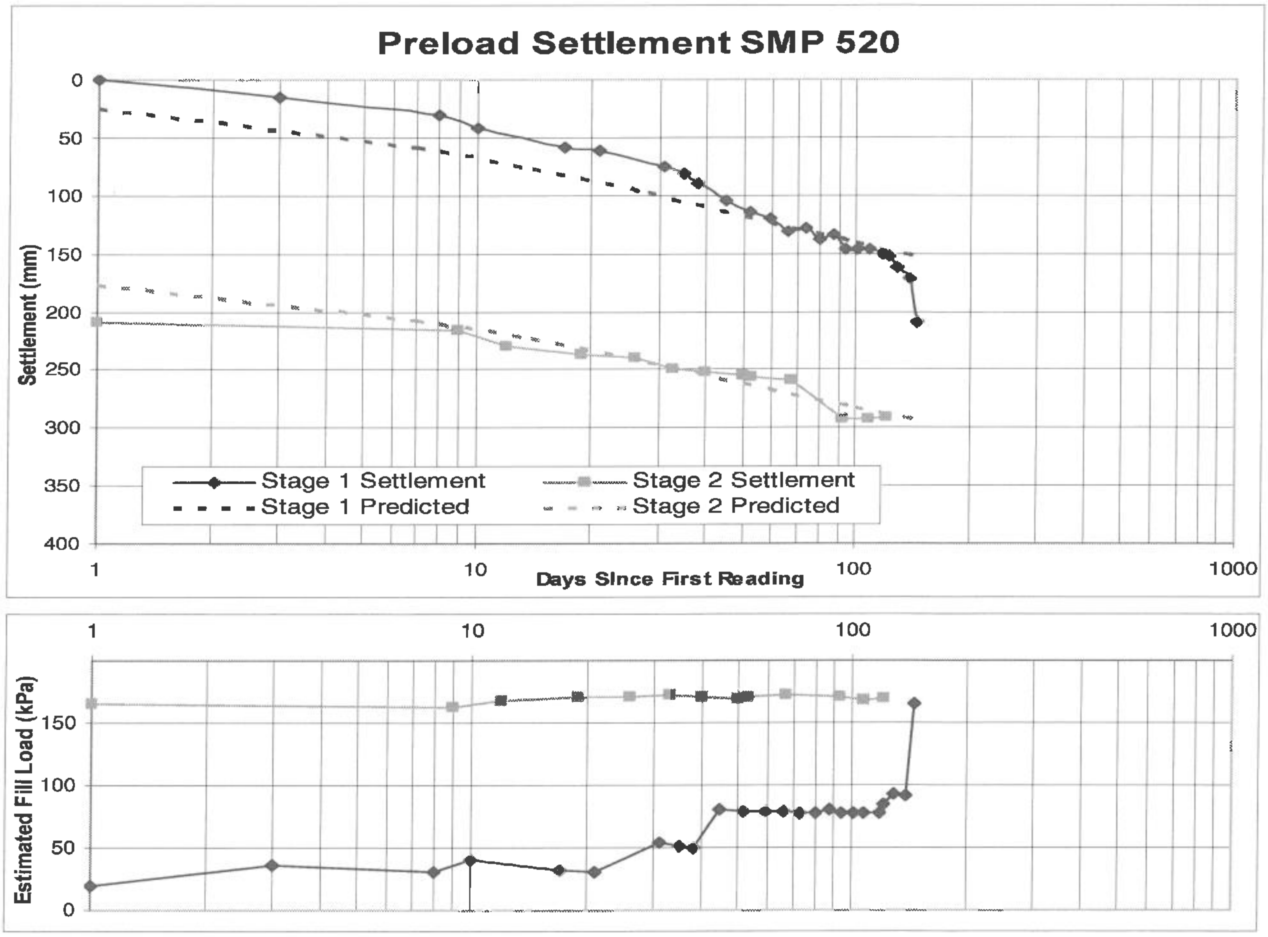

Figure 2: Typical Settlement Monitoring Plate Record

The predicted time-settlement behaviour (shown dotted in Figure 2) assumed that the load was applied quickly, however in practice it took time to build the preload up to design levels. The lower plot shows the estimated fill load for each stage of loading, based on the recorded fill height and fill unit weight. Most SMP installations recorded an acceptable agreement between predicted and measured performance, in terms of both magnitude and rate of settlement.

The 'jump' in the plots at the end of Stage 1 represents the progression to Stage 2, which is then plotted as a new phase of consolidation. It is further noted that very few monitoring locations exhibited a classical one-dimensional shape of the consolidation curve. This was mainly due to the fact that the upper clay layer was generally composed of two or three layers separated by sandy layers, and the deeper stiff clay layer also contributed to settlement; these layers consolidated at differing rates. Along the edges of the preload there was also some plastic deformation in the form of lateral squeezing (see discussion on inclinometers in Section 4.3).

4.2 PIEZOMETERS

The vibrating wire piezometers were installed beneath the preload within the upper soft to firm clay layer to monitor pore water pressures. One piezometer was also installed within the deep stiff clay layer. Their purpose was to monitor the dissipation of excess pore pressure, and augment the SMP data to assist in determining when preload could be removed. The piezometers used had a rated pressure of 350 kPa and a maximum pressure of 525 kPa.

A typical piezometer plot is presented in Figure 3 and the deep clay piezometer is shown in Figure 4. The piezometer plot shows the recorded pore pressure and estimated hydrostatic pressure, with the difference being the excess pore pressure, plotted against root-time. In this particular case, the second stage of preload was added in two separate load increments following the initial dissipation of pore pressure, in order to reduce the risk of instability.

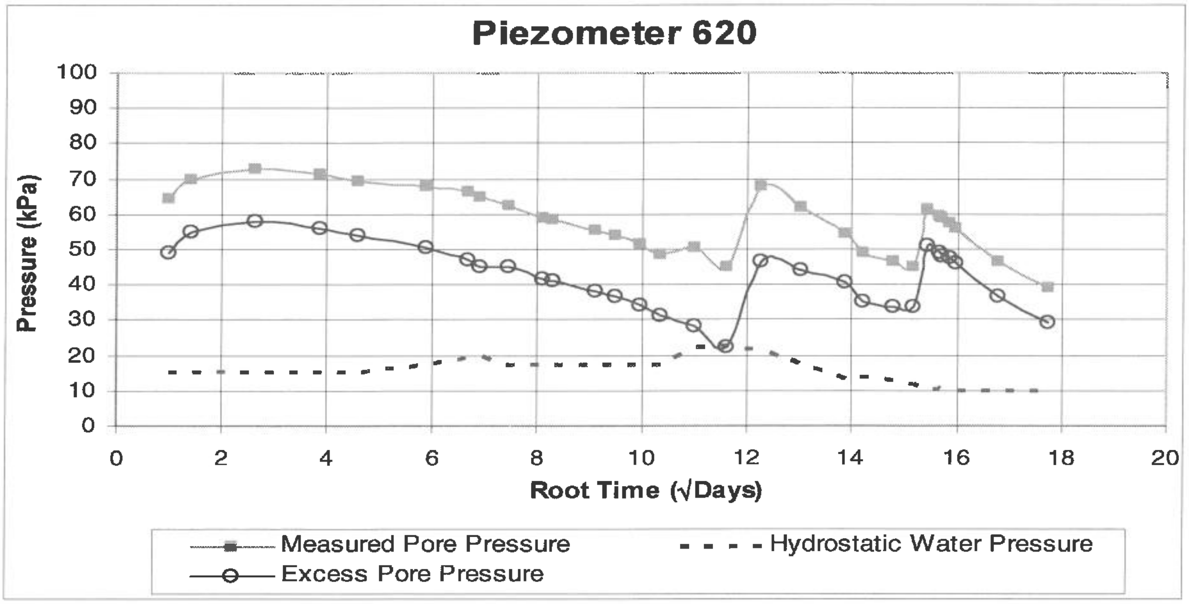

Figure 3: Typical Piezometer Plot – Upper Soft Clay

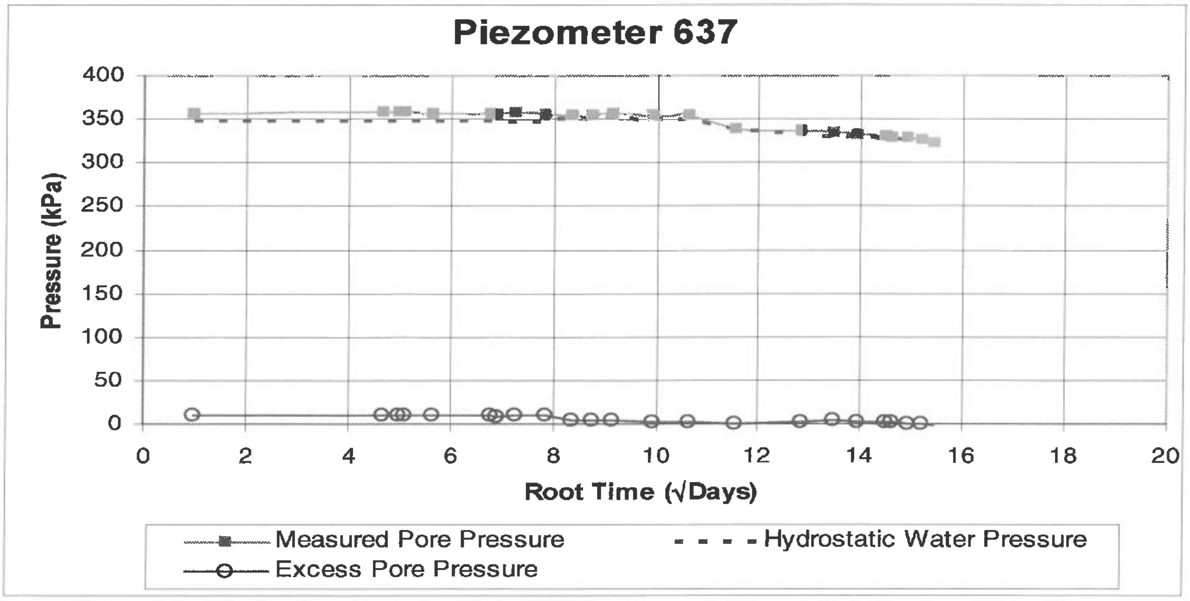

Figure 4: Piezometer Plot – Deep Stiff Clay at 35 m depth

The deep clay piezometer indicated a minor response to the preload (about 10 kPa), which was probably due to the bridging effect of 25 m of dense to very dense sand which overlies the deep clay. This is consistent with the amount of settlement recorded by the nearby extensometer for the deep clay layer (see Section 4.4).

The piezometers were the least successful of all the instruments. Despite taking great care to seal the piezometer within the clay stratum, a significant proportion did not fully respond to the applied load. They were nevertheless a useful adjunct to the other instrumentation, particularly in the critical area where the preload geometry was based on low factors of safety.

4.3 INCLINOMETERS

The inclinometers comprised 70 mm diameter ABS plastic casing, with the internal grooves oriented in line with and orthogonal to the expected direction of movement. They were read using an inclinometer probe. The inclinometers were installed to a depth of at least 1m into medium dense sand, below the upper soft clay strata, adjacent to the toe of the preload, to monitor lateral displacements during the placement of preload. Some were also installed along the interface between the preload and the existing Stacker berm (which had to remain in operation during the construction of the new stockyard), to check for any influence on the existing facilities.

Readings were taken at regular time intervals, as shown on a typical plot in Figure 5. Displacements in the orthogonal direction (i.e. along the slope) were also taken, but are not shown in Figure 5. As would be expected only very minor movements were recorded along the slope.

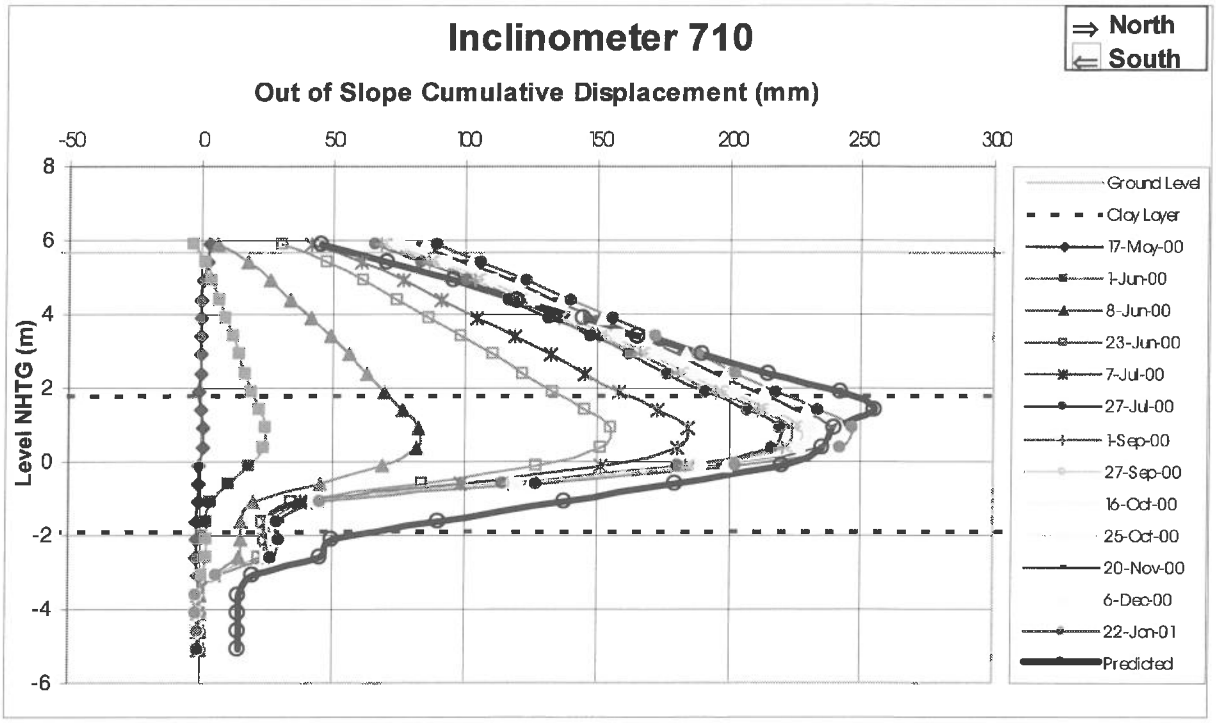

Figure 5: Typical Inclinometer Plot

Figure 5 also shows the ground level and location of the upper soft clay layer, as well as the predicted deflection, as determined by a FLAC analysis prior to commencing preloading operations. The total lateral deformation and rate of deformation was used together with the settlement plate and piezometer monitoring results to confirm the stability of the preload embankment and determine when the Stage 2 preload could be placed.

Not all inclinometer records matched the predicted lateral movement: most deformed less than the predicted amount, while a few moved more than predicted. Those which exceeded predictions were associated with the section of preload where the factor of safety was rather low. Lateral deflections in the range 450 mm to 600 mm were recorded in four inclinometers in this area. An example of this is Inclinometer 711 which is shown in Figure 6.

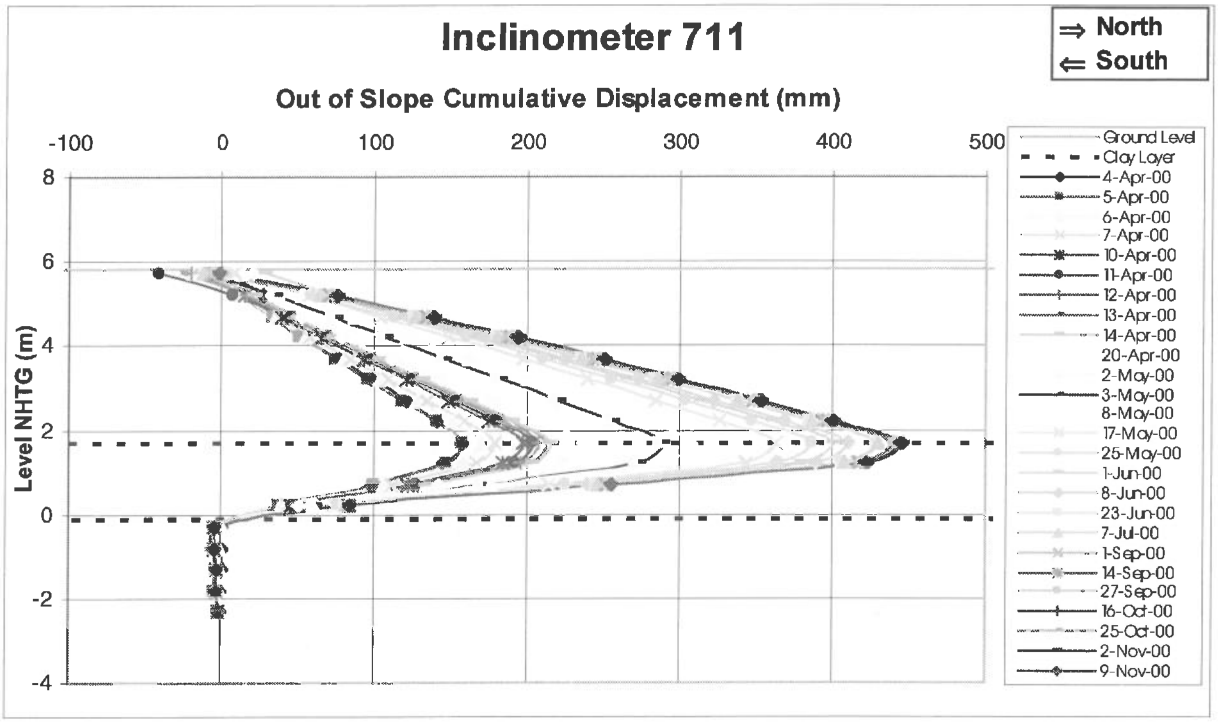

Figure 6: Lateral Deflection in Area of Low Stability

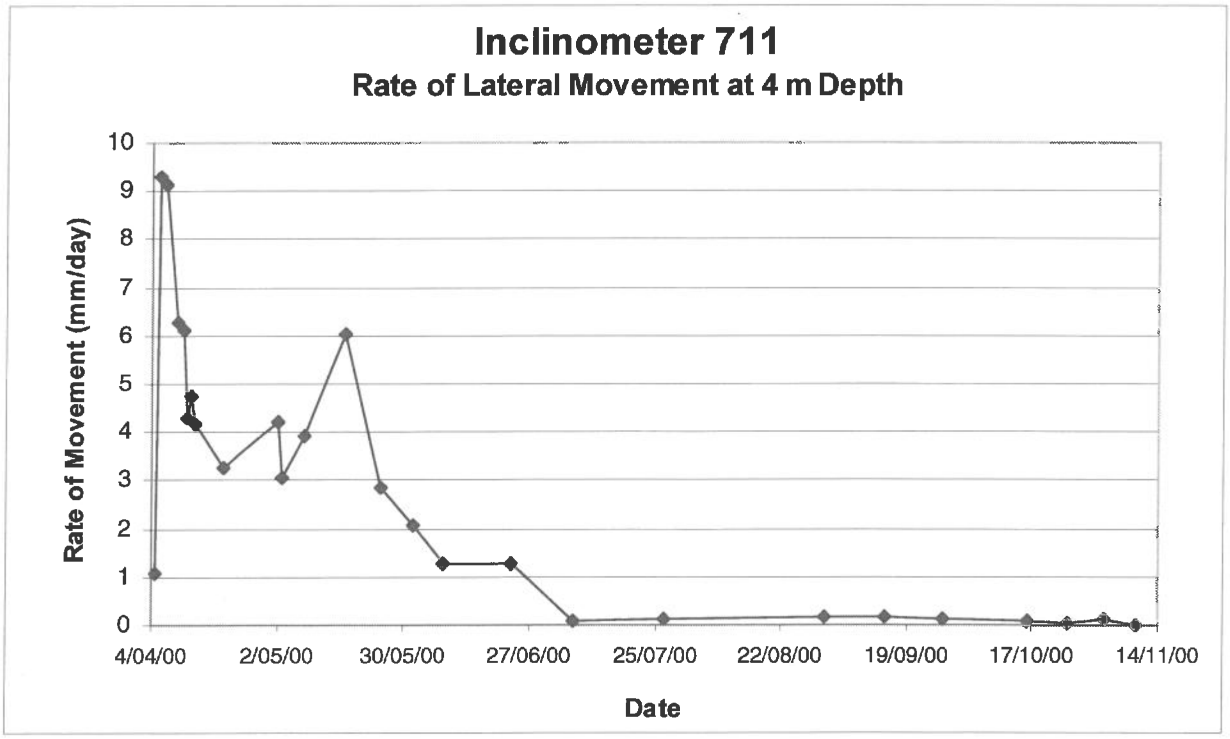

Figure 7: Rate of Lateral Movement Recorded at Inclinometer 711

The results depicted in Figures 6 & 7 indicate significant plastic deformation (squeezing) of the clay layer occurred in the area which had a low factor of safety against failure, however a failure mechanism did not fully develop for the whole profile and the preload remained intact. Failure was avoided by delaying, and then staging, the application of the Stage 2 preload. Eventually there was sufficient strength gain in the clay to accommodate the full preload.

4.4 EXTENSOMETER

An extensometer was installed to bedrock (47 m depth) at one location beneath the preload to measure the contribution of settlement in each layer to the total settlement, and in particular the deep stiff clay layer. The extensometer included 13 target magnets at various depths and a datum magnet set in the bedrock at the base. The measurements were taken over a seven month period before the extensometer had to be discontinued to make way for construction. The time-settlement plots for each magnet are too voluminous to present here, however a summary of the final settlement of each magnet is present in Table 3.

Table 3: Final Extensometer Settlements

Magnet	Stratum	Initial Level (NHTG)	Initial Depth (m)	Total Settlement (mm)
13	FILL – sand	5.551	0.883	148
12	SILTY CLAY – near top	2.583	3.581	138
11	SILTY CLAY – mid level	1.795	4.639	83
10	SILTY CLAY – near base	0.749	5.685	65
9	SAND – near top	-0.270	6.704	53
8	SAND	-9.203	15.637	39
7	SAND	-18.214	24.648	31
6	SAND	-26.115	32.549	31
5	CLAY – near top	-29.595	36.029	25
4	CLAY – upper mid level	-31.923	39.357	16
3	CLAY – mid level	-34.824	41.258	12
2	CLAY – lower mid level	-35.885	42.319	11
1	CLAY – near base	-38.290	44.724	4
Datum	SILTSTONE	-40.676	47.110	0

The results indicated that the majority of settlement occurred in the upper soft clay layer as expected, with 25 mm recoded for the deep clay layer which was about 13 m thick. Unfortunately the second stage of preload had only been in place six weeks when the monitoring was discontinued.

4.5 EARTH PRESSURE CELLS

A total of four earth pressure cells were installed beneath the preload to confirm the magnitude of the load applied by the preload. The pressure cells were vibrating wire type with a maximum pressure of 300 kPa. Two types of fill material were used as preload: dredged sand (unit weight ≈ 18 kN/m^3), and Crusher Dust (unit weight ≈ 20 kN/m^3). The type of fill and the corresponding density were taken into consideration when determining the estimated fill load for settlement, piezometer and extensometer monitoring.

Figure 8 shows the results obtained from one of the earth pressure cells. The initial readings, as the preload was being built up, were relatively low and the back-figured unit weight was inaccurate. However, once the fill load exceeded about 100 kPa, the accuracy improved and appeared to provide a reliable estimate of the fill unit weight.

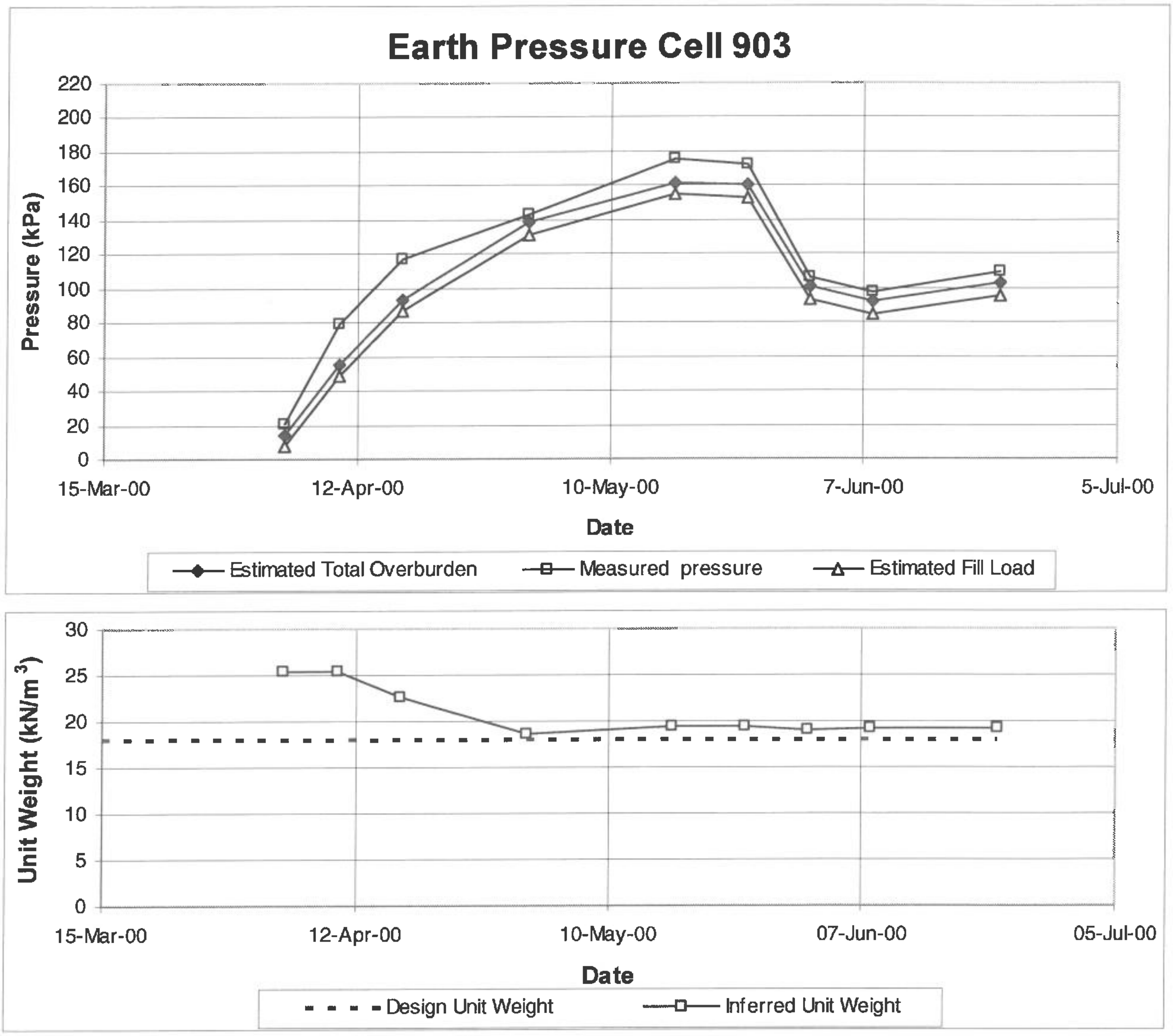

Figure 8 Earth Pressure Cell Record and Back-figured Unit Weight

The back-figured unit weight averaged 19 kN/m^3, which was satisfactory as preload design had been based on 18 kN/^{3} for the sand fill.

4.6 STRENGTH GAIN ASSESSMENT

A total of 90 CPT's were carried out at the end of Stage 1 and Stage 2 preload. A sufficient gain in strength of the upper clay was necessary to ensure adequate bearing capacity, slope stability and confirm that long term settlement predictions would be valid.

Prior to preloading, the average shear strength of the upper soft clay strata was measured at about 15 kPa. Early in the project, the report on the proposed preloading predicted that the shear strength of the clay would increase to an average 30 kPa after Stage 1 preload, and to an average of 50 kPa after Stage 2 preload.

Figure 9 below shows the results of three CPTs carried out at close to the same location before preload (i.e. initial investigation), after Stage 1 and after Stage 2. The profile of undrained shear strength has been calculated and plotted against reduced level. These plots show a 1.5 m thick clay layer underlain by three thinner layers. The shear strength before preloading ranges from 10 kPa to 25 kPa. After Stage 1 preload the clay strength has increased to an average of about 35 kPa, and after Stage 2 preload the clay strength has increased to an average of about 55 kPa. A distinct thinning of the layers is also apparent from the plots.

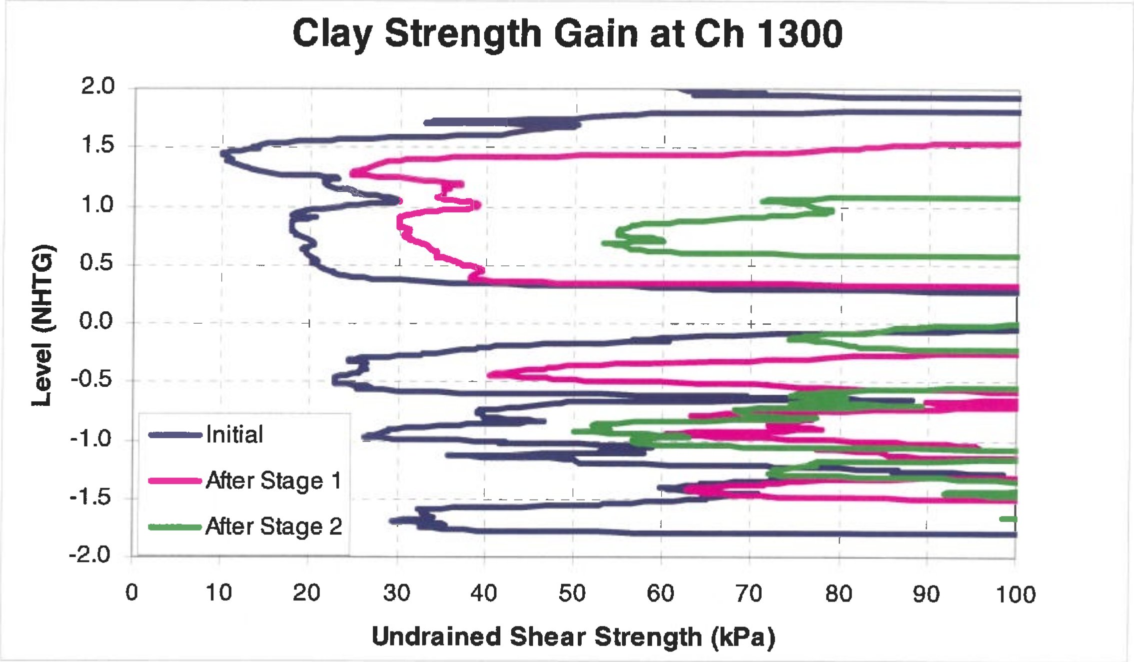

Figure 9: Strength Gain in Upper Clay from Preloading (Ch 1300, Reclaimer Berm)

5 CONCLUSIONS

The monitoring results indicated that the preload generally performed as predicted, with target settlement and strength criteria being met. Construction was completed without disruption to the existing coal handling infrastructure, and the facility has now been operating to expectations for about one and a half years. The success of the project has been attributed to the accurate and timely supply and analysis of monitoring results, together with good communication and liaison with all parties involved.

6 ACKNOWLEDGEMENTS

The author acknowledges the assistance and co-operation from Port Waratah Coal Services Limited (PWCS), and Bechtel Australia Pty Ltd. Several other members of the Douglas Partners team made significant contributions to the project, in particular Peter Will Wright, Chris Bozinovski and John Niland.

7 REFERENCES

Douglas Partners Pty Ltd, 2000, 'Proposed Coal Stockyard, PWCS Stage 3 Expansion, Kooragang Coal Loader', *Unpublished report, Project 31100A, April 2000.*

Douglas Partners Pty Ltd, 2001, 'Geotechnical Instrumentation Monitoring, PWCS Stage 3 Expansion, Kooragang Coal Loader', *Unpublished report, Project 31100B, June 2001.*

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Slope Indicator Company

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THE APPLICATION AND USE OF IN-PLACE INCLINOMETERS

Colin Viska
Slope Indicator Australia

ABSTRACT

Whilst the traversing Inclinometer has been in use since 1958, with improved electronics, dataloggers and computers the use of In-place Inclinometers has become more prevalent in the construction industry and similarly by other users of Geotechnical Instrumentation.

Inclinometers are used to monitor subsurface movements of earth in landslide areas and deep excavations. They are also used to monitor deformations in structures such as dams and embankments.

This paper will discuss the types of In Place Inclinometers, their performance and their application in the various forms of construction Geotechnical Engineers are concerned with.

Methods of data acquisition and applications of software will also be discussed.

1 INTRODUCTION

A rudimentary inclinometer system includes inclinometer casing, an inclinometer probe and control cable, and a inclinometer readout unit.

Inclinometer casing is typically installed in a near-vertical borehole that passes through a zone of suspected movement. The bottom of the casing is anchored in stable ground and serves as a reference.

The inclinometer probe is then used to survey the casing and establish its profile (or top is surveyed if the former is not feasible). Ground movement causes the casing to move from its initial position to a new position. The rate, depth, and magnitude of this displacement is calculated by comparing data from the initial survey to data from subsequent surveys.

1.1 PARTS OF THE PROBE

The inclinometer probe consists of a stainless steel body, a connector for control cable, and two pivoting wheel assemblies. The distance between the wheel pivots is the gauge length.

When properly connected to the control cable, the probe is waterproof and has been used deeper than 300 metres.

Each of the two wheel assembly consist of a yoke and two wheels. The upper wheel of each assembly is referred to as the “reference wheel”.

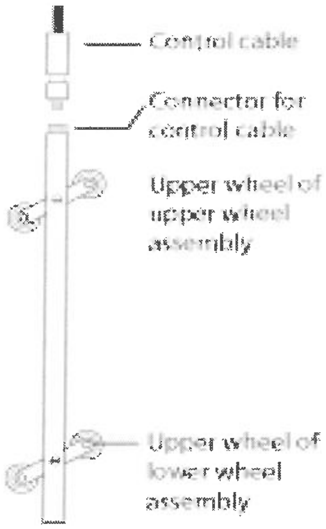


Figure 1 – Parts of the Probe

1.2 MEASUREMENT PLANES

The inclinometer probe employs two force-balanced servo-accelerometers to measure tilt. One accelerometer measures tilt in the plane of the inclinometer wheels. This is the "A"axis. The other accelerometer measures tilt in a plane perpendicular to the wheels. This is the "B" axis. Figure 2 shows the probe from the top. When the probe is tilted toward the A0 or B0 direction, readings are positive. When the probe is tilted in the A180 or B180 directions, readings are negative.

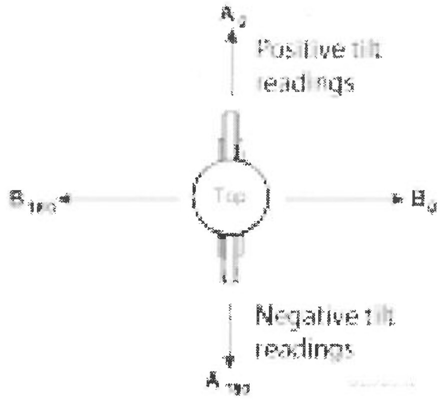


Figure 2 – Measurement Planes

1.2.1 A0 Casing Groove and Reference Wheels of Probe

Inclinometer casing is installed so that one set of grooves is aligned with the expected direction of movement. One groove, typically the "downhill" groove, should be marked A0. In a standard inclinometer survey, the probe is drawn from the bottom to the top of the casing two times. In the first pass, the reference (upper) wheels of the probe should be inserted into the A0 groove. This ensures that movements in the direction of the A0 groove are positive values.

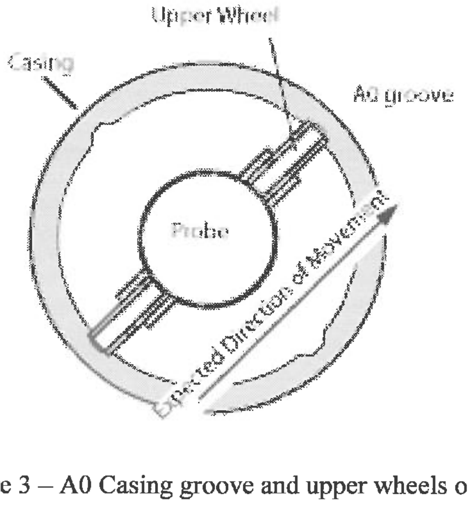


Figure 3 – A0 Casing groove and upper wheels of probe

1.3 CUMULATIVE DEVIATION

By summing and plotting the deviation values obtained at each measurement interval, we can see the profile of the casing.

The black squares at each measurement interval represent cumulative deviation values that would be plotted to show the profile of the casing.

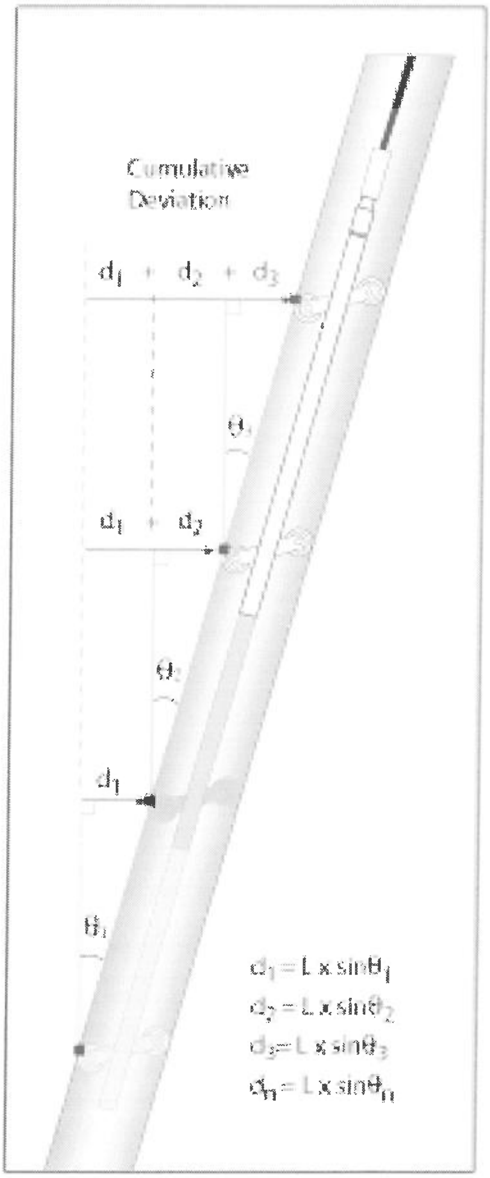
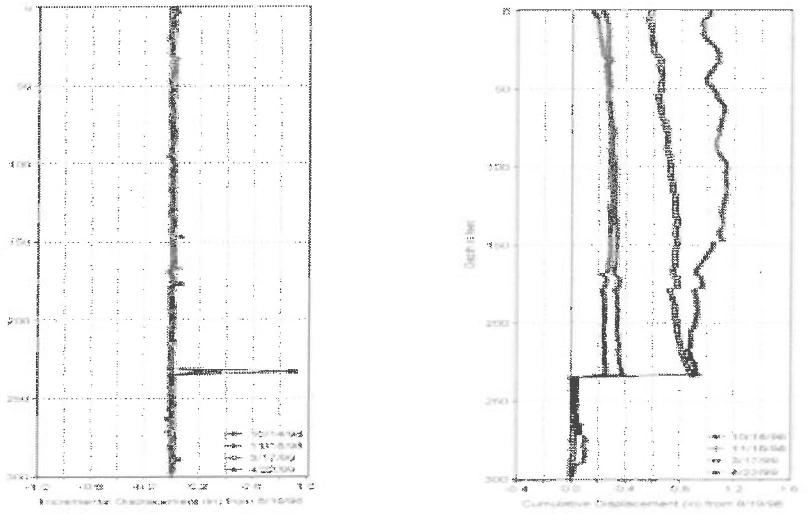


Figure 4 – Cumulative Deviation

1.4 DISPLACEMENTS

Changes in deviation are called displacements, since the change indicates that the casing has moved away from its original position. When displacements are summed and plotted, the result is a high resolution representation of movement.

Displacement is a change in deviation and shows that the casing has moved away from its original position. The figures below show plots of a borehole with a shear zone at 230 feet.



Incremental displacement plot shows movement at each measurement interval

Cumulative displacement plot shows a displacement profile. Displacements are summed from bottom to top

Figure 5 – Displacements

1.5 MANUAL REDUCTION OF DATA

Normally, computer software is used to reduce inclinometer data. Prior to 1985, all calculations were performed manually. The method of manual reduction of data is in the Slope Indicator Digitilt Inclinometer Probe Manual.

INITIAL PROFILE OF BOREHOLE CASING

When using Inclinometers it is imperative that base line readings are taken prior to any commencement of construction or excavations. This will allow any changes in the initial profile to be easily identified.

1.6 THE ADVANTAGES AND DISADVANTAGES OF TRAVERSING INCLINOMETERS

The advantages of the traversing (portable) Inclinometer are that:

- one system can be used in many boreholes
- the direction of the movement and the magnitude of the movement can be calculated
- small O.D. of probe that allows it to be used effectively in casing that moves and is deformed
- Maximum performance, ie: accuracy and precision

The disadvantages are that it is:

- labour intensive, and
- requires processing of the data by manual intervention before a comparison can be made (not real time)

To address the disadvantages, in the past six years low cost In Place Inclinometers have been designed, built and used in various applications throughout the world.

Prior to this In Place Inclinometers with servo-accelerometers were used for automatic monitoring, however market pressure has made this approach cost prohibitive.

2 STANDARD IN-PLACE INCLINOMETERS

2.1 APPLICATIONS

The In-Place Inclinometer is ideal for data logging and near-real time monitoring. Typical applications include:

- Monitoring deformation of diaphragm walls supporting deep excavations.
- Monitoring ground movements induced by tunnel construction.
- Monitoring deformations of embankments and retaining walls.
- Monitoring landslide areas above dams, highways, and railroads to provide early warning of slope failure.

2.2 OPERATION

The In-Place Inclinometer system consists of inclinometer casing and a string of inclinometer sensors. The Inclinometer casing is installed in a vertical borehole that passes through a suspected zone of movement. This zone would normally be identified by surveys with the traversing probe. The sensors, each connected with pivoting joints, are positioned inside the casing to span the zone of movement. When ground movement occurs, casing is displaced, causing a change in the tilt of the sensors inside.

The sensors measure tilt, the angle of inclination from vertical. The tilt measurement is converted to lateral deviation using the formula L sine Ø, where L is the gauge length of the sensor and Ø is the tilt angle.

Gauge lengths can be any length up to 3m. They are typically 1, 2 or 3 metres.

2.3 ADVANTAGES

2.3.1 Real Time Monitoring

The In-Place Inclinometer is ideal for continuous, unattended monitoring and can deliver readings in near-real time.

2.3.2 Rigid Gauge Tubing

Accurate displacement calculations require straight-line sensor gauge lengths. Rigid gauge tubing satisfies this requirement. Rigid gauge tubing also provides reliable performance in soft ground, where abrupt changes in profile may occur at a shear zone.

2.3.3 Dataloggers

Data loggers and post-processing software can present profile plots and trend plots of inclinometer data just minutes after the readings are obtained.

2.3.4 Removable

Wheeled sensors can be easily removed to allow verification checks with a traversing probe.

2.4 DISADVANTAGES

- More sensors the greater the cost
- Diameter of sensor and cables passing the sensor
- Components in Data logger (expense of)

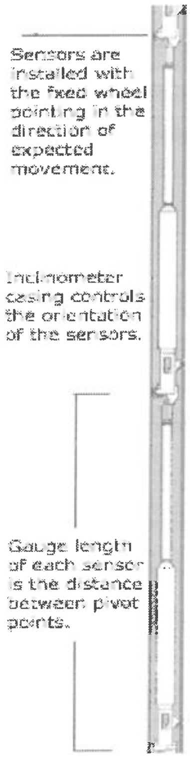


Figure 6 – In-Place Inclinometers (Fixed wheel is “reference” wheel, analogous to upper wheel)

3 MULTIPLEXED IN-PLACE INCLINOMETERS

3.1 DESCRIPTION

Standard In-Place Inclinometers have a cable running or attached to each sensor. As the number of sensors in the borehole increases, the internal diameter of the borehole can get “crowded”.

To overcome this crowding there is a version of the IPI called the Multiplexed (Muxed) IPI. This enables the string of sensors to be connected by the same signal-cable segmented by means of internal multiplexing.

3.2 ADVANTAGES

- Reduces amount of cable required
- Reduces the components required at the Datalogger
- Allows more movement in the borehole before interference (“crowding”)

3.3 DISADVANTAGES

- The sensor is more expensive
- Series connection affects reliability

4 SLED SENSORS

4.1 DESCRIPTION

Sled Sensors are intended for a low cost application not requiring maximum performance. Standard IPI & Muxed sensors use wheels that will push against the grooves of the inclinometer casing and can be easily retrieved allowing redeployment.

EL Sled Sensors use a sled that fits neatly into the groove of the casing. It is lower technology and uses a less accurate sensor.

4.2 ADVANTAGES

- Lower cost

4.3 DISADVANTAGES

- Intended for one time installation
- Only uniaxial
- Each sensor needs its own cable
- Uses less sensitive sensor (lower resolution)

4.4 DATALOGGING

Although In-Place Inclinometers can be read manually, their use with a multi-channel Datalogger makes them ideal for continuous, unattended monitoring and can deliver readings in near real time.

The datalogger can then be interrogated via modems and telephones for viewing data on the owners, contractors and/or consultants computer.

4.5 PRESENTATION SOFTWARE (POST PROCESSING)

Having collected the information from IPI's will generate a huge amount of data that will require processing. There are software packages available that will present this data in graphical form. This software also allows thresholds to be set and alarms triggered when limits are exceeded.

4.6 CURRENT INSTALLATIONS OF IN-PLACE INCLINOMETERS IN AUSTRALIA

- Hope Valley Reservoir, S.A.
- Prospect Dam, NSW
- Happy Valley Dam, S.A
- University of Wollongong, NSW
- Parramatta Rail Link NSW

5 ACKNOWLEDEMENTS

Special thanks to Randolph Lohman, Rick Monroe and Fiona Rossi who have assisted in the production of this paper.

6 REFERENCES

- Mikkelsen, P. Erik (1986) *Design and Installation of Instrumentation*, 1st Edition, Slope Indicator Co. USA
- Slope Indicator (June 2003) *Resource C.D.*

COMPARISON OF INCLINOMETERS AND IN-PLACE INCLINOMETERS

TABLE 1

INSTRUMENT	PORTABLE INCLINOMETER	EL IPI	MUXED EL IPI	EL SLED IPI
Part Number	50302510	56804121, 56804122	56804521, 56804522	56804410
Orientation	Biaxial	Uniaxial, Biaxial	Uniaxial, Biaxial	Uniaxial
Range from Vertical	+/- 53°	+/- 10°	+/- 10°	+/- 30°
Resolution	0.02mm / 500mm	0.04mm / m	0.04mm / m	0.05mm/m
Repeatability	± 0.003° .01% F.S. at 30°	± 22 arc seconds	± 22 arc seconds	± 72 arc seconds or ± 0.35mm/m
Temperature Rating	-20 to +50° C	-15 to 40°C *	-15 to 40°C *	-15 to 40°C *
Dimensions	25.4 x 653mm	38mm x 380mm	38mm x 380mm	38m x 380mm
Accuracy	± 0.25mm / reading	1mm/1m	1mm/1m	4mm/m
Gauge Length	500mm	1m, 2m, 3m	1m, 2m, 3m	1m, 2m, 3m
Casing O.D. (mm)	48, 70, 85	70, 85	70, 85	70, 85
Materials	Stainless Steel	Stainless Steel	Stainless Steel	Aluminum

*with optional, extended range calibration

Australian Geomechanics Society

Sydney Chapter

A Note on Real Time Monitoring of Tunnel Invert Slabs During Stressing

Dr Philip Pells
Pells Sullivan Meynink
N Street
Connell Wagner

Mini-Symposium
Geotechnical Instrumentation and
Construction Works Compliance Testing

Presented at:

Eagle House, Milsons Point
Sydney, Australia
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A NOTE ON REAL TIME MONITORING OF TUNNEL INVERT SLABS DURING STRESSING

by

P.J.N Pells, Pells Sullivan Meynink Pty Ltd
N. Street, Connell Wagner Pty Ltd

1. INTRODUCTION

The Burnley Tunnel in Melbourne is about 3.5km long and passes at a depth of about 60m beneath the Yarra River. In about mid-1999, prior to the tunnel being opened, problems developed with the invert lining within parts of the tunnel. These problems were widely canvassed in the Media and aspects of the remedial works were published in the Melbourne Age. This article deals with a small technical facet of material which is in the public domain. It does not deal with matters which have been in dispute between various parties associated with the tunnel.

The simple objective of this article is to record the level of accuracy which can be achieved in real time survey monitoring within a tunnel.

2. TUNNEL GEOMETRY

Figure 1 shows a cross-section through the final lining of the tunnel (Refs 1, 2 and 3). The lining is fully tanked by means of a continuous PVC membrane between the Primary Lining (rockbolts, sets and shotcrete) and the un-reinforced concrete Secondary Lining shown in Figure 1.

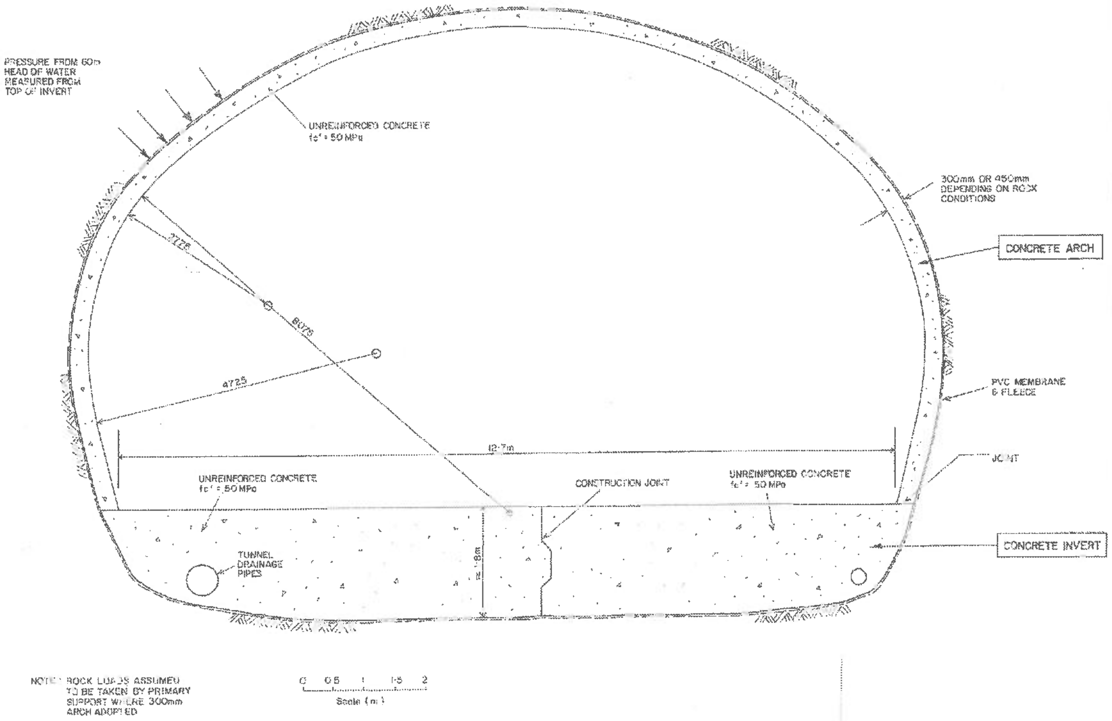


Figure 1 – Cross-section through final lining of Burnley Tunnel

The tunnel invert comprised thick slabs with a slightly arched underside. There is a longitudinal joint near the centreline as shown in Figure 1, and the slabs are 12m long along the tunnel, ie: there are transverse joints at 12m along the invert.

Each 12m long invert slab was given a number. The numbering is shown in Figure 2, which also shows the geological profile along the tunnel (Ref 4).

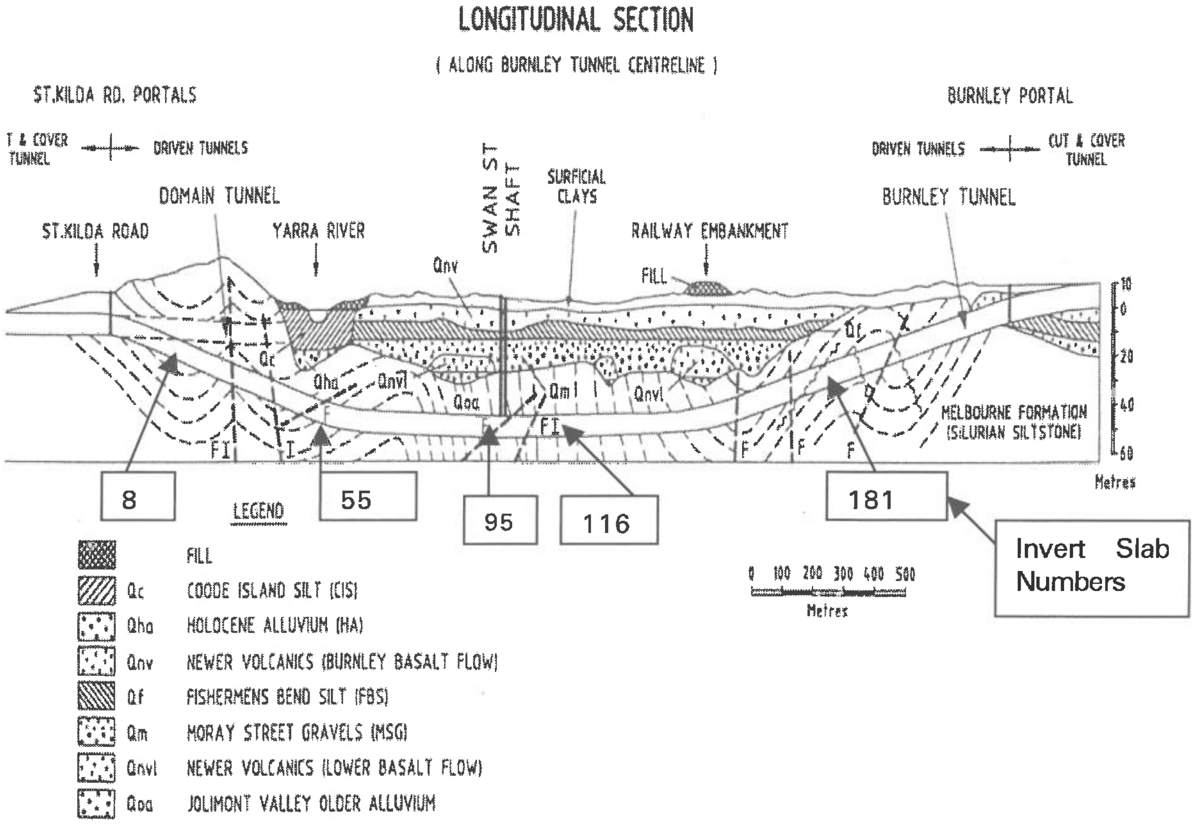


Figure 2: Longitudinal Section

This article deals mainly with invert slabs 95 to 116, ie: the slabs near the Swan Street Shaft where final hydrostatic pressures are of the order of 60m.

3. INVERT REMEDIAL MEASURES

Figure 3 shows a typical section through one of the central invert slabs where the remedial measures comprised:

- trimming about 400mm off the existing invert concrete,
- installation of sufficient anchors to resist the uplift pressures (typically 96, nominal 100 tonne working load anchors per 12m long by 12.7m wide invert slab),
- casting of a new reinforced concrete capping slab, and
- stressing of the anchors.

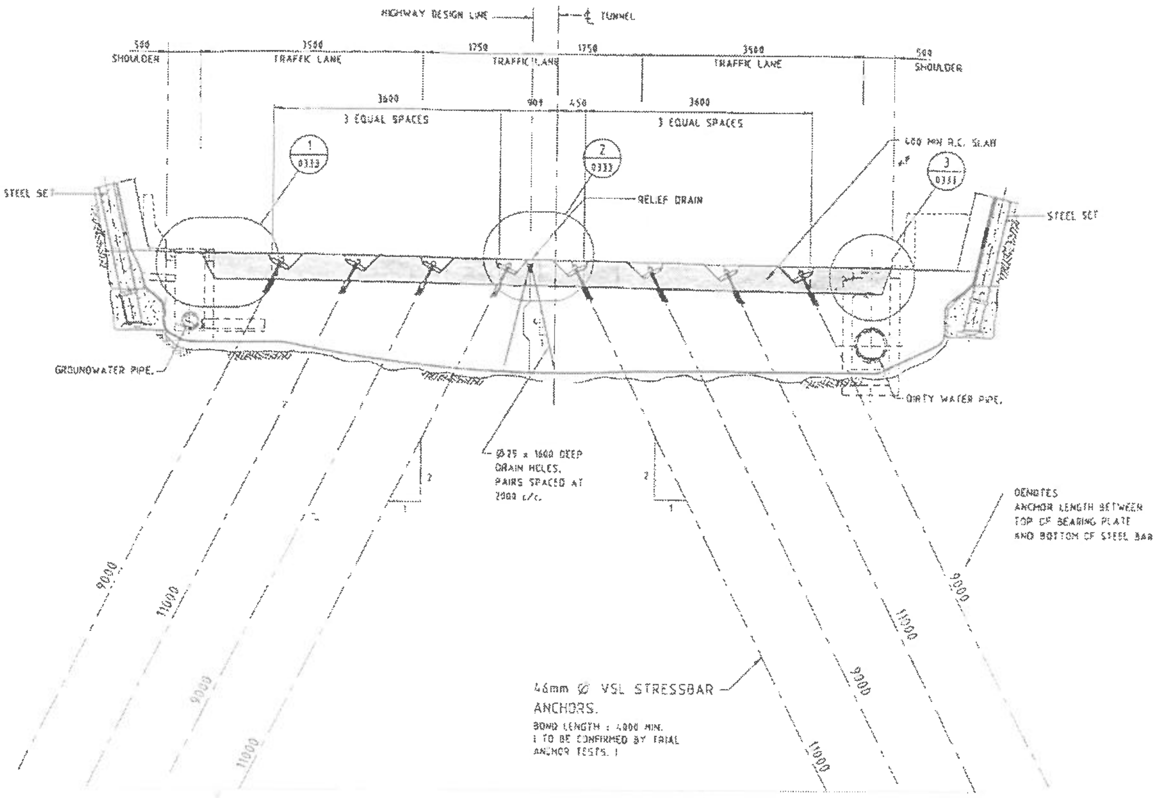


Figure 3: Multi-anchored slabs

Over the remainder of the tunnel, except for slabs 1 to 12 and 177 to 181, remedial work comprise central twin anchors, at average longitudinal spacing of about 1.2, as shown in Figure 4.

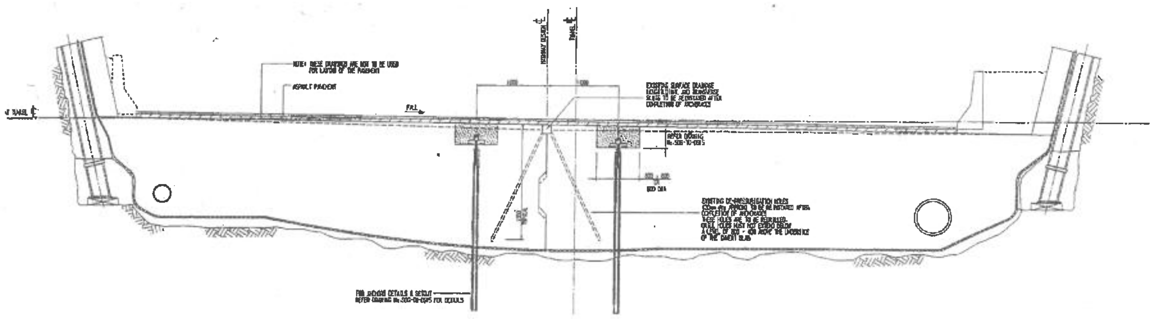
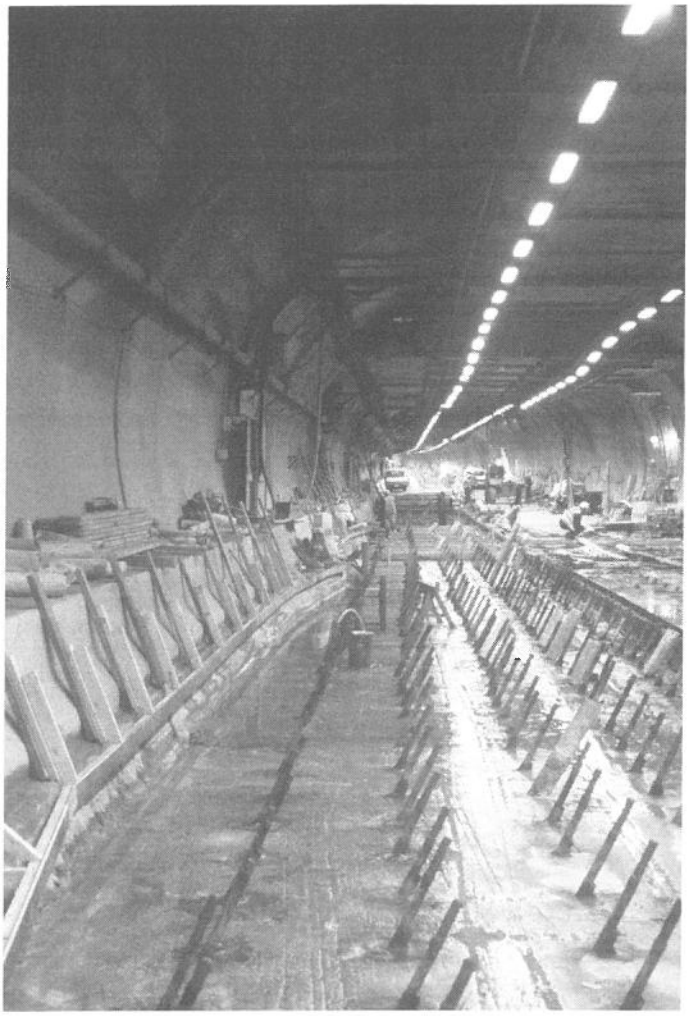


Figure 4: Twin Anchor Design

Photographs 1 and 2 show part of the central section of the tunnel during multi-anchor installation.



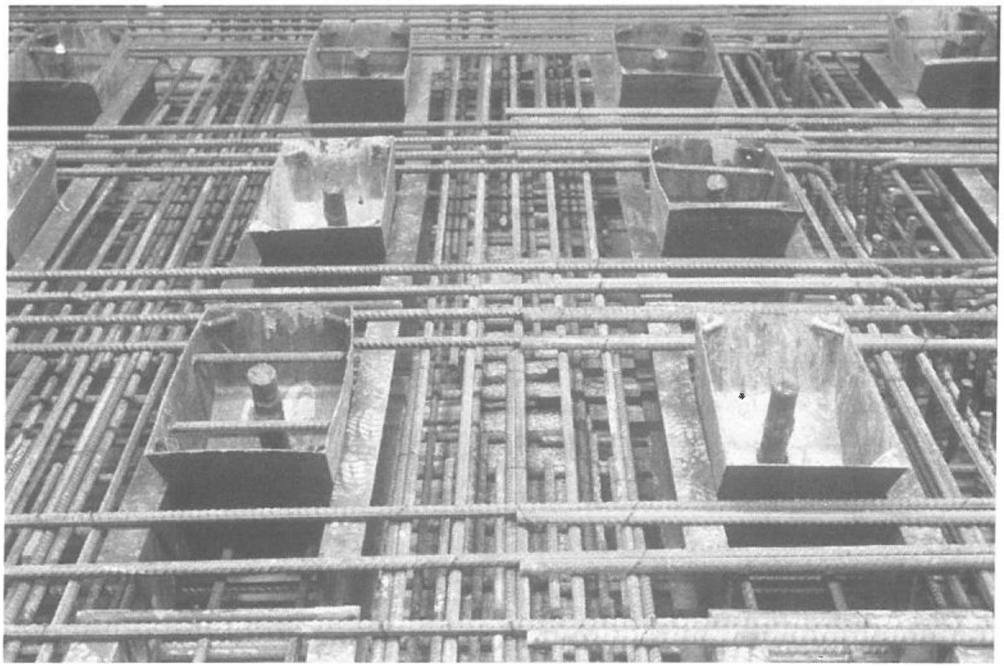
Photograph 1 – Drilling and installation of anchors

Photograph 2 – Anchors installed in one half of the invert

Photograph 3 shows stressing of a multi-anchored slab in the background, while in the foreground can be seen commencement of twin anchor remediation.

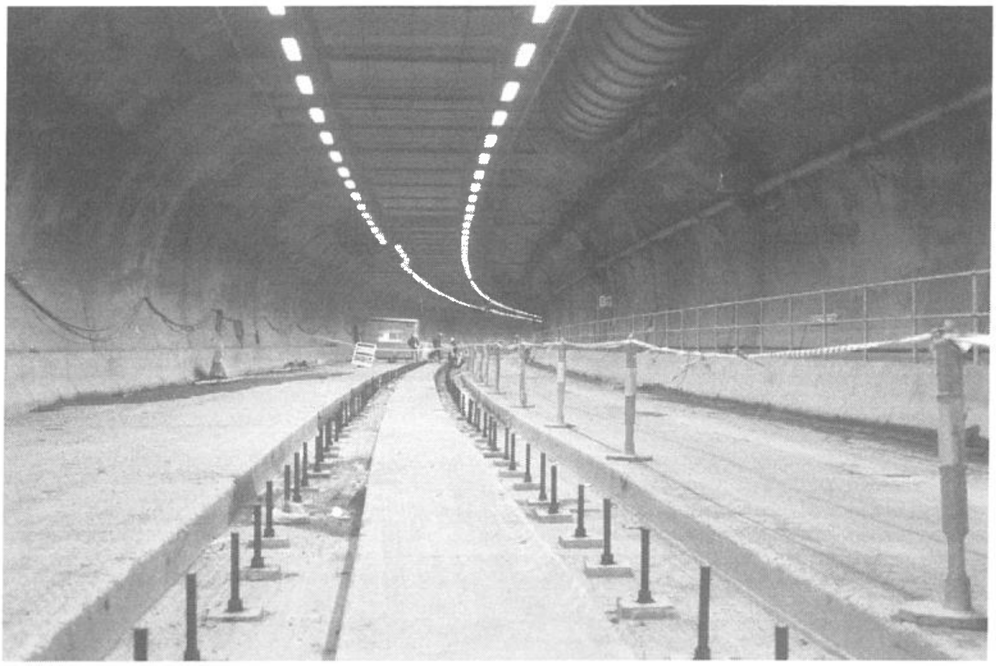
Photograph 3 – Twin anchored and multi-anchored slabs

Photograph 4 shows completed 46mm VSL stress bar anchors within the reinforced capping slab, prior to casting of the slab.



Photograph 4

Photograph 5 shows a typical length of twin-anchor remediated slabs.



Photograph 5

In total there were about 2500 anchors in the multi-anchored slabs and a further 2700 anchors in the twin-anchored slabs.

4. MONITORING OF THE INVERT SLABS DURING STRESSING

As already mentioned, each 12m long central slab was pre-stressed with about 96 anchors, with a combined load of about 10,000 tonne (ie: more than the weight of two Anzac Class Frigates). The key issue was to ensure that the slabs did not settle or twist more than a defined amount during stressing as this could have affected adjacent slabs and/or interaction with the arch lining. The task was to achieve the requisite monitoring accuracy (<2mm in level) and to be able to determine the three dimensional movement of each slab, in real time.

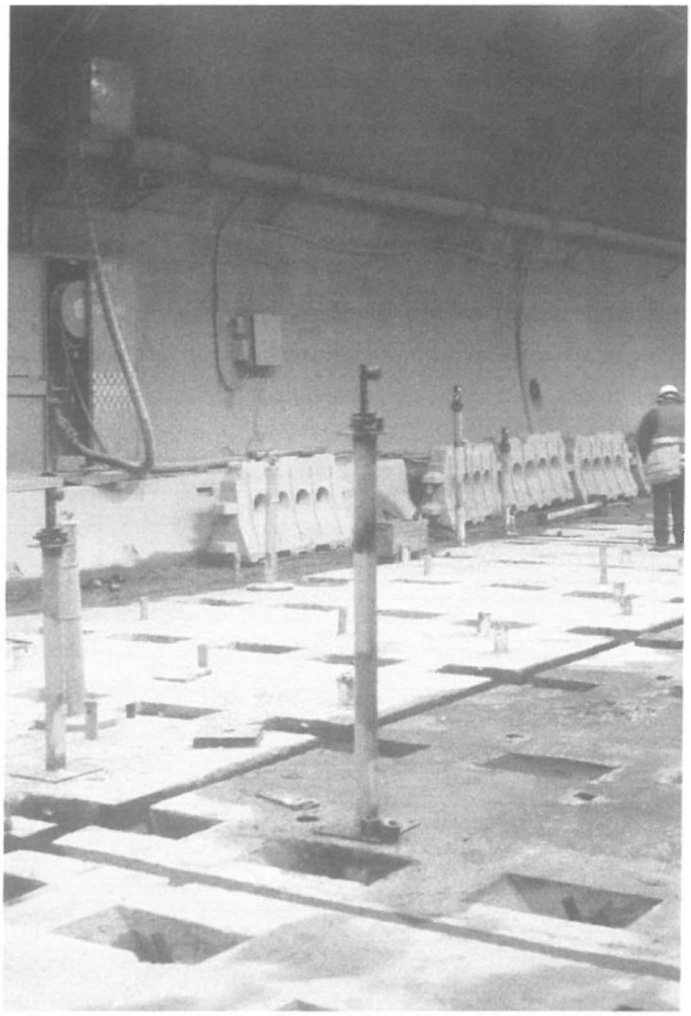
The system which was adopted comprised Geodimeter 6000 robotic theodolites mounted on heavy brackets fixed to the concrete sidewall of the tunnel. This system required the use of active prisms which in effect identify themselves to the theodolite. They also have the advantage of a light which flashes when a "hit" is registered and the visual telltale was helpful to supervisory staff casting an eye over the whole process. It should be noted that the equivalent Leica system does not need active prisms.

Each theodolite could handle about 30 prisms. At the peak of stressing operations there were about 90 prisms, requiring simultaneous operation of three theodolites. The system was supplied and managed by John Gertzel, a Melbourne surveyor.

The prisms were mounted on posts which ranged in height from about 1.2m to 2.0m. There were four prisms for each half of an invert slab (a half slab being defined by the near-central longitudinal joint and the 12m-spaced transverse joints). Photograph 6 shows the typical post arrangement. Survey distances were typically less than 30m.

Readings from the theodolites were processed and assessed in real time and the stressing sequence of the anchors within a half-slab was managed so that tilt and slab settlement was within the prescribed limits. The system consistently achieved accuracy of relative levels of better than or equal to 1mm.

When used to monitor sidewall movements within the tunnel, the accuracies were of the order of 2mm, mainly because the angles between theodolite and targets were typically much more acute than for the invert monitoring.



Photograph 6

REFERENCES:

1. Nelsen A, Porter S and Wilson C (1996). The Melbourne City Link Tunnels; Some Preliminary Design and Construction Concepts. 9th Australian Tunnelling Conference, Sydney, pp31-45.

2. Short M, Weeks N, Woodall R and Porter S (1999). Design of the Melbourne City Link Tunnels. 10th Australian Tunnelling Conference, Melbourne, pp 3-14.

3. Taylor E and Munro G (1999). The Race to Finish – From Design to Completion – 5km of Three Lane Road Tunnel in Three Years. 10th Australian Tunnelling Conference, Melbourne, pp15-20.

4. Hutchison BJ and Lamb I (1999). Construction Management of Hydrogeological Aspects of Melbourne's City Link Project. 10th Australian Tunnelling Conference, Melbourne, pp35-45.

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Settlement Behaviour of Deep Fill For Housing Development, Niddrie, VIC

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SETTLEMENT BEHAVIOUR OF DEEP FILL FOR HOUSING DEVELOPMENT VALLEY LAKE, NIDDRIE, VICTORIA

Douglas Goad
Principal, Golder Associates Pty Ltd Melbourne

1 INTRODUCTION

This paper describes our experience of processing and placing about 2.4 million cubic metres of stockpiled overburden fill into a former basalt quarry in Niddrie, Victoria.

The project encompasses many geotechnical challenges including the development of a strategy to engineer a fill platform using material varying from high plasticity basaltic clays to weathered basalt rock with boulders up to 2m in size. Critical to the residential housing development is the settlement behaviour of the engineered fill, particularly with regard to the performance of shallow footings founded on a fill thickness of up to about 35m.

At the time of preparation of this paper about 85% of the 2.4 million cubic metres of the stockpiled fill has been processed and placed back into the former quarry since the commencement of the project in April 2002. Settlement monitoring in some parts of the project commenced in July 2002. An overview of the trends of the settlement data so far is provided later in this paper.

The site is located on the western side of Steele Creek, a tributary of the Maribyrnong River, and is approximately 1.5km south of the Essendon Aerodrome. The site is 10km north-west of Central Melbourne, accessed via the Calder Freeway. Figure 1 below is an aerial photography of the site taken in 1989.

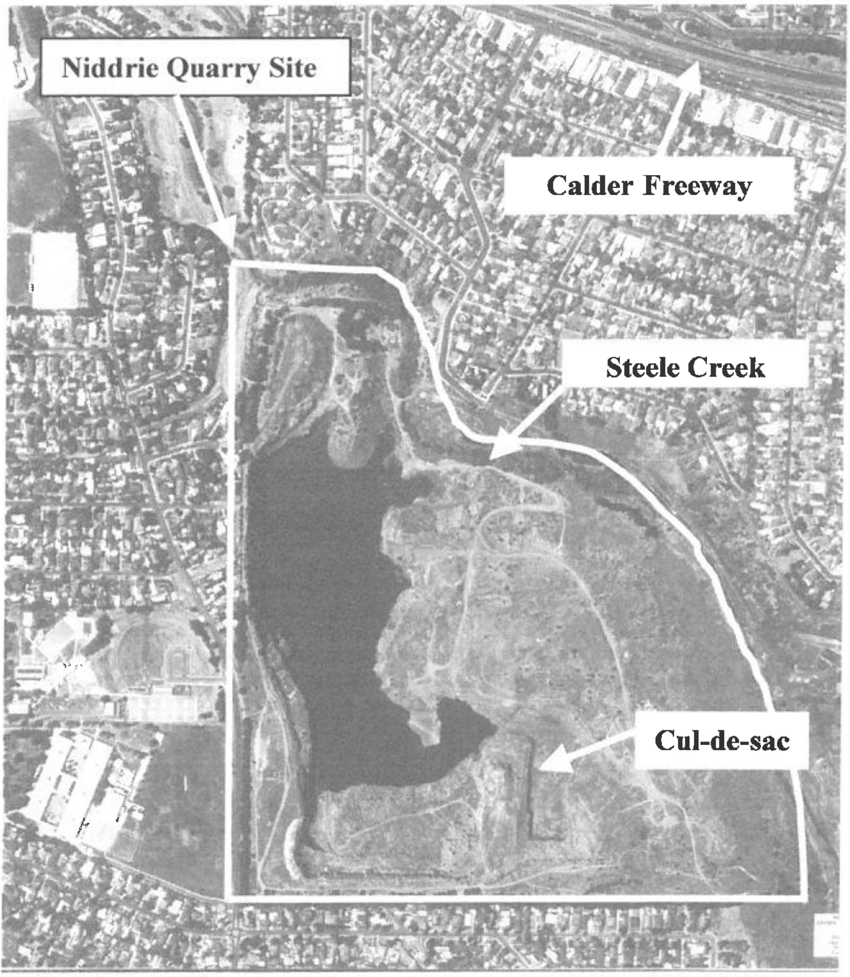


Figure 1 Aerial Photograph, 1989

2 DETAILS OF THE OLD QUARRY

The former Niddrie Quarry site covers a total area of approximately 48.4 ha, and of this the quarry hole is approximately 11 ha. The depth of water in the base of the quarry, prior to the development earthworks, was up to about 8m. At the completion of the quarrying operation about 2.4 million cubic metres of waste overburden material had been left around the edges of the former quarry hole in stockpiles up to 30m thick. Along the western edge of the quarry lake is a sheer basalt rock face, approximately 30m to 40m high. Existing residential properties are located some 10m to 20m from this western edge of the quarry hole. Figure 2 below shows a view of the old quarry hole looking south, just prior to the commencement of the earthworks.

Figure 2 View South

3 GEOLOGICAL SETTING

The surface geology of the quarry site is Quaternary age basalt known as the "Newer Volcanics", which has been established to be up to about 50m thick within the quarry area. This basalt overlies thin irregular Tertiary age deposits which in turn overlies Silurian age sedimentary mudstone, siltstone and sandstone, being the bedrock for the Melbourne region.

The basalt rock in the quarry is considered to comprise three distinct rock types. From the surface the profile comprises an ash like tuff, then weathered vesicular basalt and beneath is the massive columnar basalt which was the target for the quarrying operation. A variable thickness of residual basaltic clays of high plasticity were also found to overlie the weathered basalt rock in some parts of the site.

The stockpiles around the site generally contained the residual clays and the more weathered basalt rock overburden materials.

4 DEVELOPMENT PROPOSAL

In late 2000 the Urban and Regional Land Corporation (URLC) purchased the Niddrie Quarry site. After considering a number of options the URLC developed a vision for turning the existing disused quarry into a vibrant and modern housing development. This project is considered to be Australia's largest ever quarry rehabilitation project.

The URLC proposal is for a residential development whilst keeping approximately 30 percent of the site as open space. The key principles for the development include:

Retention of Niddrie Lake
Rehabilitation of the Steele Creek environment, abutting the east boundary
Retention and reuse of materials on-site
Minimal importation of materials

Plans have been developed for a total of 431 lots. This includes 376 conventional residential lots generally on the southern and eastern portions of the site. A number of medium density lots and a number of superlots for apartment development will also be made available. Figure 3 below shows a schematic plan of the proposed development.

Figure 3 Proposed Development Plan

The URLC, in consultation with their design team, assessed the critical aspects of their proposed development and how they might be addressed. Through various stages of planning and refinement of ideas, it was proposed to backfill the quarry, in part, using only the existing overburden materials and grading the surface of the fill down to the lake. The lake will be filled naturally by groundwater, and will form the centrepiece of the development.

5 GENERAL DESIGN PHILOSOPHY FOR ENGINEERED FILL

The design surface profile for the residential development was prepared in consideration of;

- The amount of available fill;
- The aesthetics of the site topography (in particular the need to maintain the lake as the prime focus);
- The maximum practical surface grades that could be tolerated;
- Maximising the area of residential lots founded on natural ground.

6 GEOTECHNICAL INVESTIGATION AND TESTING PROCESS

6.1 ASSESSMENT OF THE FILL MATERIALS

A number of geotechnical investigations of the materials in the fill stockpiles have been undertaken by Golder Associates and various other parties over the past 15 or so years. The fill material varied across the stockpiles but generally consists of a mixture of clay, sand and gravel, with basalt cobbles and boulders ranging in size generally up to about 2m maximum dimension.

Borehole investigations were inappropriate given the nature of the fill. Early investigations prior to the URLC purchase of the site involved test pits and trenches using large excavators. However, the maximum test pit depth was about 7m compared to a stockpile depth of up to 30m in parts.

Supplementary investigations were performed by Golder Associates and involved the excavation of test pits up to 16m deep. These investigations were conducted to allow the project team and potential earthworks tenderers to physically observe the materials to a greater extent than previously. The supplementary test pits were to a much greater depth than previously investigated and consequently the percentage of the stockpile investigated in terms of its thickness was significantly greater than all previous investigations, (that is, 60% compared to about 20% previously).

In summary, the fill materials encountered in about fifty test pit excavations across the various fill stockpiles typically comprised an average of about 35% to 40% of oversize material (ie. > 100mm particle size). A wide gradation of particle sizes was observed ranging from silt/clay to sand, gravel, cobbles, boulders and rock pieces up to 2m in size. In the main the fill stockpile material was described as a Clayey Sandy Gravel with cobbles and boulders throughout.

6.2 STRUCTURAL FILL MATERIAL TYPES

In consideration of the general nature of the available fill materials on the site, and of the type of residential development proposed, different fill zones were defined. For low rise residential areas where buildings will be supported by the fill, it was considered essential that at least the top zone of the fill comprise a well compacted high quality engineered fill. This is the zone in which excavation works for roads, services, retaining walls, building footings, batter slopes and other related excavations will be performed. This fill needed to be of a perceptually higher quality than the underlying fill. Material placed in this zone was designated as Type A Fill.

The limits of grading adopted for the Type A Fill are defined as:

- Maximum Particle size of 100mm.
- Not more than 30% exceeding 50mm particle size.
- Not more than 15% less than 0.075mm particle size.

This gradation was assessed visually during placement and by sampling and sieve testing.

Beneath the Type A Fill a coarser grade of fill was allowed. The material class for this zone was designated as Type B Fill with a nominal maximum allowable particle size of 600mm. The Type B Fill cannot be tested for compaction by conventional methods due to its coarse grading, so compaction of this material was judged predominantly by visual assessment and verification that the specified method placement had been strictly adhered to. The method specification established was based on the results of compaction trials performed prior to the commencement of the works. The low tolerance on clay content in this zone was for the purpose of reducing the potential for long term consolidation settlement.

The limits of grading for the Type B Fill material are defined as:

- Maximum Particle size of 600mm.
- Not more than 25% exceeding 300mm particle size.
- Not more than 15% less than 0.075mm particle size.

The material gradation was assessed visually during the placement and spreading process. It was also considered important that the compacted Type B Fill be free of voids to minimize the potential for the migration of fines due to subsequent saturation.

There are several areas in the development where higher level medium density housing is proposed. In such areas it is considered unlikely that the buildings will be able to be founded directly on the engineered fill due to their higher footing loads and the associated risk of excessive differential settlement. In these areas a third type of structural fill was specified. This fill is designated as Type C Fill.

The Type C Fill was designated to have a nominal maximum particle size of 200mm. This maximum particle size was derived in consultation with piling contractors so that conventional piling methods could be used to penetrate the fill in order to found piles on the natural ground beneath. The Type C Fill was placed under a method specification similar to Type B Fill. However, on the whole, the Type C Fill has been testable using conventional compaction testing methods.

The limits of grading for the Type C Fill are defined as:

- Maximum Particle size of 200mm.
- Not more than 30% exceeding 50mm particle size.

This grading of the Type C Fill was assessed during placement visually, and by sampling and sieve testing.

The Type C Fill may also comprise predominantly clay, as some long-term consolidation settlement of this zone could be tolerated, given that the building will be supported on piles and the maximum thickness of Type C fill to be placed is less than 10m. The clay fill had to be utilized somewhere on the project, and its potential for adverse impact as an engineered fill was considered least in this fill zone.

6.3 TRIAL PAD CONSTRUCTION

Prior to the commencement of earthworks a trial compaction pad was constructed to allow the project team and potential tenderers to observe the processes of blending, conditioning, placement and compaction of the engineered fill. The trial pad was used to demonstrate a method of construction which would be suitable and acceptable for the bulk earthworks project. It was a possibility that the successful tenderer for the bulk earthworks contract may propose alternative methods to those used in the trial fill. However, the trial operation was to provide tenderers with a basis to assess the aims and requirements of the earthworks design.

Different processes were trialed in the spreading, mixing, compaction and moisture conditioning of the fill materials. Variations were made in the layer thickness and the amount of compaction applied. Assessment of the trial pad was based on:

- precise level survey of a grid of points across the pad with levels taken after every second roller pass;
- visual inspection of slots excavated through each lift of the pad to observe the material grading, voidiness and compactness throughout the layer;
- sampling of the fine fraction of the material (<19mm) and testing for moisture content compared to Standard Optimum Moisture Content;
- proof rolling of the surface of each lift to check stability. A total of 3 lifts were completed during the trial.

In conclusion, the preferred placement method as a result of the trial was:

- placement of the Type B Fill in two nominal 0.5 m thick layers using tracked machinery;
- moisture conditioning the minus 19mm fraction to between SOMC to 3% DRY of SOMC;
- rolling at the top of the combined 1m thick lift with a minimum of 8 passes of an 18 tonne vibrating smooth drum roller or equivalent.

This placement method was altered following the award of the contract. The successful contractor undertook further trials which resulted in our accepting an alternative placement method that is:

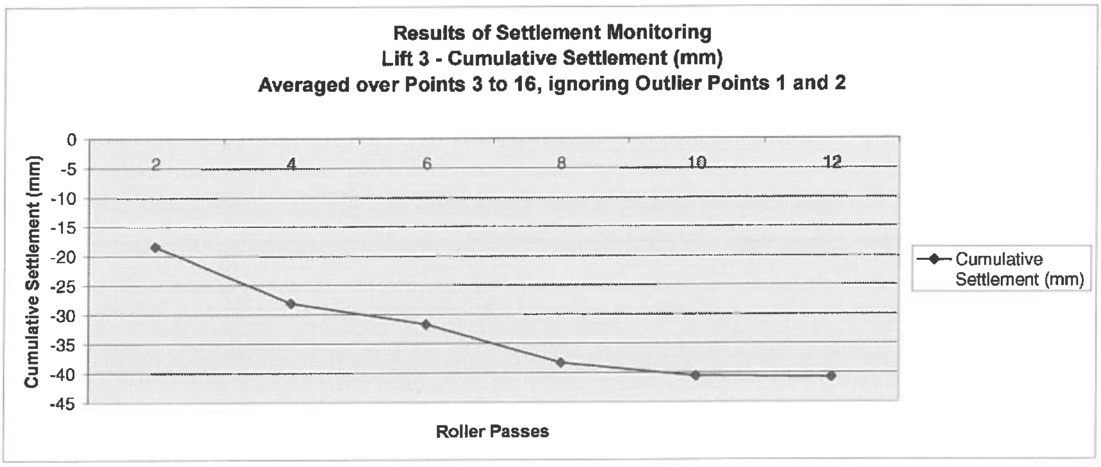
- placement of Type B Fill in two nominal 0.5 m thick layers;
- moisture conditioning the minus 19mm fraction to between SOMC to 3% DRY of SOMC;
- rolling each 0.5m layer using a static multi-wheeled 825C Compacter with a minimum of 4 passes of each 0.5m layer to achieve a maximum 1m thick lift.

Figure 4a shows the trial pad with settlement points marked by paint. Figure 4b shows the adopted spreading and compaction method.

Figure 4a Trial Pad

4b Fill Placement and Compaction

Figure 5 is an example of the results of settlement monitoring of one lift of the trial pad. The results demonstrate that further settlement of the 1m lift was negligible once 8 passes of the vibrating roller was reached.



Roller Passes	Cumulative Settlement (mm)	Cumulative % Complete
2	-18	45%
4	-28	69%
6	-32	78%
8	-38	94%
10	-41	99%
12	-41	100%

Figure 5 Trial Pad Settlement Monitoring

7 STRUCTURAL FILL DESIGN

7.1 SETTLEMENT CONSIDERATIONS

In general terms, the more densely compacted fill is, the lower its compressibility, or tendency to settle under its own weight or under an applied load. However, high levels of compaction are achieved by control over the spreading, moisture conditioning and compaction process.

If a reasonable minimum relative compaction is not achieved the potential exists for collapse settlement due to migration of material into voids, or as a result of saturation. The consequences of such settlement if it occurs after development would be serious. Similarly, if the fill contains materials that may decompose or dissolve, unacceptable settlements may occur. Selection of suitable sources of fill was therefore very important.

For predominantly granular fill, as exists for the majority of the fill materials on the site, which is subject to a reasonable level of compaction, the rate at which settlement occurs will diminish over time. Subject to an acceptable delay between fill placement and subsequent development this diminishing rate of settlement can be used to advantage in selecting the level of compaction applied. The higher the achieved relative compaction in the fill, the less delayed settlement that will occur and therefore the sooner an acceptable rate of settlement will be achieved.

Predicting the amount and rate of settlement of fill is difficult, especially where variable fill materials are used. Laboratory studies on expected typical fill materials can assist. However, for deep fills, with coarse rock included, settlement monitoring of the completed fills is considered to be essential in order to obtain an understanding of the performance of the as-placed materials as it relates to the timing of planned development.

In consideration of the long-term settlement of the fill and the recommendations given in the Australian Standard AS 3798 “Guidelines on Earthworks for Commercial and Residential Developments”, minimum compaction levels for the Type A and Type C Fills were selected. The acceptance of the compactness of the coarser Type B Fill was based on the method specification established as a result of field trials. The adopted compaction limits and maximum layer thicknesses for each structural fill type were as follows:

Table 1 Compaction Limits

Structural Fill Type	Minimum Dry Density Ratio (in accordance with AS1289.5.4.1)	Maximum layer Thickness (Compacted)
Type A Fill	98% Standard	0.3m
Type B Fill	Not Applicable	1.0m (2 x 0.5m)
Type C Fill	95% Standard	0.5m

It should be noted that:

- Settlement of the Type A, B and C Fill materials could not be accurately assessed prior to placement due to the placement method being adopted, particularly for that of the Type B Fill where up to 0.5m thick lifts were adopted.
- The settlement is influenced by the composition of the Type B Fill, in particular the proportions of boulders, cobbles, gravel, sand, clay and silt in the fill material.
- The settlement is expected to be influenced to some degree by groundwater conditions. It is anticipated that some increased settlement of the fill material will occur when it becomes saturated, as the groundwater level recovers over time.
- The amount of settlement is influenced by the thickness of the fill material.

Confidence in the expectation that the rate of settlement should be sufficiently small to allow release of some lots within a relatively short time after completion has been increased following the trial fill pad constructions, the stockpile investigations undertaken, the nature of the Type A, Type B and Type C Fill placed so far and the preliminary settlement data.

7.2 THICKNESS OF TYPE A FILL

During design development it was considered that the upper surface of the engineered fill profile would require more stringent production and placement requirements.

The upper zone of the fill profile is where all future works will be undertaken. This includes the construction of roads, services, retaining walls, building footings, battered slopes and other related excavations. For this reason it was considered that the Type A fill needed to perceptually be of a higher quality and therefore was a factor in assessing the required thickness of Type A fill.

In addition to the above it was considered that a thickness of higher quality fill at the surface would act as a bridging layer over the potentially lesser quality Type B Fill.

Taking into consideration the above factors, the design allowed for a typical thickness of 5 m of Type A fill. In certain areas of the site where the batter slopes are steeper than typical and benching of the fill is required to reduce surface grades, the thickness of Type A fill is to be 6 m thick.

Where Type A Fill is to be placed directly over a prepared natural subgrade its thickness may be less than 5m. There are also other examples where it was not considered critical that a full 5 m depth of Type A be placed. These areas included those beneath the retaining walls at the edge of the lake and beneath the polishing ponds. Some building areas have also had a reduction in Type A Fill were the finished grades are not steep and benching is not required. At these locations the minimum thickness of Type A Fill will be either 2 m or 3m.

7.3 RESIDENTIAL FOOTING DESIGN

It is intended that footing design for house lots on the engineered fill will be assessed in accordance with Australian Standard AS2870-1996, Residential Slabs and Footings-Construction. The Standard allows for controlled fill lots to be reclassified from Class P sites in accordance with accepted engineering principles and certain criteria. We anticipate that the lots will be reclassified to at least Class H sites which are defined as sites having a characteristic surface movement (y_S) in the range of 40 mm to 70 mm. The structural fill material, and in particular the Type A Fill, will comprise cobbles up to 100mm in a matrix of gravels, sands and some basaltic clays. An allowance of about 20 mm of surface movement may need to be provided for the potential soil reactivity (volume change with moisture content variation) of the high plasticity basaltic clay in the matrix of the fill material. However, it is noted that because the Type A Fill will have less than approximately 15% fines (clay/silt), this allowance should be more than adequate. Hence, to remain within a "Class H site classification" the amount of further differential movement to be tolerated by a house footing system would need to be less than 50 mm. That is, up to 20mm associated with shrink/swell movement and up to 50mm associated with ongoing total and differential settlement of the fill.

The assessment of the achievement of these criteria will be based on the rate and magnitude of settlement measured at the monitoring points both during the construction phase, for points within the fill, and following the completion of the main earthworks contract via top of fill movement measurements.

7.4 SETTLEMENT MONITORING

Settlement plates have been installed progressively as the fill has been placed. A total of sixteen monitoring installation locations have been established. Eight of these locations have two separate plates, one at a level of about 10 m above the base of the fill and a second plate at the top of the Type B Fill (underside of Type A Fill).

In addition to the settlement plates installed during fill placement, a greater number of surface settlement points will be installed immediately following completion of the works.

Settlement plates are to be monitored at the following frequencies.

During construction:	Weekly readings
Post construction:	Fortnightly readings for initial 3 months. Readings monthly thereafter

7.5 SETTLEMENT AT THE INTERFACE BETWEEN THE FILL AND NATURAL GROUND

The differential settlement will be higher on lots where there is a transition from natural soils and rock to fill material. Hence, the time lapse before release of these lots is expected to be greater.

The amount of settlement will depend on the thickness of fill material and the slope of the transition from fill to natural. Sharp interfaces with deep fill will incur greater differential settlement across this zone.

To reduce the effects of differential settlement at these sharp interfaces, the top edge of the natural ground has been battered back at a grade of 1H to 1V for the upper 3 m. This will create a transitional effect on the settlement differential. This aspect will be specifically assessed as part of the fill settlement program, with building lots in such areas only being released once the acceptable settlement criteria has been met.

8 RESULTS OF SETTLEMENT MONITORING SO FAR

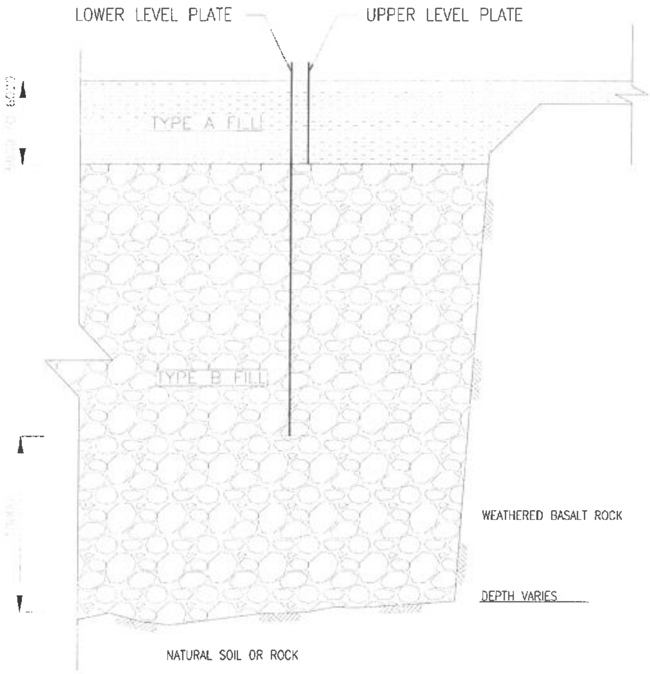
Filling on the project occurred first in an area known as the Cul-de-sac (Refer to Figure 1 for location). This was an area of the quarry that was mined late in its life and had minimal overburden across its base and a floor above the groundwater table.

five settlement monitoring devices have been installed in this area so far. These include three lower level plates, (SMP-01 to SMP-03) and two upper level plates, (SMP-04 and SMP-05). A summary of some of their details is presented in Table 2. The locations of these devices and a typical cross section showing the configuration of the plates are presented in Figures 6 and 7 below.

Table 2 Settlement Monitoring Plate Details

Settlement Monitoring Point	Date Installed	Level of Plate RL (m) AHD	Natural Floor Level RL (m) AHD	Thickness of Fill beneath Plate (m)
SMP-01(Lower)	25 June 2002	36.32	25.64	10.68
SMP-02(Lower)	16 July 2002	35.79	25.81	9.98
SMP-03(Lower)	30 July 2002	36.99	26.10	10.89
SMP-04(Upper)	6 Nov. 2002	50.61	25.75	24.86
SMP-05(Upper)	17 Oct. 2002	50.77	25.71	25.06

Figure 6 Cul-de-sac Settlement Monitoring Plate Locations



TYPICAL CROSS SECTION - SETTLEMENT PLATES

Figure 7

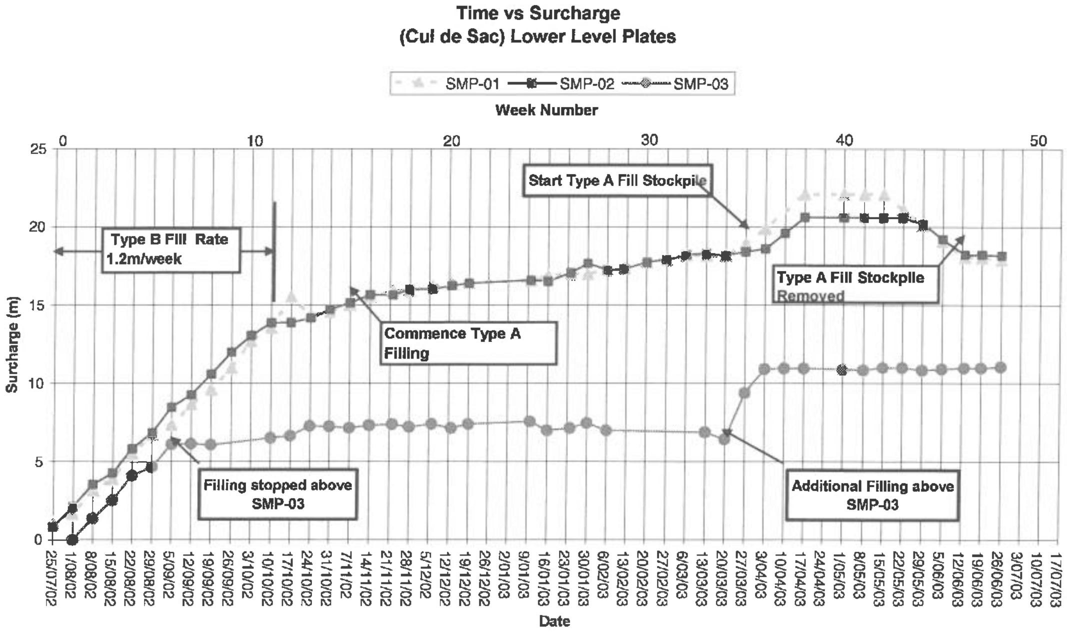
Figure 8 Upper and Lower Monitoring Plates with Temporary Type A Fill Stockpile

Lower Level Plates

Settlement Monitoring Plates, SMP-01, SMP-02 and SMP-03, lower level monitoring plates, were installed in the Cul-de-sac once the thickness of Type B Fill had reached about 10 m at each position.

Following installation, filling above the plates continued at the rates shown on the Time versus Surcharge Plot below.

Plot No.1

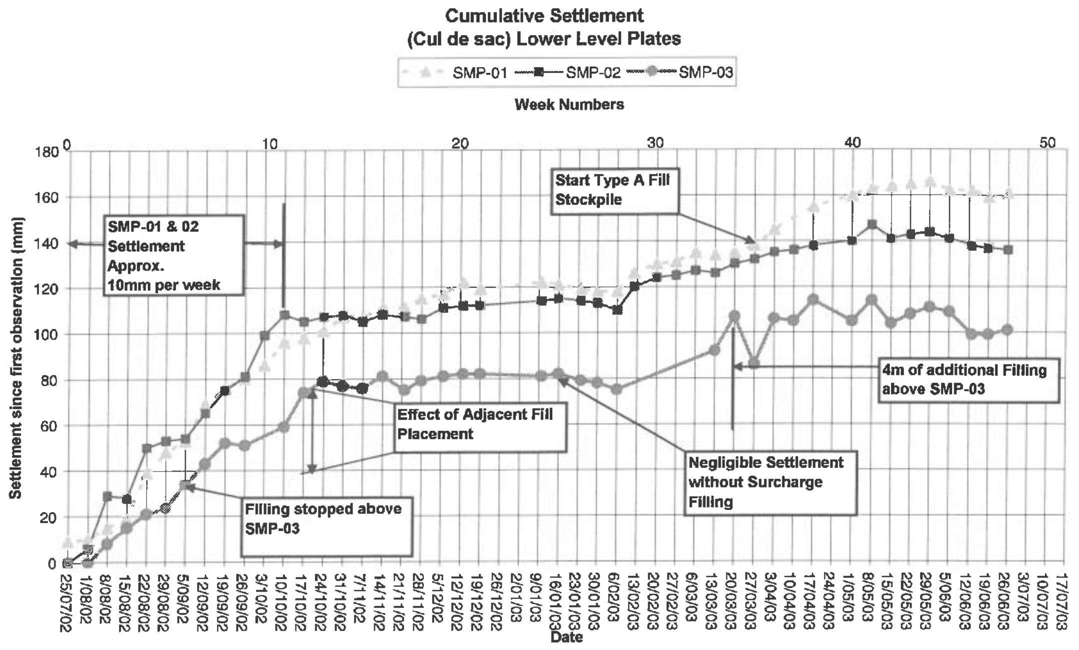


Points of note on Plot No 1 include:

- Filling continued at a relatively fast rate for the first 11 weeks (to 10/10/02), a rate of about 1.2 m per week.
- SMP-03 was located near the edge of the fill cell for the Cul-de-sac. At about five weeks after its installation fill above the plate had reached about 6 m. Following this time no further fill was placed above the plate until about the 20th March 2003 when a further 4 m of fill was placed over a 2 week period as a consequence of the abutting cell construction. No further fill was placed above this plate for the following 12 weeks (Weeks 36 to 48).
- Between about Week 11 and Week 15 the rate of Type B fill placement reduced. Only about 1 m of filling was placed in this period.
- On about the 7th November 2002, the placement of Type A Fill commenced in the Cul-de-sac. Its rate of placement was relatively slow compared to that for the Type B Fill. The thickness of Type B fill at the locations of SMP-01 and SMP-02 was about 25 m.
- About 3 m of Type A fill was placed between Week 15 (7/11/02) and Week 35 (27/03/03), a period of 20 weeks.
- Due to site constraints, on about Week 35 (27/03/03), Type A Fill was temporarily stockpiled in the Cul-de-sac area to a height of about 4 m for a period of about five weeks.
- SMP-02 was located at the edge of the Type A Fill stockpile and hence its impact on settlement was equivalent to about 2 m of surcharge.
- The Type A fill surcharge was removed by Week 47 (12/06/03).

Plot 2 below shows the cumulative settlements of SMP-01 to SMP-03 in the Cul-de-sac. The settlements reflect compression of the 10 m thickness of Type B fill beneath the plates as a consequence of the self weight of this zone and the added surcharge of fill above.

Plot No.2



Points of particular note on Plot No.2 include:

- The settlement rate of SMP-01 and SMP-02 mirrors the fast placement rate of Type B Fill above in the first 11 weeks. The average settlement rate was about 10 mm per week.
- Settlement of SMP-03 was less than SMP-01 and SMP-02 but continues at a steady rate until Week 12. Although Type B fill was not placed directly above SMP-03 after Week 6, its settlement was influenced by the load spreading effect of the adjacent placed Type B Fill.

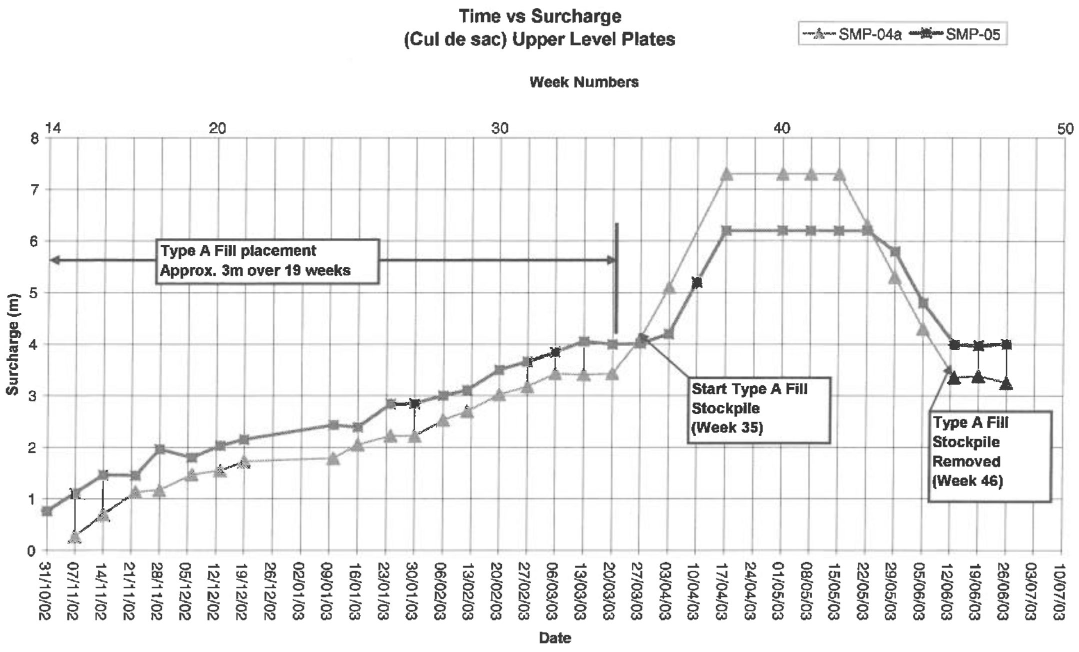
- Settlements of SMP-01 and SMP-02 were between 25 mm and 35 mm for the period Week 11 to Week 35, during which time about 4 m of additional fill was placed.
- Settlement of SMP-03 was negligible between Week 12 and Week 30 when no fill was placed in its vicinity.
- The impact of the Type A fill stockpile on SMP-01 was to increase settlement by about 25 mm, with about 5 mm of elastic rebound on its removal.
- The impact of the Type A fill stockpile on SMP-02, due to being at the edge of the stockpile was less with about 15 mm additional settlement but still resulted in a rebound of about 5 mm.
- The addition of 4 m of fill over SMP-03, between Weeks 34 and 36 resulted in about 20mm to 30 mm increase in settlement.

Upper Level Plates

At the locations of SMP-01 and SMP-02 the upper level Settlement Monitoring Plates SMP-04 and SMP-05 were installed respectively. These plates are located at the top of the Type B fill, at the interface of the Type A and Type B fill, and have about a 25m thickness of fill beneath them.

Plot No 3 below presents the Time versus Surcharge plot for SMP-02 and SMP-05.

Plot No.3

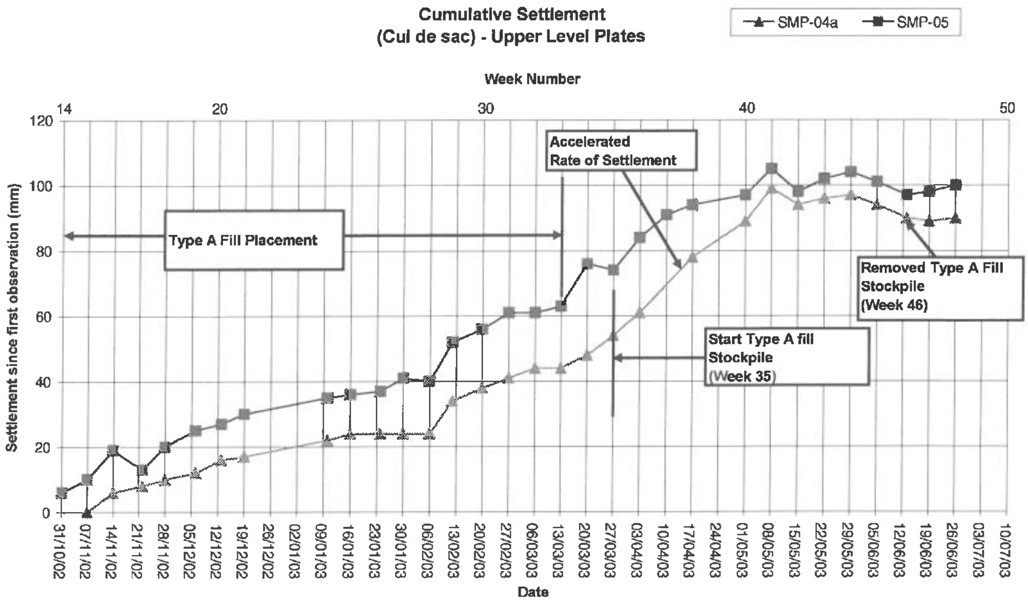


. Points of note on Plot No. 3 include:

- About 3 m of Type A fill was placed above these upper level plates during the period Week 14 (31/10/02) to Week 33 (13/03/03).
- The temporary Type A Fill stockpile was about 4 m thick at SMP-04 and about 2.2 m thick and SMP-05.

Plot No. 4 below shows the cumulative settlements of SMP-04 and SMP-05 in the Cul-de-sac. These settlements reflect compression of about a 25 m thickness of the Type B Fill beneath the plates as a consequence of the self weight of this zone and the added surcharge of Type A fill above.

Plot No. 4

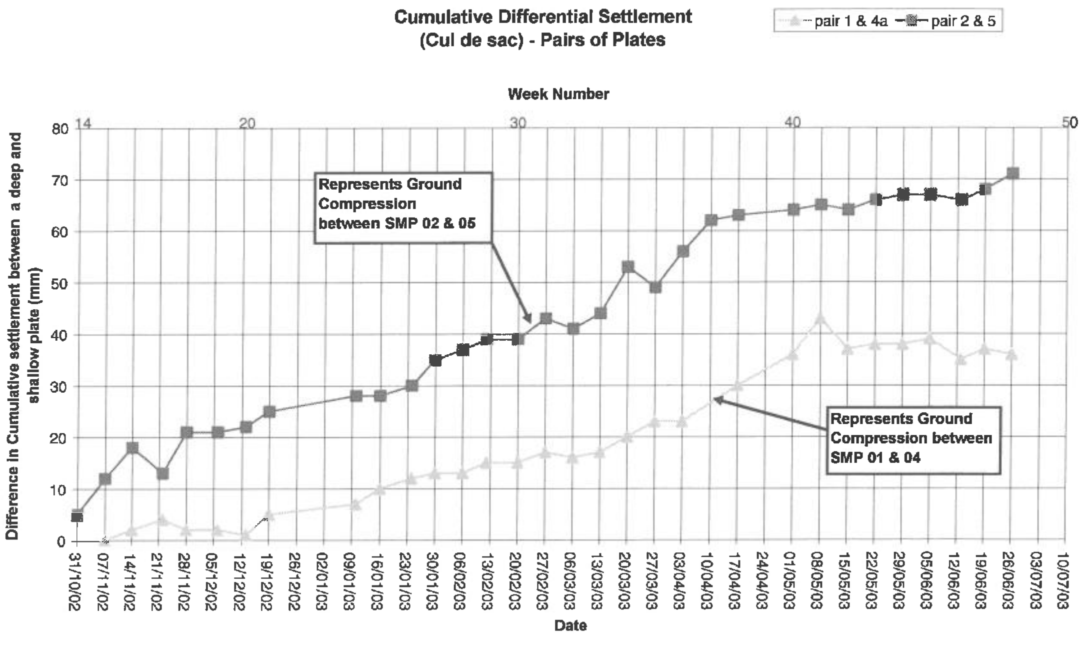


Points of particular note on Plot No 4 include;

- The total settlement of SMP-04, between Week 14 and Week 42, was about 90 mm.
- Removal of the temporary stockpile by Week 46 indicates a rebound of SMP-04 and SMP-05 of about 5mm.
- SMP-05 showed a similar amount of settlement to SMP-04 but slightly higher at 100 mm over the 34 weeks since installation.
- The rate of settlement of SMP-04 accelerated markedly during the period that the Type A fill stockpile was being placed.

Plot No.5 shoes cumulative differential settlement between the pairs of settlement monitoring plates. These plots represent the compression of the fill between the upper and lower plates.

Plot No.5

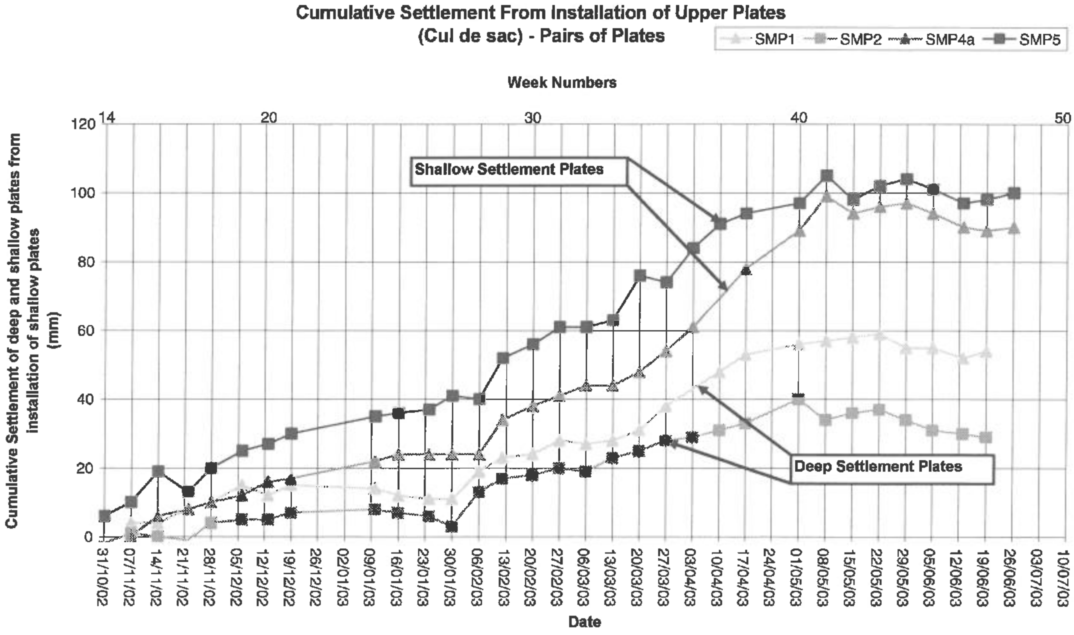


Points of particular note include:

- The compression of the fill between SMP-01 and SMP-04 was only about 40 mm compared to about 70 mm for the fill between SMP-02 and SMP-05. The reasons for the 30mm difference between these pairs of plates is uncertain. It is presumed to be associated with differences in the compressive nature of the materials between each location.
- Compression of the ground between the upper and lower plates has been small to negligible since Week 40 (0 to 5 mm).

The following Plot No. 6 shows the cumulative settlements over time of both the upper and lower plates, but only since the upper plates were installed in about November 2002. As would be expected, the deep settlement plates showed a lesser amount of settlement over this period compared to the upper plates, that is, 30mm to 60mm compared to 90mm to 100mm respectively.

Plot No 6.



8 CONCLUSIONS

At the time of this paper the bulk earthworks contract for the project has been running for approximately 15 months and about 85% of the 2.4 million cubic metres of fill has been placed. The adopted processes and methodologies for the placement of the various fill types have proved satisfactory. The preliminary settlement data has indicated that:

- Compression of the bottom 10m of fill occurred at a rate of about 10mm per week whilst fill above was being placed at a rate of about 1.2m per week.
- Settlement of the lower plates slowed significantly when the rate of surcharge fill placed above slowed.
- When fill placement above a plate had stopped, negligible ongoing settlement of the plate occurred over a measuring period of about 12 weeks. This indicates the fill response to be predominantly elastic.
- Placement and removal of the temporary fill stockpile indicated a generally immediate elastic settlement and rebound response of the fill for both the upper and lower plates.
- The total measured settlement (compression) of the 25m thickness of fill at the location of SMP-01 & 04 was about 250mm, and at SMP-02 & 05 was about 240mm, (ie. Approx. 1% of the fill thickness). It is noted that this includes only the measured settlement of the fill after the plates were installed.

The preliminary settlement data suggests that because of the predominantly granular and well engineered nature of the fill that estimated long term total and differential settlements of the fill should be sufficiently low enough to allow relatively early release of the lots for residential building construction.

Figure 10 Photograph in January 2003 - View South

9 REFERENCES

AS 3798 (1996). Australian Standard. Guidelines on Earthworks for Commercial and Residential Developments.1996

AS 2870 (1996) Australian Standard. Residential slabs and footings – Construction. 1996

Golder Associates Pty Ltd. (2002) *Geotechnical Design Criteria, Valley Lake, Residential Development, Niddrie.* Report 02612073/120. Prepared for the Urban and Regional Land Corporation. Dated 18 October 2002.

Golder Associates Pty Ltd. (2001) *Construction of Trial Compaction Pad Valley Lake, Niddrie, Victoria.* Report 00612146/042. Prepared for the Urban and Regional Land Corporation. Dated October 2001.

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Sydney Chapter

Using Bender Elements to Measure Soil Stiffness at Small Strains in Laboratory Samples

Dr David Airey
University of Sydney

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USING BENDER ELEMENTS TO MEASURE SOIL STIFFNESS AT SMALL STRAINS IN LABORATORY SAMPLES

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ABSTRACT

Over the last two decades improved laboratory techniques have shown that soil stress-strain responses are highly non-linear over the range of strains and stresses that are relevant around foundations and excavations. Accurate predictions of settlement and of the stresses in the ground depend on knowing this non-linear stress-strain response. One of the parameters needed to describe this response is the stiffness at very small strains. A variety of techniques are available to measure this stiffness. In this paper the bender element method is described. One of the attractions of using bender elements is that they can be easily incorporated into existing equipment, and at least in principle, the test is very simple to conduct. A high frequency pulse is applied to one bender element and the time for this to be detected at the other end of the test specimen is recorded. It is shown how bender elements can be incorporated into existing tests with minimal hardware requirements. A method of automating the test interpretation is presented which should enable reliable stiffness measurements to be obtained from commercial laboratory environments. This method makes use of cross correlation between input and output signals. It is argued that the approach is easy to integrate into existing test control programs, and it is shown how the technique may be implemented at relatively low cost.

1. INTRODUCTION

Accurate prediction of ground deformations around soil constructions of all kinds remains problematic. Over the last two decades it has become apparent that accurate predictions require a better understanding of soil's stress-strain response, which has been found to be highly non-linear over the range of strains (and stresses) that develop under working conditions. Many authors (e.g. Burland, 1989, Atkinson, 2000) have shown that the majority of the ground beneath and around foundations and excavations experiences direct strains of less than 0.1%, and if successful predictions of ground movement are to be made the behaviour at strains of 0.01% or better is necessary. A typical variation of secant modulus with strain is shown in Figure 1. The modulus is approximately constant for strains of less than 0.001%, and then drops significantly over the range of interest in the ground, between 0.001% and 0.1%. As pointed out by Burland (1989) these observations have far reaching practical and fundamental consequences. The non-linearity of the stress-strain response can have significant effects on soil-structure interaction, stress distributions, and displacement profiles around loaded areas and excavations. Also the interpretation of field measurements and in-situ tests needs to consider this non-linearity.

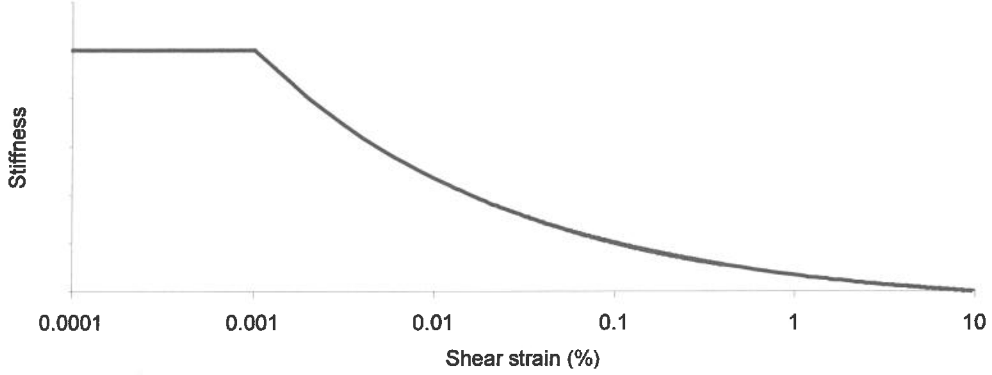


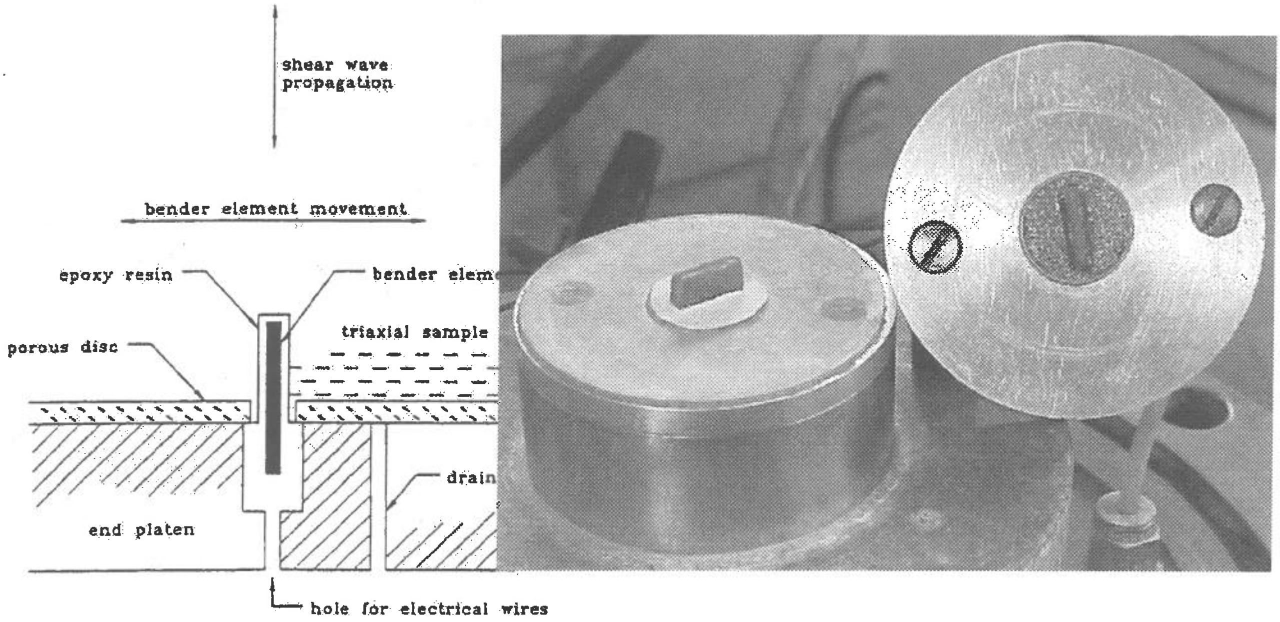
Figure 1 Typical tangent stiffness- shear strain response

Detailed design and analysis of major geotechnical structures taking account of soil non-linearity and other important features of soil behaviour is a complex task, but if this is done good predictions are possible (e.g. Jardine et al, 1991). For routine design Atkinson (2000) has suggested that the parameters needed are the stiffness at very small strains (<.001%), E_o, the failure strength, and the strain at failure. From these parameters it is possible to estimate the variation of modulus with strain and hence obtain compatible loads and settlements. Atkinson (2000) also suggested that the modulus at small strains could be obtained using bender elements. He stated that, "in principle, these tests are sufficiently simple to perform and the results are sufficiently reliable for routine analysis". Since 1997 we have been using bender elements at Sydney University for tests on cemented sands. The tests are simple to perform, but obtaining reliable interpretation has been the subject of considerable research and development (Mohsin and Airey, 2003). We believe that we now understand how to obtain reliable results, and that it is possible for these to be obtained during routine tests. This paper gives a brief overview of bender elements: what they are, how they work, and how to interpret the results.

2. BENDER ELEMENTS

The concept of using piezo-ceramic bender elements to measure the shear wave velocity in soils is not new. It was first suggested by Shirley & Hampton (1978) and closer to home was reported by Meyer et al. (1996) at the 7th ANZ geomechanics conference. Over the last two decades many researchers have incorporated bender elements into triaxial, oedometer and direct shear apparatus. The increasing popularity of this procedure is because of the increasing importance attached to knowing the shear modulus at very small strains, its ease of implementation, and practical difficulties with the use of alternative on-sample measurements.

The principle behind the operation of the bender elements is that certain "piezo-electric" crystals will become polarized when stress is applied to them, or conversely when placed in an electric field the crystals will change their shape slightly. The bender elements used in most geotechnical laboratory applications, known as bimorphs, are comprised of two thin strips of piezo-ceramic material bonded together and typically have dimensions of 13×10×0.5mm. They are mounted in a slot in the end platens of the triaxial or other device, as shown schematically in Figure 2a. The bender elements must be encapsulated in an epoxy for waterproofing, and located in the slots in the end platens so that the bender elements extend about 3mm into the sample. The gap between the slot and bender is filled with silicone sealant. The final arrangement is shown in Figure 2b.



(a) (b)

Figure 2 (a) section through end platen showing details of bender location (after Meyer et al, 1996), and (b) photograph showing bender elements mounted in 50 mm diameter end platens

The principle behind the interpretation of the bender element test is very simple. An electrical pulse is applied to one bender element, which is converted to mechanical energy and causes waves to propagate through the soil. A second bender element detects the wave and converts the mechanical wave to an electrical pulse. By measuring the time delay, T, between the applied and received pulses a shear wave velocity, v_s can be calculated knowing the length, L, between the tips of the bender elements. Hence the shear modulus, G, can be determined knowing the bulk density, ρ, from

$$G = \rho \, v_s^2 \tag{1}$$

Unfortunately, detection of the time delay is not straightforward and this has resulted in different researchers using a range of methods to obtain the "correct" arrival time. The following methods have been reported:

- Direct observation of the first arrival of the shear wave
- Direct observation from characteristic points (peaks, troughs) in the input and output signals
- Direct observation of the second arrival of the output signal
- Cross-correlation of input and output signals
- Frequency domain analysis to indirectly determine the phase angle, which is related to the time delay
- Frequency domain analysis using phase-sensitive detection techniques

Experimental and numerical results presented by Arulnathan et al. (1998) suggest that none of these methods are totally reliable, but under the right conditions all these methods are capable of giving the correct time delay (Mohsin and Airey, 2003). For all methods the factors that have the most impact on the accuracy of the estimated time delay are the frequency and shape of the input waveform, but there is no accepted procedure for selecting appropriate frequencies and waveforms.

Of the methods of estimating the time only those that analyse the complete waveform are suitable for automation, and hence of providing reliable and operator independent results. The method that has been developed at Sydney University and is discussed below is based on the cross-correlation of input and output signals.

3. EQUIPMENT AND PROCEDURE

The equipment required to implement bender element tests is shown schematically in Figure 3. Two arrangements are shown. Figure 3a is the conventional arrangement, which comprises a function generator to provide an input pulse and an oscilloscope to view both input and output traces. This is all that is required if the time is to be estimated from the traces on the oscilloscope. If any waveform analysis is to be carried out, as recommended here, then the information from the oscilloscope must be downloaded to a computer. To avoid the cost associated with a function generator and oscilloscope an alternative arrangement has been developed as shown in Figure 3b. This makes use of a computer sound card to generate the pulses and a computer analogue to digital (A-D) card to record the waveforms.

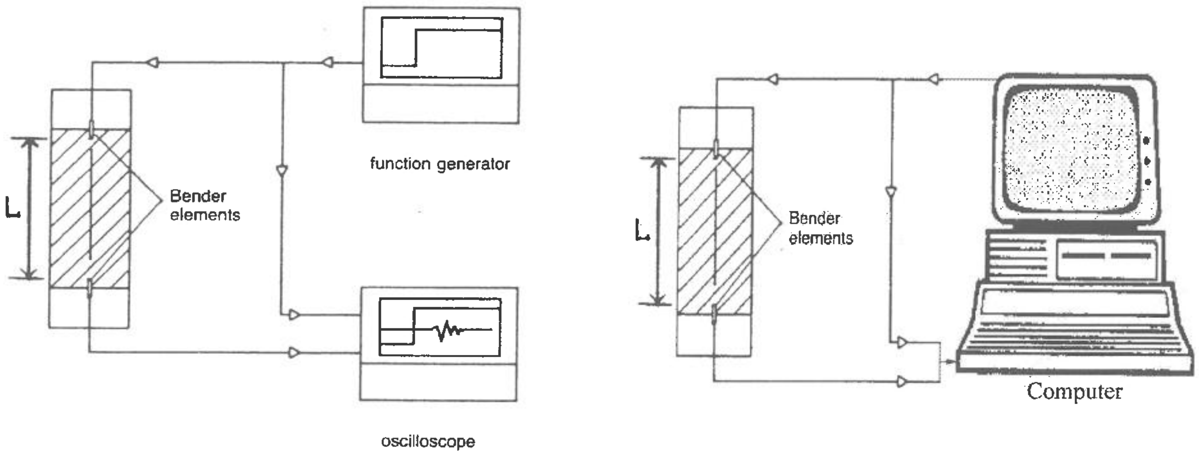


Figure 3 (a) Conventional equipment arrangement, and (b) Alternate low cost arrangement

The equipment we have been using consists of a 15 MHz HP33120A function/arbitrary waveform generator to provide excitation of the bender elements, and a Yokogawa DL1520L digital oscilloscope to monitor the input and received waveforms from the bender elements. An input signal amplitude of ±10 V has been used. After passing through the sample the output signal has been amplified to give a peak-to-peak amplitude of approximately 8 mV. The pulses, typically a sine wave with a frequency of 10 kHz, have been sent repetitively from the function generator at a rate of 100Hz. To remove the effects of random noise the oscilloscope is set up to display the average of the last 256 signals. During a test the averaged responses on the oscilloscope can be automatically downloaded to the computer at regular intervals. If a sound card is to replace the function generator it needs to provide pulse frequencies of up to 20 kHz and this requires the latest generation of (96kHz) sound cards. We are using a SoundBlaster Audigy 2 card although other sound cards have similar technical specifications. The sound cards can be programmed to output any desired waveform, however, the approach we have followed is to make use of a freeware program "Soundarb" that emulates all the functions of the function generator that are needed for the bender elements. A 16 bit A-D card with a 200kHz sampling rate is used to record the waveforms. The same A-D card can also be used to monitor and record the load, displacement and pressure data in the triaxial tests. To overcome noise some averaging of the signals is required and this requires accurate timing, which can be provided by the sync output from the sound card. It has been found that satisfactory traces can be obtained averaging 25 records of the output signal, and taking a single record of the input.

For both equipment arrangements after the computer receives the data the cross correlation of the input and received signals is computed. This is achieved by obtaining the fast fourier transforms, G(f) of the input signal (X(t)), and H(f) of the response signal (Y(t)). The cross-correlation CC(τ) can then be calculated using

$$CC(\tau) = IFFT(H^*(f)G(f)) \qquad (2)$$

where IFFT indicates the inverse fast fourier transform, and H*(f) is the complex conjugate of H(f). The fast fourier transforms have been performed using freely available software routines (Press et al. 1986) that can be easily linked into existing data-logging and control programs. The times of the positive peaks in the cross correlation signal are then recorded. There is no need to save the waveforms, although as discussed below it is desirable to view the waveforms to assist in selecting the correct time delay and for quality control purposes.

When implementing bender elements it is important that the elements are aligned so that they bend in the same plane, and with the same polarity so that positive displacement gives a positive signal for both source and receiver. The polarity can easily be checked by placing the bender elements in contact, and this also enables a check to be made for any the time lag in the electronic equipment.

4. RESULTS

For two similar waves shifted in time the peak in the cross correlation signal gives the time delay. However, due to resonance of the soil-bender element system, wave dispersion, and wave reflections the peak in the cross correlation is rarely found to coincide with the correct time delay. Some authors have cited this, along with the dissimilar input and output waveforms, as a reason for not using cross correlation. Nevertheless, when dispersion is minimal and the soil is behaving as a linear system, as is assumed in interpreting the results, one of the peaks should correspond to the correct time delay (Oppenheim & Willsky, 1997). Therefore the procedure adopted has been to record the times of all the significant maxima in the cross correlation signal occurring at or before the peak correlation. Typical waveforms showing the input and output signals at a confining stress of 100 kPa are shown in Figure 4. The cross correlation is also included on this figure. The peak in the correlation is labeled 1, and the preceding maximum is labeled 2. In this case it can be seen that maximum 2 corresponds approximately to the time of the first arrival of the shear wave. In Figure 5 shown below it will be seen that there are many maxima in the cross correlation and the correct time delay can correspond to the third and occasionally fourth maximum preceding the peak. How the correct time delay can be determined is discussed in the next section.

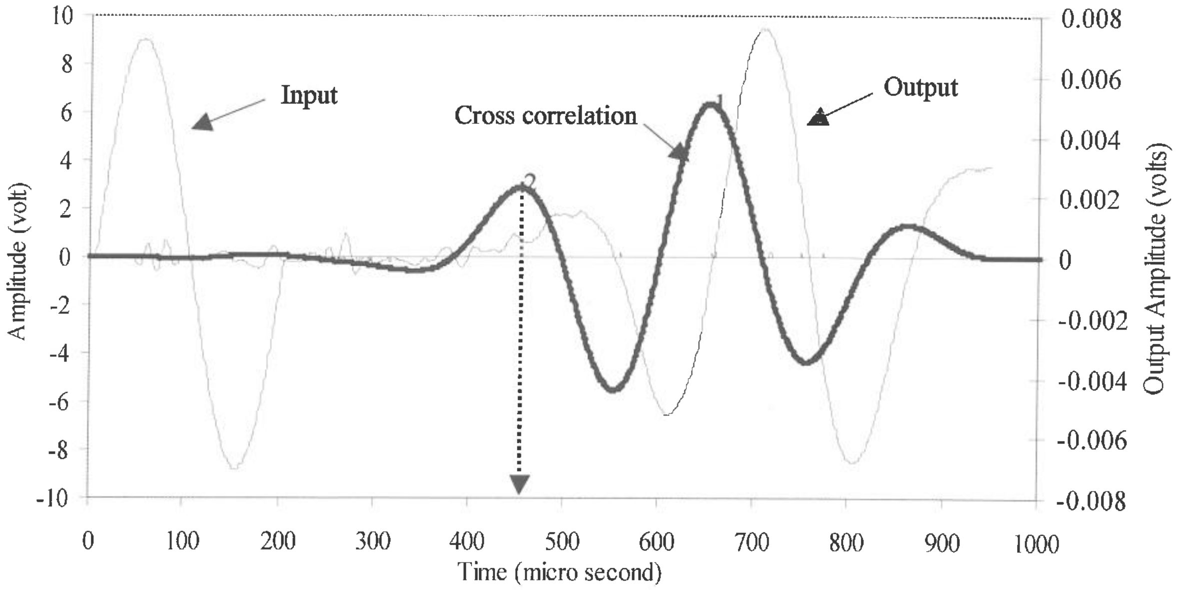


Figure 4 Typical bender element signals for Toyura sand, p' =100 kPa, L = 102mm

To demonstrate that the low cost arrangement using the sound card and computer A-D card perform satisfactorily a comparison is presented in Figure 5 of the input and output waveforms together with their cross correlation produced by: (a) the function generator and oscilloscope; and (b) the sound card and A-D card. Both plots show very similar input and output signals, and the times of the maxima, marked 1, 2, 3, from the cross-correlations are identical. The waveforms in Figure 5b are noisier than in Figure 5a because the output signal has only been averaged 25 times using the A-D card compared to 256 times using the oscilloscope. This appears to have no effect on the timing of the correlation peaks.

The output voltage in Figure 5b is shown to be similar to that produced using the function generator, but to achieve this some additional amplification of the signal was required. For a linear system, such as provided by the soil sample, the output amplitude is directly proportional to the input and the lower voltage from the sound card, ± 2.25V, compared to ± 10V for the function generator inevitably leads to a lower output signal. However, it should be noted that using a 16 bit A-D card and a sound card there is no need to have signal amplification despite the lower signal level. Amplification was only used here because the signals were being displayed on the oscilloscope, which had a lower resolution.

5. DETERMINATION OF CORRECT TIME DELAY

As already noted reliably determining the correct time is the biggest challenge when using bender elements. When using cross correlation there are two main factors to consider:

- The frequency and wave form provided by the function generator/sound card
- How to determine which is the right peak in the cross correlation

Other studies (e.g. Jovicic et al, 1996, Arulnathan et al. 1998) have reported that both the waveform and frequency can influence the identification of the correct time, particularly if using the first arrival of the shear wave. When using cross correlation it has been found (Mohsin and Airey, 2003) that if the frequency is greater than some critical value the timings of the peaks in the cross correlation do not change. The critical frequency depends on the shear modulus, and increases as the modulus increases. For sands subjected to a range of confining stresses between 20 kPa and 2000 kPa, corresponding approximately to shear moduli between 20 MPa and 400 MPa, the critical frequency varied from 8 kHz to 15 kHz. From these observations it appears that using as high a frequency as possible is desirable. However, the maximum frequency that can be used is limited by increasing attenuation in the soil, by the bender elements, which cannot transmit pulses accurately at high frequencies (>25kHz but depends on bender element), and by the response becoming noisier and the first arrival becoming increasingly difficult to detect.

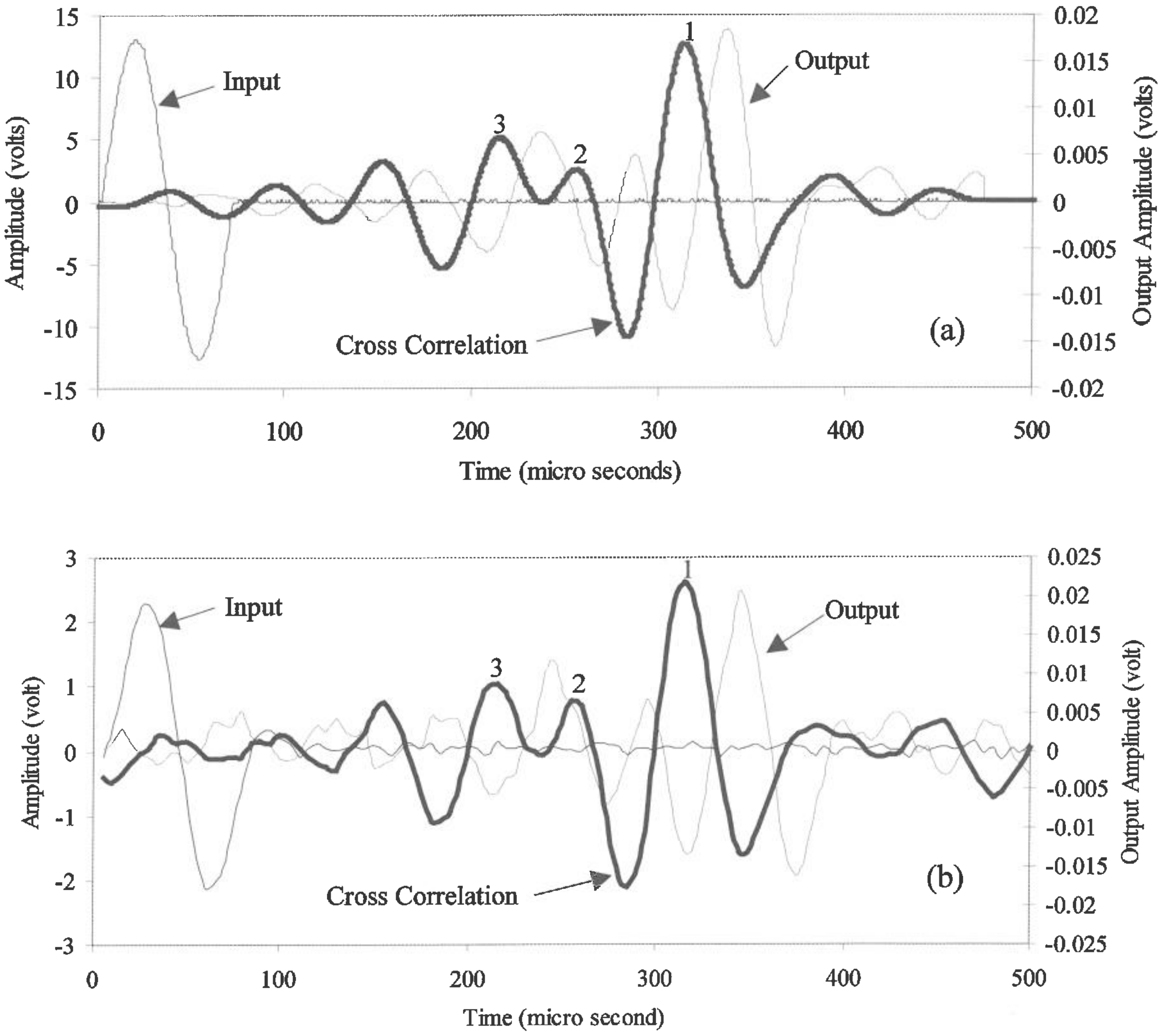


Figure 5 Traces from (a) Function generator/Oscilloscope, (b) sound card/A-D

A variety of waveforms have been investigated including single sine pulses, sine pulses with two or three waves, a triangular pulse and a chirp comprised of three sine waves of different frequency (Mohsin and Airey, 2003). All the pulsed waveforms give essentially identical travel times when the frequency is greater than the critical value. The best waveforms have been found to be a triangular input wave, which gave the least frequency dependent travel time, and a chirp waveform comprising a packet of sinusoidal waves with frequencies of approximately 20 kHz, 13 kHz and 8 kHz. Having a range of frequencies in the input signal is an advantage as this results in less variability in the estimated times and avoids the need to adjust the input pulse frequency during a test.

The major limitation when using cross correlation is that the times of several peaks in the cross correlation must be recorded because there is no simple way of determining which peak is giving the correct time. This is particularly a problem when testing sand as the first arrival can be difficult to detect, as for example in Figure 5. It has also been seen in Figure 4 that the maximum in the cross correlation giving the correct time is not necessarily associated with the greatest correlation. To resolve this problem the following approach has been used. The signals are correlated at regular intervals so that a continuous variation of the times of all the maxima is obtained. Figure 6 shows a typical result during isotropic compression of Toyura sand. At each confining stress 3 or more points are shown corresponding to the times of

Figure 6 Influence of confining stress on the timing of the maxima in the cross correlation signal

the various maxima in the cross correlation signal. The different symbols on the plot refer to the different peaks in the cross correlation. In general it has been found that the lowest monotonically varying value gives the correct arrival time.But, if only a small range of confining stress is considered this may not be reliable. To overcome this uncertainty it is recommended that the correct time at low confining stress be identified by adjusting the frequency until the arrival time can be confidently estimated. As the input frequency is changed it is possible to observe a small range of frequencies where the response signal has at least one peak-trough cycle at the same frequency as the input. In this "optimum" frequency range the time delay can be determined between corresponding peaks or troughs in the input and output signals. This can then be used to indicate which curve in Figure 6 is the correct one. At later points in the test only with this continuous variation is it possible to confidently detect the correct peak from the cross correlation data.

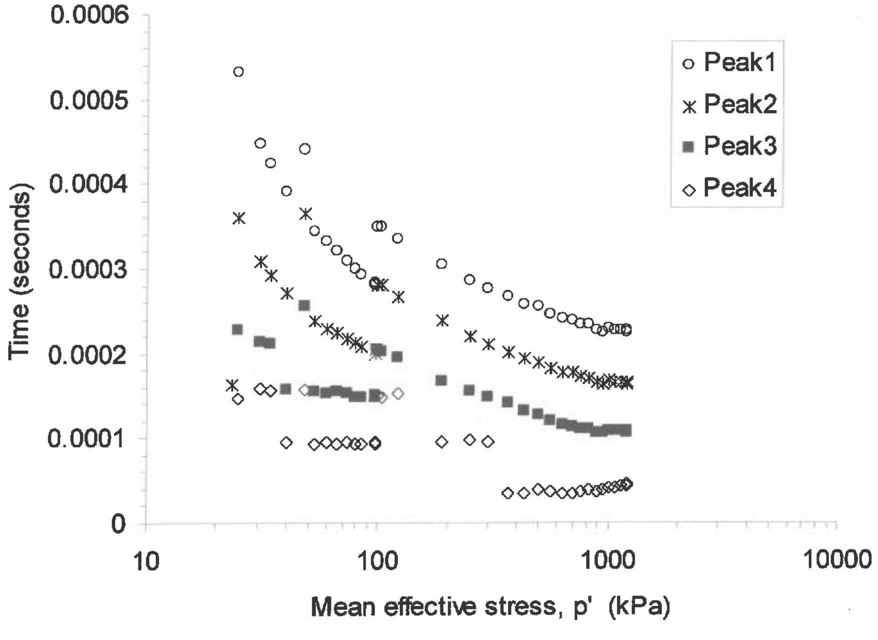
The cross correlation technique has been criticised because it is computationally intensive (e.g. Blewett et al. 1999) requiring the capture and analysis of complete waveforms and the use of fast Fourier transforms. However, with modern fast computers the time to capture the data and complete the calculations is not significant, and data storage is minimal as only the times of the peaks in the cross correlation need to be saved. It is also probably desirable to save a few waveforms to confirm the identification of the correct peak in the cross correlation and for quality control purposes.

Mohsin and Airey (2003) have shown that this method is capable of giving values of G_{max} that agree with values obtained by torsional shear and resonant column tests. As the method gives reasonable results and removes any operator dependence it is believed that the method outlined here is suitable for use in commercial laboratories.

6. SUMMARY

The stress-strain response of most soils is highly non-linear over the range of stresses and strains relevant to working conditions around foundations and excavations. The stiffness at very small strains is one of the parameters needed to define the stress-strain response. Using bender elements it is possible to measure the small strain stiffness during standard soil mechanics tests such as triaxial, oedometer, and simple shear. These devices are relatively cheap and are easy to install in the end platens of the test apparatus. It is possible to perform the tests making use of already existing hardware such as computer sound cards and existing data-logging equipment.

Obtaining reliable measurements from the method requires some care in the selection of frequency and waveform of the input pulse, and in the interpretation of the wave travel time. It has been shown that cross correlation can give reliable and operator independent results. However, it is important to realize that the time is obtained from one of the maxima in the cross correlation signal, and not necessarily the peak correlation as would be expected if the signals were simply



shifted in time. It has been found that the frequency of the input pulse must be above some critical value that increases with effective confining stress. To avoid having to adjust the frequency during a test a Chirp waveform comprised of sine waves with frequencies of 20, 13 and 8kHz has been found to work satisfactorily for sands with confining stresses from 20kPa to 2MPa.

The major limitation of this technique is that it is difficult to determine which is the correct peak in the cross correlation signal. The data show that the arrival times predicted by several peaks in the cross correlation vary monotonically during the test. To select the correct peak a visual check of the waveforms may be required at some stage of the test.

The technique is relatively cheap to implement and should be within the capabilities of commercial testing laboratories.

ACKNOWLEDGEMENT

The development of the bender element technique has been supported by the Australian Research Council, through the Centre for Offshore Foundation Systems (COFS). Valuable assistance has also been provided by Dr Collings with the signal analysis and by PhD students AKM Mohsin and Shane Donohue and undergraduate students Craig Covich, Jack Huang and Edgar Ramani.

REFERENCES

Arulnathan, R., Boulanger, R.W. and Reimer, M.F. (1998). Analysis of bender element tests, *Geotechnical Testing Journal,* 21(2), 120-131

Atkinson, J.H. (2000) Non-linear soil stiffness in routine design, *Geotechnique* 50, No 5, 487-508

Blewett, J., Blewett, I.J. & Woodward, P.K. (1999), Measurement of shear-wave velocity using phase-sensitive detection techniques, *Canadian Geotechnical Journal*, 36: 934-939

Burland, J.B. (1989) Ninth Laurits Bjerrum Memorial Lecture: Small is beautiful – the stiffness of soils at small strains, *Canadian Geotechnical Journal* 26, 499-516

Jardine, R.J., Potts, D.M., St John, H.D. and Hight, D.W. (1991) Some practical applications of a non-linear ground model, *Proc. 10th European Conference Soil Mechanics and Foundation Engineering*, Florence, 1, 223-228

Jovicic, V., Coop, M.R. and Simic. M. (1996) Objective criteria for determining Gmax from bender element tests. *Geotechnique*, 46(2), 357-362.

Meyer, V.M., Pender, M.J., Nishihara, A. and Duske, G.C. (1996) Elastic soil properties from bender element tests, *Proc. 7th ANZ Geomechanics Conference,* 123-128.

Mohsin, A.K.M. and Airey, D.W. (2003) Automating Gmax measurement in triaxial tests, To appear *Deformation Characteristics of Geomaterials*, IS LYON03, Sept

Oppenheim, A.V. and Willsky, A.S. (1997) *Signals and Systems*, 2nd edition, Prentice-Hall

Press, W.H., Flannery, B.P., Teukolsky, S.A. and Vetterling, W.T. (1986) *Numerical Recipes, The art of scientific computing,*

Shirley, D.J. and Hampton, L.D. (1977) Shear wave measurements in laboratory sediments. *Journal of Acoustical Society of America,* 63(2), 607-613.

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Dr Julian Seidel
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Geotechnical Instrumentation and Construction Works Compliance Testing

Compliance Testing and Instrumentation of Piles

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1. Introduction

This paper is intended to provide an overview of the issue of foundation pile instrumentation, and to help the practitioner understand some of the options that are available, and some of the issues which are pertinent to the choice, specification and interpretation of piling instrumentation.

The paper takes a wide view of what constitutes piling instrumentation. In some, this may be generic devices which are utilized in piling instrumentation - for instance to measure strain or displacement. In other cases, the paper looks at piling-specific technology or measurement systems which have been developed to collect particular information.

In general, piling instrumentation is used to ensure construction works compliance testing. It is used as part of a testing regime, or a quality assurance system in order to ensure that the completed pile, as a representation of the whole foundation, will perform satisfactorily for the life of the structure which is to be supported.

Although this is by no means a unique problem in geotechnical engineering, the fundamental problem that we face with ensuring compliance with foundations is that the entire system is buried. The quality and fitness of the foundation is dependent on a large number of issues :

- the type of foundation element (see Figure 1);
- the quality of the construction process;
- the experience of the supervisory and construction personnel;
- the quality of the site investigation;
- the variability and sensitivity of the ground conditions discovered in that investigation;
- the designer's awareness of and response to those conditions;

Our confidence in the foundation also depends on :

- the type of verification testing which is undertaken
- the frequency of that verification testing

The Australian Standard, AS2159-1995 : Piling – design and installation, accounts for these issues in Section 4 – Geotechnical Design. In particular, Table 4.1 and Table 4.2 provide guidance to the designer on the selection of the geotechnical strength reduction factor. These tables are reproduced in Figure 2 and Figure 3.

The primary function of these tables is to draw the designer's attention to the risk factors and issues which affect foundation construction. The use of a range of ϕ_g values instead of a single value is intended to highlight and encourage the benefits that accrue from good practice – in site investigation, in design, in construction and in verification. Ideally, the additional costs of good practice should be more than compensated by the lower foundation cost which results from the higher ϕ_g values which can be justified.

Pile loads – particularly in Australia – are becoming ever higher for a given section. Reinforced concrete piles which 20 years ago may have been good for a 40 tonne working load are now expected to carry working loads of 200 tonne or more in some instances. Part of this may be the result of improvements in material technology, but it is more due to economic imperatives which have forced us to exploit piles to their limit in a need for design efficiency, and hence cost efficiency. Increased loads and stresses are typically associated with reduced redundancy and higher risk. It is important that the increased risk is compensated by increased monitoring and verification.

As intimated above, at a basic level, there are two approaches to ensuring compliance of foundations. These will hopefully be applied in combination rather than isolation. The first is to ensure compliance of the completed foundation with the performance specification. The second is to ensure the compliance of the construction process with good practice. We will deal with these matters separately.

2. Compliance with the Performance Specification - General

2.1 Testing programs

We can attempt to verify compliance with a performance specification by undertaking some form of instrumented testing before the construction contract, or during or after completion of the construction. The purpose of testing prior to commencement would be confirm the validity of the design and the construction process, and give maximum opportunity to modification of either with minimum cost and disruption prior to commencement of the contract. There are, of course, balancing time and cost issues related to pre-contract test programs which often make them economically unattractive. In the absence of a pre-contract program, the project partners should strive for some tests at the earliest possible time to maximize the benefit of testing. In a large contract, the testing program should be developed to give both temporal and geographical distribution of testing across the site. Critically loaded piles, or critical geotechnical conditions should be given greater emphasis.

2.2 Representativeness

It should be remembered that the costs associated with 100% compliance testing will be prohibitive in most (but not all) cases. There is a requirement, therefore, that the piles tested are either chosen on the basis of their having the likely worst performance, or alternatively that they are statistically representative of the whole foundation. Due to high costs, it is unlikely that static load testing, for instance, will ever be justified in sufficient numbers to be statistically significant. Therefore, the designer should generally target the worst likely pile for static load testing. This may be possible if the designer has an experienced representative on site, and if good construction records are available with which to make an informed decision. Nominating test piles in advance is an almost certain way of ensuring compliance of the test pile. It may, however, say nothing about the representativeness of the pile. Alternative methods such as dynamic load testing which are rapid and relatively low cost, may allow a statistically representative sample of piles, with good geographical distribution to be evaluated. A different philosophy could be used for their selection.

2.3 Proof or failure testing

In practice, most foundation testing is undertaken only as a contractual requirement, and to satisfy the nominated performance specification – i.e. a nominated 'factor of safety' above working load or in limit state reaching S^*/ϕ_g , or meeting a maximum deflection criterion at working load or equivalent. Such proof testing is usually adequate for the purposes of the contract, however, testing to 'geotechnical failure' is of tremendously more value to the designer, and should always be encouraged. The feedback available to the designer may have direct benefits to the contract in either optimising the foundations, or extrapolating to other areas or depths. In the longer term design predictions on subsequent projects in the same geology will be more realistic. It is noted that testing to geotechnical failure does not render a foundation element useless. In fact, work hardening often improves the performance of the geotechnically failed pile relative to untested piles. Of course, pile tests should not in general be taken to structural failure, and structurally failed elements must not be incorporated into the structure.

2.4 Standards

Pile testing is covered in Section 8 of AS2159. Section 8.3 covers the particular requirements for static compression, tension and lateral load testing. Section 8.4 covers the particular requirements for dynamic compression testing. Section 8.5 provides some brief clauses on integrity testing methods.

It is pertinent to emphasize that AS2159 does not preclude the use of new or different testing technologies, but notes (in 8.2) that these may be used at the discretion of the designer.

AS2159 specifies a standard incremental static load test procedure, and default acceptance criteria for static load tests. Similar standard procedures and acceptance are nominated for dynamic pile tests. However, it is important to note that variations to both the standard load schedules and the acceptance criteria are allowed. Designers are encouraged to develop load schedules and acceptance criteria which are relevant to the particular structures being supported.

2.5 Why test?

It follows from the previous section that we may wish to instrument and test a pile to determine :

1. capacity – axial compression or tension, lateral. This is a test which relates to verification of the ultimate limit state design;
2. load-deflection response – axial compression or tension, lateral. This is a test which relates to verification of the serviceability limit state design, and the structure-foundation compatibility;
3. pile integrity – this is for construction verification, especially for cast-in-situ piles, but also for driven preformed piles;
4. construction control – this will mainly be dealt with in respect of monitoring of installation. However, it is necessary to relate the outcomes of foundation verification tests to foundation installation parameters to ensure that reliable installation criteria are developed (e.g. the pile driving "set").
5. foundation system verification – as noted before, the greater purpose of pile testing is to ensure the suitability of the complete foundation system.

The following sections will discuss the available test methods which address these requirements.

3. Evaluation of Capacity and Load-deflection response – Test options and Instrumentation

The determination of capacity, load-deflection response and integrity are undertaken subsequent to construction, as a check on compliance with the performance specification. Static test methods, and quasi-static test methods (e.g. Statnamic®) can provide an assessment of both capacity and load-deflection response. Dynamic methods are separated into high-strain and low-strain methods. High-strain methods can provide assessments of capacity, load-deflection response and pile integrity. Low-strain methods provide information only on pile integrity. These techniques will be discussed further in this section.

3.1 Routine Static Load Testing

Routine Static Load Testing - General

The following discussion relates primarily to the most common compression load testing, however, the requirements for tension and lateral load testing are equivalent.

Static load tests are typically monitored only at the pile-head for the purpose of establishing the pile-head load-deflection response. It is extremely important that both load and deflection are measured accurately. AS2159-1995 has an absolute requirement that load is measured using a load cell. Measurement of the jack pressure using a manometer is no longer considered acceptable – studies have shown that such readings can be 20% or more in error due to very small system eccentricities.

Deflection must be measured by dial gauges or electrical transducers to an accuracy of the lesser of 1% of pile diameter or 0.5mm. Displacement measurements can often be made to much greater accuracy. AS2159 requires four dial gauges or electrical transducers to be employed at equal distances around the pile head.

The accuracy of displacement measurements depends on shielding the devices from sunlight or other environmental influences. Furthermore, a critical issue is that the deflection is relative to a stable reference, or that the reference movement is measured and that appropriate corrections are applied. The reference beam should be founded outside the footprint of the test to minimize the influence of the pile and any reaction systems on the reference beam. Pile-head movements may be measured or supplemented with precise levelling measurements from well outside the zone of influence of the test. Reference beam movements are typically monitored at each load stage using the precise level.

Depending on the reaction system used, reaction piles, anchors or kentledge supports should be monitored regularly for deflection to ensure stability and warn of any imminent failure. Anchors or reaction piles may also be monitored with load cells. Examples of static load test arrangements are shown in Figure 4.

All these measurements may be taken and recorded manually, however, it is preferable, both from the aspect of objective reliability, and for reasons of safety that measurements be made electronically and logged automatically. Access to the pile head during the progress of the test should be discouraged or disallowed. Use of dataloggers is now very common, and systems are available for total automation of static load tests including both control and monitoring by feedback control.

The interpretation of ultimate pile capacity – even if taken to "failure" is often not clear and unequivocal. As summarized by Fellenius (1980), there are many alternative proposals and constructions for interpretation of pile load tests which may result in a wide range of capacity estimates. There are a number of analytical or graphical methods which attempt to assist with interpretation of the comparative base and shaft contributions, however, these are at best only approximate. In some cases, the separation can be deduced intuitively.

Routine Static Load Testing – summary of issues

Features	**Requirements**
Generally accepted as the reference test	Reaction system and jack
Typically 7-10 days after installation	Sufficient distance between pile and reaction system
Require 100% of test load as reaction	Load cell for measurement of applied load to 2%
Compression, tension or lateral	4 dial gauges, or transducers with at least 0.5mm accuracy, or precise level
	Comply with loading regime in AS2159
Advantages	**Disadvantages**
Direct (requires no interpretation)	Expensive
Easy to understand	Time consuming
Gives most engineers confidence	Possible errors due to interaction with reaction system
	Errors from manometer (up to 20%)
	Not statistically significant
	Difficult to extrapolate logically to other piles on site
	Can't assess time effects
	No resistance distribution unless instrumented

Routine Static Load Testing – Concluding remarks

Static load testing has been in decline for many years since the advent of rapid and cheaper alternative test methods. However, despite these new tests being particularly suitable for driven piles, the philosophy which has developed of reducing the number of static load tests has been applied as a blanket approach to all piles types.

Static load testing should be reintroduced as a standard requirement for all piling contracts, as is done in most other countries. In particular, more static load testing should be demanded for piles where there is either no direct feedback during the installation process, or where the feedback (e.g. torque) is not a reliable measure of capacity, i.e. bored piles, CFA and screw piles.

As an industry we need to establish a reliable database of test results for the new pile types that we are installing and for the higher loads that we are applying.

3.2 Instrumented Static Load Testing

AS2159-1995 does not provide any guidance on instrumentation of the pile shaft, which is the preferred method of evaluating shaft and toe resistance distributions. In Australia, instrumentation of the pile shaft is uncommon, and would essentially be used only in research applications, or in cases where the potential for indirect determination of resistance distribution by measurement of the variation of strain along the pile length has the potential for significant cost savings. In other countries (e.g. Hong Kong), multi-level instrumented pile load tests are routine.

Pile instrumentation depends on measurement of strain in the pile shaft. Measured strains at each level are multiplied by assumed section modulus and shaft area to infer the shaft load. The average mobilized shaft resistance at each level are computed from the successive load differences and the intervening pile surface area. There are a number of fundamental difficulties with interpretation of this data.

1. For concrete piles, the modulus must be assumed. It is normal to calibrate the system with a set of measurements just below the pile head. Given the measured pile cross-sectional area, and the known load, the effective modulus near the pile head can be determined. The modulus may vary along the pile length, but this cannot be determined or accounted for.
2. Concrete has a highly non-linear stress-strain behaviour, with initial tangent modulus being many times higher than the secant modulus at high strain levels. The relevant modulus will therefore vary with load level, and the strain at any location in the pile.
3. For cast-in-situ piles, the cross-sectional area may also vary, and this will also remain unknown and may influence the accuracy of the results.
4. Interpretation is normally based on an assumption of a strain-free pile at the commencement of the test. This is unlikely to be true for driven piles, and may be untrue for drilled shafts. Residual stresses in the pile will lead to an overestimate of shaft resistance and an underestimate of end bearing. Special procedures are required to more accurately determine the resistance distribution.

Given the difficulties of installing strain measurement devices in harsh construction environments, measurement redundancy is highly recommended. This is particularly important near the pile head, where any bending effects coupled with inactive instruments, could result in a significant misinterpretation of the section modulus. The available devices for instrumenting static load tests are well summarized by Sellers (1995).

Instrumented Static Load Testing - Strain measurement devices

Two types of sensors are typically used for measurement of strain in instrumented static load tests – electrical resistances strain gauges, and vibrating wire strain gauges.

Vibrating wire strain gauges are generally preferred, as they are less susceptible to the effects of moisture and the length of cables, have significantly superior long-term stability, and require less datalogger channels for monitoring. For steel piles, either weldable, or boltable types are available (see Figure 5). Consideration must be given to physical protection of the gauges during driving, and the cables must be secured to avoid inertial effects. For cast-in-place concrete piles, either embedment strain gauges, or so-called sister bars are used.

Sister bars are based on vibrating wire strain gauge technology, and are constructed from 150mm of debonded high-strength steel containing an axially mounted vibrating wire strain gauge. The instrumented section is attached at either end to reinforcing bar extensions which act as the bond length, and ensure strain compatibility of the debonded section with the surrounding pile. A schematic of a sister bar is shown in Figure 6.

Embedment strain gauges are also based on vibrating wire strain gauge technology, and comprise a tube with two closing end flanges or blocks to which the internal wire is attached (see Figure 7). The gauge is embedded in concrete – the end flanges ensuring strain compatibility.

Electrical resistance strain gauges are the only suitable alternative in cases where dynamic events are being recorded (e.g. Statnamic or dynamic load tests).

In lateral load tests, it may be of interest to monitor pile bending strains to compute bending stresses and moments. Strain gauges must be suitably located in the plane of bending about the neutral axis. Gauges will generally be located particularly in the upper part of the pile and at low embedments, where bending stresses will be maximum.

Instrumented Static Load Testing - Displacement measurement devices

In order to fully analyze the pile base stress-strain response, it is necessary to determine both the base load and the base movement. The base movement may be only approximately inferred from the known pile-top movement, and the strain measurements along the pile shaft. However, a direct measurement of base movement will be much more accurate. Typically, this is determined using so-called 'telltales', however, retrievable extensometers and fibre-optic measurement systems are now also available to measure displacements with great accuracy.

Telltales are used in both cast-in-situ piles and preformed driven piles. They comprise an anchor which is embedded in the pile base (or at some other depth of interest). An open steel or plastic pipe is attached to the anchor, and houses a stainless steel or fibreglass rod which can be clipped to (and unclipped from) the pipe base. The top of the rod moves in sympathy with the anchor, and can be monitored using a dial gauge, an LVDT or other displacement measurement device. An example application is shown in Figure 10.

Retrievable extensometer systems are available to provide multiple displacement measurements along the pile length. A special steel or plastic tube is cast into the pile, and a string of vibrating wire strain gauges with associated anchors is lowered into the pipe. The anchors are locked within the tube and the distance between the anchors can be progressively monitored with the vibrating wire strain gauges. The string can be removed on completion of the test, and reused on successive tests. The system components are shown in Figure 8, and a schematic of the system is shown in Figure 9.

Fibre optic sensors are a new displacement measurement technology with many great features, but currently provided at relatively high cost. These transducers can be provided in single lengths of between 200mm and 50m, and have a measurement accuracy of 2µm, regardless of sensor length. They are insensitive to temperature, electromagnetic fields, humidity, vibrations and corrosion, require no calibration and have excellent long-term stability. There are different specific fibre-optic technologies, each with different specific features and advantages. The cost of these systems probably precludes use on small and routine projects. A fibre optic sensor placed within a reinforcing cage is shown in Figure 11.

Instrumented Static Load Testing - Load measurement devices

Load cells which rely on calibrated strain measurement are typically use to measure pile head load. As for strain measurements in the pile, vibrating wire strain gauge or electrical resistance technology can be used for load cells. However, dynamic load cells must use electrical resistance strain gauges. Load cells must be matched in size to the pile head and the jack which is used to apply the load. Sellers (1994) notes that mismatches of jack and load cell size may cause load errors of up to 10%.

Manometers attached to the loading jack measure hydraulic pressure, which can be converted to equivalent load, depending on laboratory calibration. However, under field (as opposed to laboratory) conditions, the calibrations may be invalid and load overestimation in excess of 20% are reported (Fellenius, 1980). AS2159-1995 does not allow the use of manometers for load measurement.

Instrumented Static Load Testing - Measurement of inclination

Inclinometers may be embedded in piles in order to measure the deflected shape of piles under lateral test loads. Inclinometer probes track along proprietary grooved casings which maintain the orient the probe. The probe contains accelerometers which measure the inclination of the probe in two directions. Inclination measurements can be converted to lateral displacement by integration.

3.3 Bi-directional Load Testing

Bi-directional load testing is also known by the name of the device, the Osterberg cell or O-cell. This is a form of static load test which applies the load from the bottom of the pile, rather than from the pile head. The technique is particularly designed for testing of drilled cast-in-situ piles, such as rock-socketed piles, as well as non-circular elements such as barettes. However, some tests have been performed even on driven concrete piles.

In order to perform this test, a proprietary hydraulic pressure cell, or flat jack must be placed and bedded at the pile base. The cell is connected to the pile head with hydraulic lines to allow cell inflation, and instrumentation lines and conduits to permit measurement of the response.

The particular benefits of bi-directional load testing are that :

1. a cell with a given capacity can test piles to loads of up to twice that capacity, as the shaft is loaded in an upward direction at the same time as the base is loaded in a downward direction;.
2. the base response can be loaded and measured directly
3. the shaft resistance provides the reaction, and it may not be necessary to provide any further kentledge or reaction system;

Bi-directional load testing depends on there being sufficient shaft resistance available to provide the required reaction. It may therefore not be suitable for predominantly end-bearing piles. Conventional kentledge may be used in association with the test to increase the available reaction. In addition, care should be taken in the acceptance and interpretation of the results, as the shaft resistance is being mobilized in an upward direction and the cell expansion actually generates tensile stresses at the cell level. The stress regime around the pile base may therefore not be representative of normal service loading, but should provide a conservative assessment.

The following table shows the diameters and nominal capacities of one manufacturer's bi-directional load cells.

Nominal Diameter		Nominal Capacity *	
(in)	(mm)	(kips)	(MN)
9	230	400	1.80
13	330	800	3.60
21	540	2000	8.90
26	670	3600	16.0
34	870	6000	27.0
Note : The total test capacity is twice the nominal O-cell capacity			

Bi-directional load cells may also be located at locations other than the pile base. The shaft resistance in particular sections of the pile (e.g. a rock socket) can be determined by having cells located at preselected locations. Cells may also be combined in groups of two or more at the same depth, and inflated simultaneously in order to increase the available test load. By combination, test loads of up to 133MN have been achieved.

The test instrumentation includes :

- two telltales to measure movement of the lower surface of the cell on opposite sides
- an internal LVWDT (linear vibrating wire displacement transducer) or extensometer to measure the cell expansion
- a manometer to measure cell pressure, from which the applied load is derived
- pile-top movement measurements
- reference beam movements

A schematic showing a typical instrumentation arrangement for an O-cell test, including telltales, is shown in Figure 10.

Bi-directional load testing has not been undertaken in Australia to date. It must be performed by a specialist contractor who manufactures, supplies and installs the cells in conjunction with the foundation contractor. The same firm then performs and monitors the load test. The installation is extremely important, and must be undertaken to a tight specification by experienced personnel to ensure both the success of the test, as well as the integrity of the pile. Excessive delays in installation, or poor concreting using inappropriate mixes could compromise pile capacity. On completion of a test, the hydraulic oil in all cells is displaced by grout, so that the pile can be incorporated into the foundation as a structural element.

Bi-directional Load Testing – Summary of Issues

Features	**Requirements**
Pile loaded by flat-jack from base	No reaction system (generally)
Requires 50% of test load as reaction	Specialist contractor to supply and install (U.S., Singapore and Malaysia)
Loads up on shaft and down on base.	No reaction system (generally)
Cells available from 230mm (4.0MN test load) to 870mm (55MN test load)	
Advantages	**Disadvantages**
Direct (requires minimal interpretation)	Some of the disadvantages of static load tests
Separates load-deflection response of base and shaft	Generally only for bored piles
Can be installed at multiple levels	Test pile must be nominated in advance
Sometimes implemented with strain gauges	Installation of cell may affect construction
Uses shaft resistance as reaction	Fails at 2 x minimum of shaft resistance or end bearing
No interaction with reaction	Cell expansion produces tensile zone near base

3.4 Rapid Load Testing

Rapid load testing is more commonly known by its proprietary name – Statnamic® load testing. Statnamic derives its name from a combination of the words "static" and "dynamic". This is because it is a dynamic test of longer duration than PDA tests (see later). This tends to reduce the relative importance of dynamic effects, and make the pile move more (but not exactly) like a rigid body.

The test does not give a direct value of static capacity, or load-displacement response and some interpretation is necessary. Simplistic analysis techniques are available (e.g. the unloading point method), however, wave equation analysis is the author's preferred method of evaluation.

The dynamic event in a Rapid Load Testing is generated by launching a constructed mass above the pile using slow-burning explosive in a combustion chamber between the pile and the mass. The inertial force of the launched mass generates an equal and opposite reaction from the pile. A schematic of the Statnamic test, and some views of the test set-up are shown in Figures 12 and 13.

There are several distinct advantages of Statnamic testing, and interesting applications have emerged. The test is somewhat less expensive than static load testing, however, the test can be performed in a much shorter time (perhaps 1/3 of the time for an equivalent static test). The tests can be performed in both vertical and lateral directions, and tests have been conducted on whole pile groups. Offshore testing is another good application, as the seawater can be used as a reaction. Test rigs for loads up to 50MN are available, although the limit in Australia is currently 15MN. A relatively small number of tests have been conducted in Australia.

The technique requires specialized skill, both in the design of the explosive charge to be used, and in the construction and execution of the test. If the explosive charge is excessive, then large dynamic effects will be generated, and the unloading point method will not satisfactorily account for these effects. However, for more 'balanced' charges, when the pile-soil system either remains in the elastic range, or the imposed force does not exceed the ultimate geotechnical capacity, the dynamic effects will be suitably small, and simplistic analysis may be sufficient. Long piles will usually require a more rigorous wave-equation analysis.

Rapid Load Testing – Instrumentation

The applied load is measured directly by a load cell placed between the device and the pile head. This obviates the need for any assumption of pile modulus or area.

The pile-head movement is measured using a laser-activated photo-voltaic displacement transducer. Sometimes accelerometers are used as a back-up to the laser displacement transducer for direct determination of acceleration, which is relevant to the inertial component of the response.

Rapid Load Testing – Summary of Issues

Features	**Requirements**
Pile loaded by long duration combustion	Reaction system is launched by combustion
Requires 5-10% of test load as reaction	Requires special solid fuel, required charge is estimated
Device available in Australia to 15MN, overseas to 50MN	Specialist contractor to supply and install (Franki-Keller is the agent in Australia)
Slow dynamic test, usually with reduced dynamic effects (c.f. PDA), hence resistance more similar to static	Usually for larger diameter bored piles
Advantages	**Disadvantages**
Takes less space than static test (less reaction)	Not significantly cheaper than static load testing
Direct load-settlement output	One hit only before test must be re-setup*
Little correction required if pile "set" is low	Dynamic effects may be significant in some cases
Lower compression and tension stresses in pile compared to dynamic	Standard analysis method is quick but unsophisticated
Can do lateral load testing	No resistance distribution unless instrumented
Can test pile groups (load limit)	*some multi-hit devices available overseas

3.5 Dynamic Load Testing

Static load testing techniques are both expensive and time-consuming. As noted in the previous sections, these techniques have distinct advantages and benefits, but they are also not without their short-comings and difficulties.

Dynamic load testing techniques were developed as a rapid and relatively inexpensive alternative to static load testing in the late 1970's and early 1980's, and have been in use in Australia since 1982. Dynamic load testing, or PDA testing as it is more commonly known, is a prescribed method in AS2159-1995, and section

8.4 of that standard provides requirements for the equipment, procedures, acceptance, supervision, recording and reporting of these tests.

The primary purpose of dynamic pile testing is to determine the pile capacity. However, it will be seen in section 4 that dynamic pile testing also has (equally) important benefits in providing construction control for driven pile projects. Dynamic load testing was developed specifically for the testing of pre-formed driven piles, however, it can be, and has been applied to the testing of bored piles, CFA piles, and barettes. Such extensions are possible, but require a higher level of testing expertise and review, with a particular understanding of the critical assumptions.

The fundamental difference between static and dynamic testing methods is that the dynamic methods are conducted during pile driving, when the pile is in significant motion. At this time, the pile is subjected not only to static soil resistance forces, but also dynamic forces that result from the relative pile-soil motion. Static pile capacity is therefore not a direct output of the test. The fundamental challenge for this method is to isolate the static and dynamic components of driving resistance so that reliable estimates of static capacity can be predicted. The success of this task is highly dependent on the reliability of the models of static resistance and dynamic resistance used in the interpretation of the test measurements.

The analysis techniques for dynamic pile testing are all based on one-dimensional wave mechanics. When the pile driving hammer impacts, a stress-wave is generated which travels the length of the pile, and is reflected from any shaft resistance, from any change in cross-section, including damage, and from the pile toe. The reflected waves can be interpreted to determine resistance, and even the detailed distribution of shaft resistance along the pile length, and the component of toe resistance.

Approximate results are available in real-time using a closed-form analysis called the 'Case Method', which depends – sometimes highly – on a so-called damping factor, J, which is a function of soil type and a number of other factors. Case Method estimates provide a preliminary guide to pile capacity, and may be correlated against a more rigorous wave equation analysis called CAPWAP® or its equivalent TNOWAVE®. This desk-top PC analysis provides the most reliable estimate of total capacity, resistance distribution (by 1 metre segment), and a predicted load-settlement response. Given that the test is very rapid, this prediction does not include any estimate of creep movement.

The test has evident benefits in terms of cost and time. These can either result in lower project costs, or alternatively, and more reasonably, the testing cost can be spread over a large testing sample. In Australia, typically between 5 and 10% of piles on a driven pile project are tested using this method. International practice varies widely, with some countries testing as little as 0.5% (Korea), and other countries requiring testing of 25% (Sweden). The ability to test a significant sample – and 5 to 10% could be considered statistically significant – provides the designer and client with an increased level of confidence, which is another benefit of this type of test.

Two types of tests are undertaken – driving and restrike tests. Driving tests are during pile installation, whereas restrike tests are subsequent to pile installation (by hours, days, weeks or months). Generally restrike tests will provide the best estimate of long-term static capacity, as these will incorporate any time-dependent capacity changes (e.g. set-up or relaxation). Restrike tests can be nominated on any random pile after installation, depending on the installation records.

As noted, the test method does not provide a direct evaluation of static capacity, and this inevitably reduces the reliability of the test. This is reflected in the lower range of capacity reduction factors which are specified for dynamic pile tests (0.65 to 0.85) compared with static load tests (0.70 to 0.90). The difference is small, and some element of this small difference may be attributed to the greater overall level of confidence provided by the higher percentage of piles dynamically tested.

Dynamic Load Testing – Integrity Assessment

As indicated earlier, any change of cross-sectional area, or damage will generate a reflection to the incident compression wave from the hammer. A reduction in area (or modulus), or damage will generate a tension reflection. This can be clearly differentiated from the compressive reflections which are generated from shaft resistance. Based on the relative magnitude of the tensile reflection, and the timing of that reflection, both the severity and location of any damage can be inferred from PDA tests. An approximate estimate is provided in the field (designated the Beta factor), but wave equation analysis will provide a more reliable assessment of the damage feature.

Dynamic Load Testing - Instrumentation

The instrumentation required for dynamic pile tests comprises:

- re-usable bolt-on electrical resistance strain gauges (because of the dynamic nature of the strain record). The strain is converted to a force-time record;
- accelerometers to measure pile-head acceleration. This is then integrated to produce a velocity-time record. Different accelerometer technologies (piezoresistive, piezoelectric or capacitative) are used.

Figure 14 shows the instrumentation used for dynamic pile testing – in this case for offshore testing, with a duplicate set of gauges attached. Figure 15 shows a typical data acquisition and analysis unit used for dynamic pile testing.

In general, two strain transducers and two accelerometers are attached at least 1 pile diameter below the pile-head. Average measurements take account of any pile-bending induced by uneven hammer impacts. In some cases (e.g. for cast-in-situ piles and spiral-welded steel tubes), 4 strain transducers are usually required to ensure reasonable data quality.

Dynamic Load Testing – Summary of Issues

Features	**Requirements**
Pile loaded by short duration impact	Field computer with data acquisition for strain and acceleration measurements
Requires 1-2% of test load as impact weight (reaction)	Specialist testing house (or contractor in-house)
Test load using normal driving hammer or purpose built for cast-in-situ piles	Computer wave equation analysis program
Measurement of stress wave input and response of pile	Usually for driven piles, but can be extended to cast-in-situ piles
Advantages	**Disadvantages**
Rapid	Output indirect (force and velocity vs time)
Cheap	"Black box" to most engineers
Extensive (site variations, statistical approach)	Static behaviour is interpreted from dynamic response
Evaluate driving hammer	Requires experience and skill to evaluate
Evaluate pile stresses	Requires special care for testing of cast-in-situ piles
Evaluate pile condition	Can be manipulated if procedures not in place
Evaluate changes with time	
Evaluate resistance distribution (feedback)	
Direct link to Hiley	

4. Evaluation of pile integrity – Test options and Instrumentation

Physical coring provides direct evidence of defective concrete or construction. Coring is a direct physical testing method which may allow indirect confirmation of satisfactory pile performance based on a structural assessment only. No reliable evaluation of the geotechnical performance should be inferred from any of these tests. The core may be used to assess concrete quality by visual inspection, strength testing or chemical analysis. However, it must be remembered that the retrieved core represents only a small statistical sample of the pile cross-section.

Due to these limitations, researchers have developed a number of indirect techniques for assessment of pile construction since the late 1960's. These indirect techniques are all based on the evaluation of small-strain wave transmissions or reflections. A variety of indirect testing methods exist, and these are well summarized in Turner (1997).

The applications, limitations, advantages and disadvantages of each indirect testing technique differ, and will be summarized later. However, the following general points should be taken into account for indirect testing:

- indirect testing may provide a qualitative rather than strictly quantitative assessment of pile shape, integrity and condition;
- indirect testing is often applied to 100% of all piles on a project. Anomalous piles are identified by differences in response to a reference response determined for the majority of the pile population;
- assessments (of pile length or geometry or condition) are based on interpretation of acoustic wave transmissions or reflections.
- the nature of the anomaly that has generated the received reflection or modified the received transmission can only be inferred.
- the significance of anomalies is generally related to the size of the reflection or transmission effect, however, factors such as soil resistance may significantly affect the response.
- acoustic methods are limited in their sensitivity – for instance, vibration testing may be unable to detect defects smaller than 10 to 15% of the pile cross-section (the threshold value), even under ideal circumstances.
- anomalies inferred from measurement of responses in excess of the threshold value should be confirmed by physical testing (e.g. coring).
- interpretation of defect locations, or pile lengths is based on an assumed wavespeed. If the pile length is known, then it may be possible to back-calculate the wavespeed. Wavespeeds in piles will vary with method of transmission (1-dimensional or 3-dimensional), strain level, concrete strength and concrete age.

4.1 Cross-hole sonic logging

This test is referred to as the Sonic Logging Test (SLT) is AS2159-1995. The test is largely unknown in Australia, but is extensively employed overseas for large diameter bored piers.

Cross-hole testing (or sonic logging or sonic coring) is a technique which is used for the evaluation of construction defects in newly cast drilled shaft piles. This technique has the advantage of not being limited in depth of application. The test is performed by simultaneously lowering an ultrasonic transmitter and separate receiver down two tubes cast into the pile during original construction. Generally 4 tubes (often more for larger diameter piles) are cast into the pile by attaching them to the reinforcing cage which is lowered into the pile prior to concreting. The reinforcing cage serves as a rigid skeleton to which to attach the tubes so that the spacing between the tubes is fixed. The principle of operation is shown schematically in Figure 16. With 3 tubes embedded in the pile, a total of 3 unique ray paths can be evaluated, whereas with 4 tubes embedded in the pile, a total of 6 unique ray paths can be evaluated.

The transmitter emits a pulsed sinusoidal wave train with a predetermined frequency which corresponds to a wavelength of between 50 and 100mm. The receiver responds to the arrival of the wave train, by oscillating at the same frequency. Typical probes are shown in Figure 17. As the two probes are progressively lowered (or raised) within a pair of tubes, a continuous profile of the receiver response can be plotted. The positive and negative cycles of the oscillation are traditionally recorded as "waterfall" diagrams which are 1-bit (1 and 0, or black/white) diagrams showing these positive and negative components of the received sinusoidal wave train.

Two parameters of particular importance can be interpreted from the test results. Traditionally, and most simply, the time of first wave arrival (the FAT or first arrival time) can be computed. This is a simple matter of scaling the first response off the waterfall plots. Knowing the distance between the tubes, the FAT information can be used to evaluate the speed of the wave transmission, which exceeds the speed of one-dimensional compression waves generated in (1-dimensional) vibration testing. Any defective concrete or anomaly which exists between two tubes will either delay the wave (by causing the wave to travel a greater distance around the anomaly), or entirely block the wave from being received. England (1991) in a private communication reported in Turner (1997) recommends that only variations in transit time of more than 15 to 20% of the norm for the site should be further investigated. This recommendation seems to be widely adopted in the industry.

Although waterfall diagrams are still used, more recently developed sonic logging equipment systems generally have enhanced capabilities. The computation of received energy contained in the arriving wave requires more sophisticated electronics with greater sensitivity and resolution, as well as more advanced computational routines. Later equipment allows interpretative information, including FAT logs and Energy logs to be plotted. These plots include some filtering algorithms which can assist in revealing significant features. The additional interpretation of received energy, and wave amplitude can further assist in the interpretation of anomalies. Figure 18 is a screen shot showing plots of FAT, relative energy and the traditional "waterfall" diagram.

Turner (1997) describes anomalies such as soil inclusions, wash-out, bentonite, and honeycombing as the type of effects which may be detected by sonic logging. Of course, the effect of such defects will be a function of the volume of concrete affected. The technique does not necessarily identify all defects, especially defects which may be on the outer perimeter of the pile, and therefore not delay the transmission of the wave between the tubes, nor significantly reduce the energy transmission. Although such defects may not represent a large percentage of the cross-sectional area, they may be significant to the long-term durability of the pile if reinforcement is exposed or the cover reduced. For example, if the bores exhibited overbreak due to loss of stability of the borehole wall during excavation, this would not be detected by sonic logging. If any wall collapse caused intrusion of soil into the bore during concreting, but the intrusion did not impinge inside the reinforcing cage, this again may be undetectable by this system.

Problems can arise if the sounding tubes are not well coupled with the concrete (e.g. due to smear or local air voids), and movement of the geophone within the sounding tube could also cause variations in response. As with all indirect measurement systems, errors are possible due to equipment, analysis and interpretation problems, and only experienced personnel with well-maintained and reliable equipment should be used. Sonic coring tests are intrinsically indirect and there are no simple criteria to 'pass' or 'fail' piles on the basis of these tests alone. The sonic coring technique, however, provides a cost-effective screening test to identify piles which have imperfections within their acoustic integrity that may have some structural significance. Such piles normally warrant further investigation and engineering evaluation. It is recommended that the test results should be evaluated in conjunction with pile construction records and site investigation reports which can often indicate the possible causes and physical nature of acoustical irregularities.

Despite these limitations, sonic logging has the significant advantage of being able to test or assess a much larger percentage of the pile volume than is assessed by direct physical coring. Furthermore, under normal circumstances, sonic logging can confirm the length of pile cast.

4.2 Pulse Echo and Impulse Response Methods

The pulse echo (PIT) test and the impulse response method are physically similar NDT test methods, which have different associated analysis and presentation techniques. These test techniques are denoted SIT and SVT in AS2159-1995. These techniques are typically applied to cast-in-situ piles, particularly CFA piles, and are both simple and fast one-man operations. It is recommended that these techniques be applied to every pile on the site. Construction problems may be evidenced by differences from the underlying signature response of the pile population.

Both techniques are applied at the pile head by impacting a small hand-held hammer on a clean and sound surface. Some head preparation is necessary, however, this is usually minimal and may involve only spot grinding. The pile head response is measured using either an extremely sensitive accelerometer, or a geophone (induction coil). A schematic of the test arrangement is shown in Figure 19. It is noted that the impulse response method must use an instrumented hammer, whereas the use of an instrumented hammer is optional with the pulse echo method.

The pulse echo test is interpreted in the time domain, in much the same way as high-strain dynamic tests. Pile necks or defects reflect a tension response, whereas pile bulges reflect in compression (see Figure 20). Due to the effects of soil damping, the low strain signal returning from depth must be amplified, and exponential digital amplification techniques are used to enhance the signal and allow features at depth to become more prominent. The technique is semi-quantitative, and there is a risk that with injudicious data manipulation non-existent features can either be "created", or alternatively real features can be suppressed. Strict guidelines should be followed with respect to data acquisition, analysis and presentation, and independent review of the electronic data is recommended.

The impulse response method differs in that the response is interpreted in the "frequency domain" (see Figure 21). The nature and location of features are determined by examining the dominant harmonic and sub-harmonic frequencies of the response. The effect of any significant anomaly in the pile shaft will be to superimpose additional dominant modes of vibration, which will be a function of the length to the defect, and the nature of the anomaly (e.g. an increase or decrease in section).

As indicated, the nature of the physical feature which generates the acoustic reflection can only be inferred. Some knowledge of the construction or service history may assist in interpretation of the physical defect. In many cases, visual inspection – e.g. by coring – may be required to confirm the true nature of the defect. It should also be noted that the acoustic response represents an average section response, and a reduction in impedance due to loss of section in the heart of the pile, or by necking, or by reduction in material quality may be indistinguishable. These techniques are generally considered to be sensitive to changes in acoustic impedance of 15% or greater.

Pulse echo and impulse response methods may be applied in the following applications :

- Evaluation of pile lengths (timber, concrete or steel)
- Detection of defects, and partial or total loss of pile section
- Comparative evaluation of relative condition for piles in a group
- Comparative evaluation of pile-head stiffness for piles in a group
- Estimation of depth to stiff soil layers

Vibration and impulse response methods have the following limitations:

- Generally penetration of the wave, and hence evaluation of pile length and condition, is limited to length/diameter ratios of between 20 and 50 depending on the pile and soil conditions, due to attenuation of the impact signal by soil damping.
- The transmitted signal will be further diminished by any changes of impedance along the pile length which cause reflections (of energy). This further limits the ability of the method to discriminate features at depth.
- Vibration and impulse response methods are able to discriminate changes in pile impedance of 10-15% at best. It would be expected that the ability to detect anomalies decreases with depth because of attenuation of the signal.
- Vibration and impulse response methods may only be able to reliably detect features with an axial length of 0.8m to 1.0m because the pulse length of the input wave is 3 to 4m. A sinusoidal frequency of 2000Hz corresponds to a wave length of about 2m. By use of smaller hammers or higher frequencies, pulse/wave lengths may be reduced, and the length of discrimination reduced, however, the penetration of such short duration pulses may be poor.
- Vibration and impulse response methods may not be able to detect gradual changes in pile impedance
- The condition of the upper section of a pile may not be reliably determined, particularly if the pile has a large diameter. This is because in the upper section, the wave is radiating from a point contact to the full lateral extent of the pile, and does not travel as a plane wave in the upper section. The average mobility, N, may be used as a measure to estimate pile-head condition.
- Detection of the pile toe may be difficult if it is located in material with similar acoustic impedance.

- These tests do not provide information on pile capacity. Vibration and impulse response tests can be used to determine the comparative dynamic pile-head stiffness, which can possibly be used as a relative guide to selection of a pile for static load testing. Caution must be exercised against over-interpretation of the low-strain dynamic pile head stiffness values determined by transient response testing.
- Varying ground conditions can generate secondary reflections which may confuse the interpretation of these tests. Ellway (1987) suggests a 1:5 reduction in shear modulus (from upper to lower layer) will result in a complete reflection of the incident wave.
- Vibration and impulse response tests should as far as possible be applied and interpreted on the basis of the response of piles relative to the average response of the pile population. Because of the interaction of section, material and soil effects, caution must be exercised in absolute evaluations of single piles without reference to tests on other piles.

4.3 Parallel seismic testing

The parallel seismic test is an acoustic method based on measuring the effect of an impact on, near or above the head of a foundation in a borehole which has been drilled adjacent and approximately parallel to the foundation. The system set-up is shown schematically in Figure 22. A small hand-held hammer, similar to those used for seismic echo and impulse refraction energy tests imparts a quantum of energy. The impact may be on the top or side of the pile, if the pile is exposed. However, if the pile is buried or built into a cap, the impact may be on the cap or a structural component connected to the cap. It is only necessary that there be a path which allows energy transmission from the point of impact to the pile.

The impacting hammer must be equipped with a trigger which responds to the impact, and fixes the time of impact, and start of data recording. The data recording device may be any analogue or digital system, but a digital recording system would be most common and allow most rapid analysis.

The energy that is imparted to the system will travel through the structural foundation components, and is also radiated from the foundation elements through the surrounding soil. The speed of travel in the structural components will be significantly higher than the speed of travel in the soil because of the much higher modulus of the structural materials.

The borehole drilled parallel and adjacent to the foundation element (e.g. pile) to be evaluated is filled with water, and a receiver is positioned within the borehole. The water serves to "couple" the receiver to the soil medium, and to detect the pressure wave generated in the water. Olson et.al. (1998) discusses the use of an alternative set-up comprising geophones fixed to the inside of a PVC casing grouted within the borehole using bentonite or bentonite-sand, or back-filled with sand.

The first wave to arrive at the receiver after impact will take the wave which travels fastest from the point of impact to the receiver. Because of the faster transmission in the structural components, the path of the earliest wave will be one which maximizes travel in the structure and minimizes travel in the soil. This wave will effectively travel in the structure to the elevation of the receiver, and then move horizontally through the soil to the receiver. Later waves reach the receiver by less favourable paths. By repeating the test as the receiver is progressively raised from the base of the borehole by constant increments, the variation of the geophone response can be plotted as a function of time and location. A typical set of test responses is shown in Figure 23.

When the receiver is above the base of the structural element (e.g. pile toe), the time of arrival will reflect the additional travel length in the structural material. However, as the receiver is lowered below the pile toe, the increase in travel time will be a function of the increasing travel length in the soil from the pile toe to the receiver. This is evident as an abrupt change in the gradient of the interpreted line of first arrival times (see line signal received in Figure A18). The depth of the pile toe can be inferred from the depth at which there is a change in the gradient of the first-arrival time. The angle of refraction of the waves at the pile/soil interface will result in a slight overestimation of the foundation length. Interpretation of the technique relies on a significant differentiation of the stresswave speed in the pile and the surrounding ground. In general, there will be sufficient difference between pile and soil to allow effective interpretation of the parallel seismic test. However, the length of piles that are socketed or embedded in rock may be difficult to establish by this method.

4.4 Other integrity testing methods

- Bending Wave method
- Ultraseismic method
- High-strain dynamic pile testing (PDA) – see section 3.
- SASW (Spectral analysis of surface waves)
- Ground Penetrating Radar
- Borehole Radar
- Induction Field Test
- Seismic Tomography
- Nuclear Radiation Methods, including gamma ray
- Resistivity methods
- CCTV inspection

Low strain integrity testing – Summary of Issues

Features	**Requirements**
Used for evaluation of shape length and condition of new or existing foundations	Field computer with data acquisition for acceleration measurements
Range of technologies all based on small strain waves	Specialist testing house (or contractor in-house)
No single technique covers all cases	Generally some associated analysis program
Can establish pile length to +/- 5% in the right conditions	Usually for cast-in-situ piles, but also for driven (timber bridge or wharf) piles and other poles
Sensitive to changes in cross-section of about 15 to 20%	
Technique should be targeted to problem on a case-by-case basis	
Advantages	**Disadvantages**
Generally low cost	SE and IR techniques have limited penetration
Very fast	Requires expertise to interpret
Portable equipment	Indications of problems may be relative (to reference response) rather than absolute
SE and IR methods require minimal preparation	Problems should be confirmed by other evidence, e.g. coring
Rapid way to screen 100% of all piles for more detailed evaluation, as needed.	Beware over-interpretation!
	DOES NOT provide an estimate of capacity

5. Construction Control – Equipment and instrumentation options

The previous sections have described a range of tests for compliance testing with respect to pile capacity, load-deflection response and pile integrity. As important as these are to a foundation contract, a true quality assurance approach should require the following two elements :

1. Experienced and capable construction personnel and supervisory staff.
2. Monitoring and evaluation during the construction process.

The first requirement is fundamental, and sadly the expertise and experience of supervisory field staff seems to be in decline rather than on the ascendancy. Furthermore, with less static load testing being undertaken on cast-in-situ piles in particular, the valuable opportunities for field staff to receive regular feedback on the quality of the piles they produce are missing.

In the absence of these feedback opportunities, the development of technologies provide significant opportunities for objective measurements to be made during the construction process.

It is absolutely fundamental that early monitoring and evaluation be undertaken in order to provide immediate warning of any anomalies at the site, and to allow corrective action to be taken before the consequences to the foundation, the contract and the project program are magnified out of proportion.

The particular type of monitoring depends primarily on the type of foundation being installed. Some available systems will be discussed in the following parts :

5.1 Driven piles

Dynamic Pile Testing

Dynamic pile testing (PDA testing) has been discussed in sections 3 and 4 in the context of determination of pile capacity, load-deflection response and pile integrity. PDA testing can also provide a very effective construction control function, if properly considered and built in to the pile driving contract. This function is as important as its ability to estimate pile capacity, but is generally overlooked. Dynamic pile testing can measure or estimate the following parameters :

- maximum pile head compression stress (average and maximum bending);
- maximum average stress at the pile toe
- maximum section tensile stress along the pile shaft
- effective energy transfer from the hammer to the pile, and hence hammer efficiency
- pile damage and location
- pile set and temporary compression (to reference to site physical measurements)

By effective monitoring of piles during the complete installation process, it is possible to develop a comprehensive driving plan which will maximize the chance for the pile to be driven safely but efficiently. Efficiency is important for the contractor, but the need to ensure that the pile that reaches founding level has not been overstressed, and will therefore be durable for the life of the structure, is paramount.

The results of monitoring and capacity evaluation must also be effectively connected to the Hiley formula to ensure that a reasonable pile acceptance criterion is developed.

Monitoring must be conducted regularly throughout the contract to ensure that variations in hammer performance over time, as well as stratigraphic variations across the site are properly accounted for, and reflected, if necessary, in the acceptance criteria.

Dynamic pile testing equipment provides a complete approach to quality assurance and construction control for driven piles of all kinds.

5.2 Bored piles/drilled shafts

There are two particular aspects of bored pile construction which should be monitored to ensure adequate performance – the condition of the pile base and the condition and properties of the shaft or socket, if the pile is drilled to rock.

Socket Inspection Device (SID)

Drilled shafts may be designed to carry their load by a combination of shaft resistance and end bearing. If taken to reasonable quality rock, the end bearing may be a very significant component. However, if the construction control is poor, leaving a lot of debris at the base before and after concreting, the available end bearing may be extremely low, and may even be non-existent at deflections which are compatible with the structural performance. Ensuring base cleanliness is therefore an important aspect of bored pile construction, especially to rock.

The socket inspection device (SID) was developed by the Country Roads Board (now VicRoads) for the Westgate Freeway project. SID is an inspection bell which houses a high resolution video camera and is used to inspect the bottom cleanliness of drilled shafts prior to placement of concrete. The inspection bell is lowered from a service platform to the bottom of the shaft, and the operator can view the condition of the bottom via the camera. The bell is fitted with a depth gauge to indicate the thickness of debris on the shaft bottom. The SID also has the capability to sample the sidewalls of shafts in soil in order to evaluate the buildup of slurry along the sidewalls. A photo of the ground level equipment of the SID device is shown in Figure 24.

Shaft Callipering

This is a virtually unknown technique in Australia. Callipering is undertaken in order to establish the gross shape of the borehole prior to concreting. This is used to ensure that the borehole has not experienced any collapse which will affect the concreting process and the interpretation of low strain tests and may affect the performance of the pile.

Callipering can be undertaken by mechanical means, or more commonly and accurately by acoustic or sonic methods. Acoustic callipering must be performed in fluid, and will require calibration depending on whether the fluid is water, natural or synthetic drilling muds.

Socket Roughness Measurement

The capacity and load-deflection response of piles socketed into rock, particularly with long sockets is significantly dependent on the socket shaft resistance. The available resistance is a function not only of rock strength, but also rock jointing and socket roughness. The design method developed at Monash University for rock-socketed piles (Rocket) uses these parameters, including the socket roughness as program inputs.

The SocketPro device allows socket roughness to be accurately measured using laser profilometry. This equipment is shown in Figure 25.

5.3 Continuous flight auger (CFA) piles

The construction of continuous flight auger (CFA) piles requires a high degree of operator expertise. The success of the operation is dependent both on the torque, the rate of auger advancement and rotation during the initial drilling phase and the rate of lifting, concrete supply and pressure during extraction.

Because of the strong need for quality control of this type of piling, several companies manufacture equipment for the monitoring and control of CFA rigs, and a range of this equipment is used by some Australian foundation contractors.

The torque, the rates of rotation and the rates of advancement and extraction can be easily and reliably measured. Most systems measure these parameters.

By contrast, the rate of concrete supply, and the supply pressure are very difficult to monitor. Ideally, these parameters should be known at the auger tip. However, reliable measurements at this location have eluded all manufacturers to date. Concrete supply is measured either by measuring the number of pump strokes, and multiplying this by an assumed volume per stroke. The supply rate may be in error because of an incorrect calibration, 'short' pump strokes, or because of discontinuities in the line which mean that the volume delivered at the auger head is not the same as that measured at the pump. Alternatively, magnetic flow meters may be fitted in series behind the pump, and these can be calibrated against concrete or grout volume. Varying mix properties may affect the accuracy of the volume measurement, and discontinuities in the line may still make the delivered volume in error.

The concrete pressure is typically measured at the top of the 'gooseneck', as reliable measurement at the pile tip is complicated by the severe wear and tear at this location. The pressure at the pile tip should be in excess of the measurement at the gooseneck by the effective head of concrete/grout, assuming a full concrete/grout column. The effective head of concrete/grout is difficult to assess because of losses in the auger stem, and arching effects in the relatively small diameter tube. Arching will depend on the fluidity of the concrete. If the column of concrete/grout is not maintained, the pressure at the pile tip may bear no relationship to the pressure measured at the gooseneck.

Recognizing the fundamental problems with these measurements, one manufacturer has developed an alternative strategy for ensuring the integrity of the concreting process. The critical requirement for ensuring a sound shaft is that the tip be under positive pressure at all times. Assuming that the concrete is 'sealed' at the bottom of the auger, a positive concrete pressure will act to reduce the effective weight of the auger loaded with soil. The technique is based on a process of measuring this effective weight at regular intervals and during reverse auger rotation.

5.4 Screw piles

Steel screw piles have become a popular piling system for both domestic and commercial construction during the past few years. For some of these systems, the capacity is determined using empirically derived charts which relate installation torque to compression capacity. Having undertaken some experimental and analytical research on screwed-in piling, it is my view that torque is a poor predictor of compression capacity. As a consequence, any correlation should be demonstrated by sufficient correlations with static load testing in accordance with AS2159-1995. Such correlations should be auger-specific and either site- or geology-specific. Any change in auger dimension, flight angle or construction would require new correlations.

5.5 Pressed-in piling

Pressed in piling systems have also become available in Australia as an alternative to conventional piling systems. These systems were developed in China where there is significant experience of this type of pile. These systems are attractive with respect to minimization of the noise and vibration effects normally associated with precast concrete piling. In theory, pressing rather than driving a pile will also minimize stresses, just as Statnamic® testing will generate lower stresses than dynamic testing.

With this system, piles are pushed into the ground at a slow rate using grips powered by high capacity hydraulic jacks. The pile capacity is determined by manometer connected to the jacks. No specific deflection measurement is made, but 'refusal' is estimated visually.

As discussed in Section 3 with respect to static load testing, the use of manometers to measure jack load is not recommended. Load cells are required under AS2159-1995 for static load tests. Furthermore, the draft Chinese code on pressed-in piling requires such piles to be given up to 5 repeated cycles of load before the capacity can be assumed to be stabilized. In addition, the effective static capacity may be reduced, depending on the pile length, with short piles having the greatest reduction factor applied.

6. Foundation System Verification

Sections 3 to 5 have discussed techniques that are available to test and monitor individual piles. Where a piled foundation system comprises single piles acting essentially independently (i.e. at sufficient spacing), the performance of the system as a whole may be inferred directly from the testing of individual piles. The designer's confidence of the whole foundation system will be a function of :

- the performance of those piles tested, and the variability of the performance;

- the number of piles tested – both as an absolute number and as a percentage of the total pile population;
- the reliability of the test method(s) used;
- the representativeness of the piles tested;
- the variability of the ground conditions;
- the redundancy (or otherwise) of the system;
- the consequence of failure.

These factors may be incorporated into a statistically based analysis to ensure a satisfactory degree of confidence.

If the piled foundation system comprises piles which interact significantly as a group, further account must be taken of the effect of such group interaction on both the serviceability and capacity of the pile group. In general, but not always, deflections under service load will increase due to group effects, and ultimate capacity will be reduced relative to the capacity of the same number of piles acting individually.

It is always important to establish whether the group action may make critical a mechanism of failure which is not critical during the loading of an individual pile. Uplift capacity of piles in jointed rock, or the behaviour of pile groups founded on discrete layers underlain by softer materials would be two examples.

Extrapolation of group effects, and consideration of different failure mechanisms must be based on classical geotechnical analysis, hopefully based on a rigorous site investigation.

7. Conclusions

There are many aspects of foundation construction, construction control and verification that require compliance testing and instrumentation. The wide range of compliance testing and instrumentation have been highlighted in this paper, and the main features and requirements, advantages and disadvantages have been summarized.

It is important that in designing the quality assurance and testing regime for a foundation project, the functional requirements of the structure be considered, and the ground risk be understood in the context of the foundation type that is proposed.

By combination of the available techniques with the structure needs and the construction risk, an effective and co-ordinated compliance testing program can be developed to cover the issues of construction control, pile integrity, serviceability and pile capacity.

As this is a specialized area, guidance should be sought with respect to the development, execution and interpretation of the testing program.

8. Disclaimer

The opinions expressed in this paper are those of the author, and do not necessarily reflect those of Monash University.

9. Acknowledgments

The kind assistance of Mr. Slav Tchepak of Vibropile, Mr. Barrie Sellers of Geokon, and Mr. Jack Hayes of Loadtest Inc. in the preparation of this paper, are gratefully acknowledged.

10. References

AS2159 (1995). Australian Standard. Piling – Design and Installation. Standards Australia. ISBN 0 7262 9884 0.
Ellway, K. (1987). Practical guidance on the use of integrity tests for the quality control of cast-in-situ piles. Ground Engineering, Vol. 20, No. 7, October : 8 – 13.

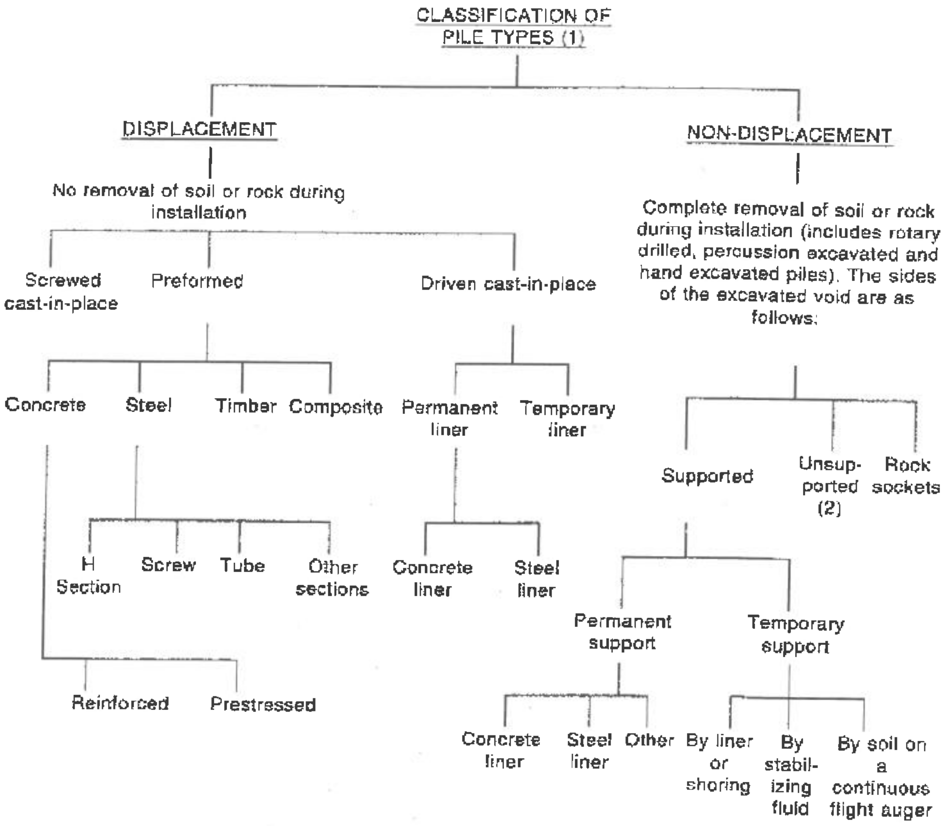
Fellenius, B.H. (1980). The analysis of results from routine load tests. Ground Engineering, September 1980 : 19-31.

Sellers, J.B. (1994). Load Cell Calibrations. Geotechnical News – September 1994 – BiTech Publishers.

Sellers, J.B. (1995). Pile load test instrumentation. Seminar on Instrumentation in Geotechnical Engineering, Geotechnical Division of the Hong Kong Institution of Engineers, May, 1995.

Turner, M.J. (1997). Integrity Testing in Piling Practice. Report No. 144 of the Construction Industry Research and Information Association (CIRIA). ISBN 086017 473 5. 336pp.

Olson L.D., Jalinoos F, and Aouad M. F. (1998). "Determination of Unknown Subsurface Bridge Foundations". A Summary of the NCHRP 21-5 Interim Report. US Department of Transportation. Federal Highway Administration. Geotechnical Engineering Notebook. Geotechnical Guideline No 16.



NOTES:

1 Pile types for which there is no established experience may not fall into these categories.

2 Unsupported. This classification refers to piles in which the ground is left exposed during excavation.

Figure 1 – after AS2159-1995 Figure 1.1 Classification of Pile Types

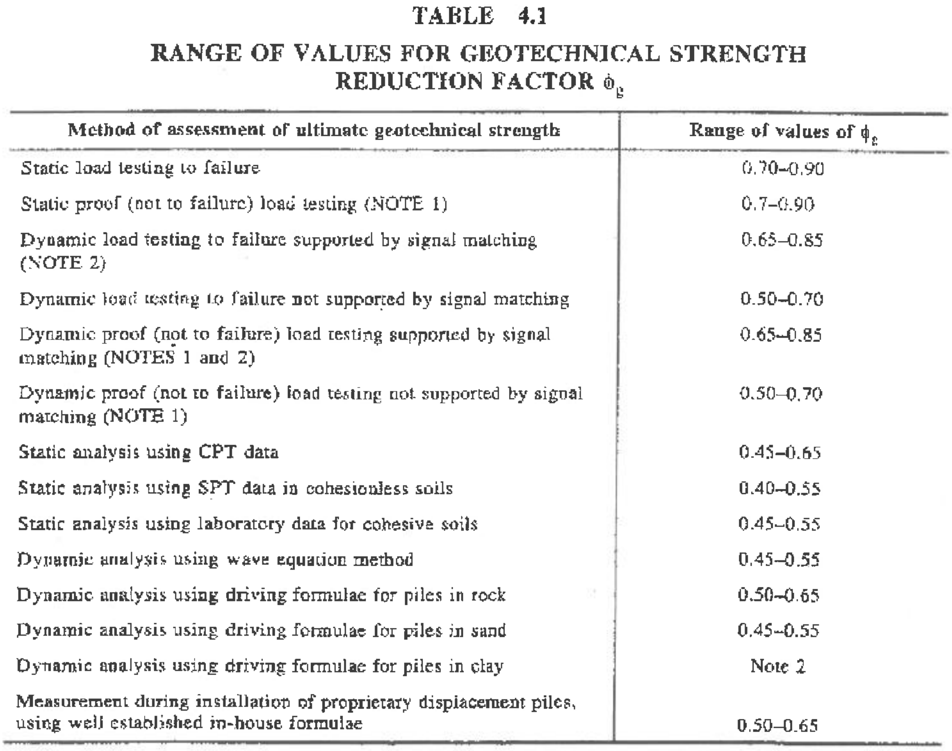


TABLE 4.1

RANGE OF VALUES FOR GEOTECHNICAL STRENGTH REDUCTION FACTOR ϕ_g

Method of assessment of ultimate geotechnical strength	Range of values of ϕ_g
Static load testing to failure	0.70–0.90
Static proof (not to failure) load testing (NOTE 1)	0.7–0.90
Dynamic load testing to failure supported by signal matching (NOTE 2)	0.65–0.85
Dynamic load testing to failure not supported by signal matching	0.50–0.70
Dynamic proof (not to failure) load testing supported by signal matching (NOTES 1 and 2)	0.65–0.85
Dynamic proof (not to failure) load testing not supported by signal matching (NOTE 1)	0.50–0.70
Static analysis using CPT data	0.45–0.65
Static analysis using SPT data in cohesionless soils	0.40–0.55
Static analysis using laboratory data for cohesive soils	0.45–0.55
Dynamic analysis using wave equation method	0.45–0.55
Dynamic analysis using driving formulae for piles in rock	0.50–0.65
Dynamic analysis using driving formulae for piles in sand	0.45–0.55
Dynamic analysis using driving formulae for piles in clay	Note 2
Measurement during installation of proprietary displacement piles, using well established in-house formulae	0.50–0.65

Figure 2 : AS2159-1995 Table 4.1. Range of values for geotechnical strength reduction factor, ϕ_g

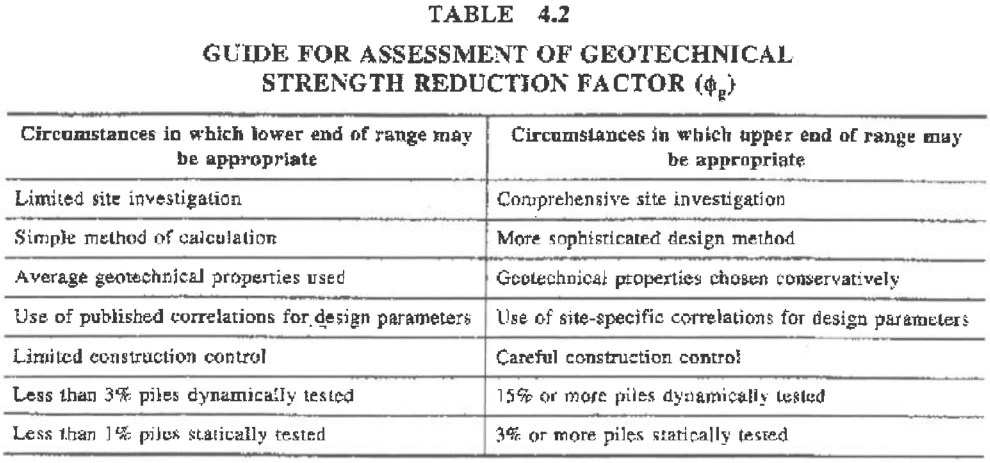


TABLE 4.2

GUIDE FOR ASSESSMENT OF GEOTECHNICAL STRENGTH REDUCTION FACTOR (ϕ_g)

Circumstances in which lower end of range may be appropriate	Circumstances in which upper end of range may be appropriate
Limited site investigation	Comprehensive site investigation
Simple method of calculation	More sophisticated design method
Average geotechnical properties used	Geotechnical properties chosen conservatively
Use of published correlations for design parameters	Use of site-specific correlations for design parameters
Limited construction control	Careful construction control
Less than 3% piles dynamically tested	15% or more piles dynamically tested
Less than 1% piles statically tested	3% or more piles statically tested

Figure 3 : AS2159-1995 Table 4.2. Guide for assessment of geotechnical strength reduction factor, (ϕ_g)

Figure 4 (a) – static load test using rock anchors for reaction

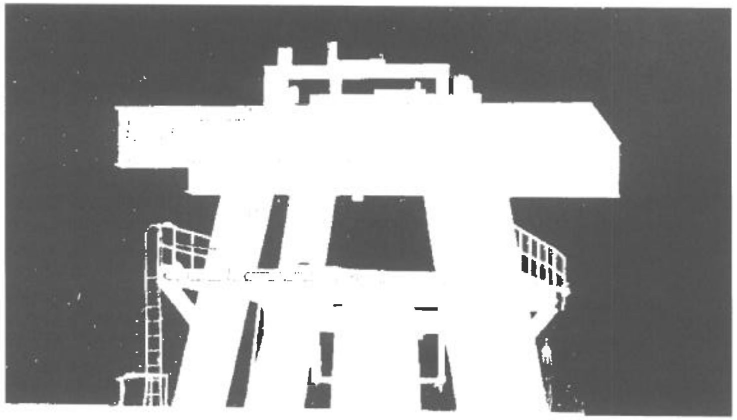


Figure 4 (b) – static load test using uplift piles for reaction

Figure 4 (c) – static load test using kentledge (concrete blocks) for reaction

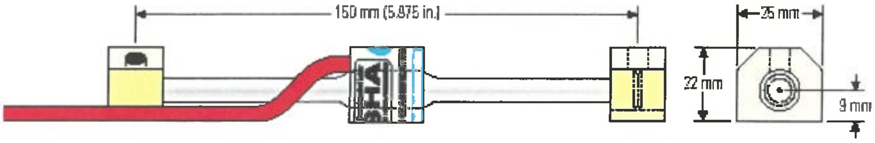


Figure 5. Schematic of a weldable vibrating wire strain gauge

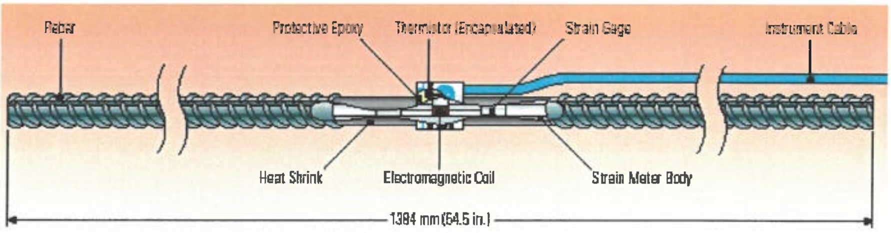


Figure 6. Schematic of a 'sister bar' vibrating wire strain gauge

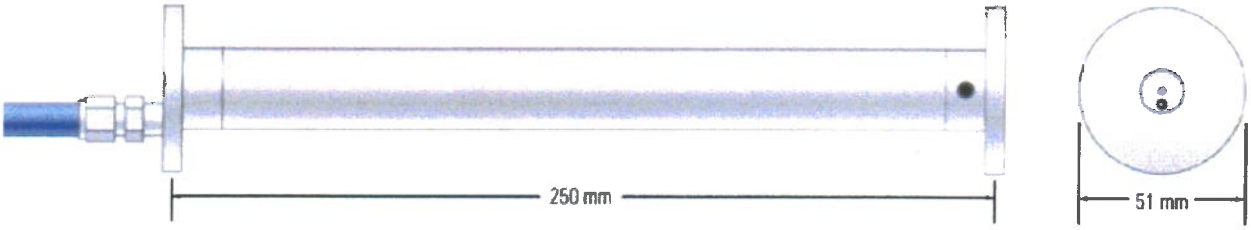


Figure 7. Schematic of a concrete embedment vibrating wire strain gauge

Figure 8. Components of a retrievable extensometer

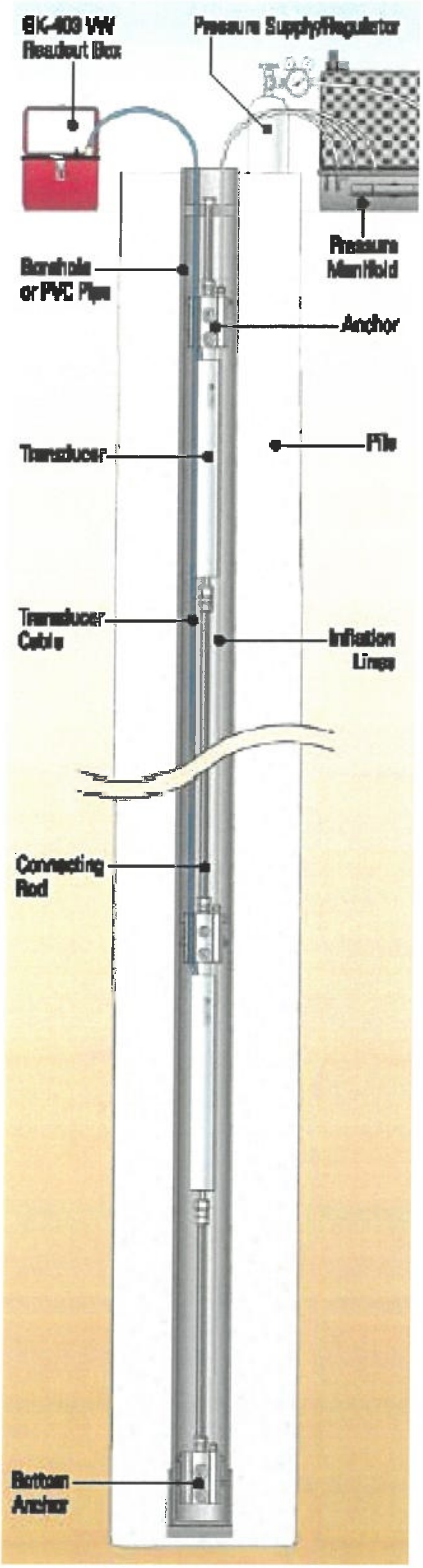


Figure 9. Schematic of retrievable extensometer

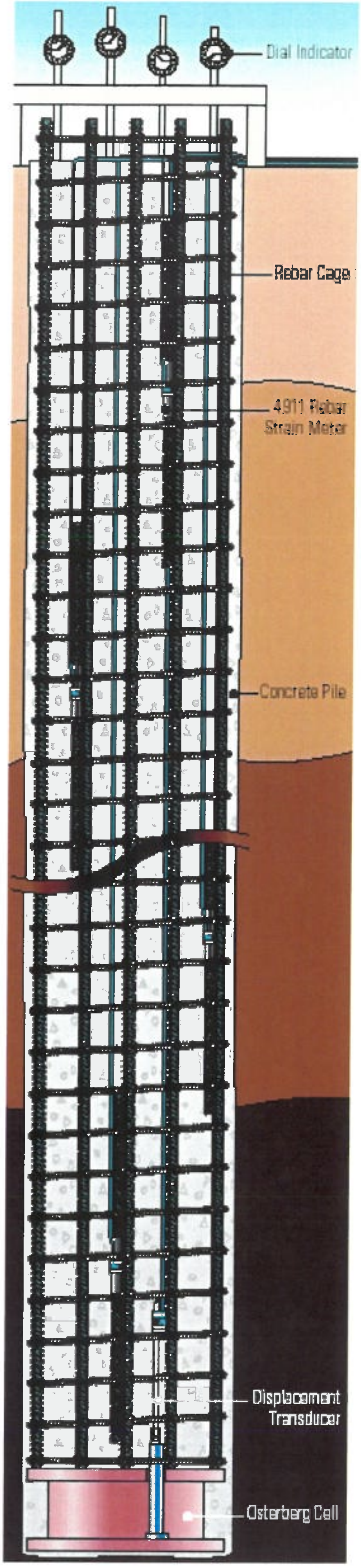


Figure 10. Schematic of O-cell test with use of telltales

Figure 11. Fibre optic sensor placed along reinforcing bar.

Figure 12. Two views of O-cells for bi-directional load tests

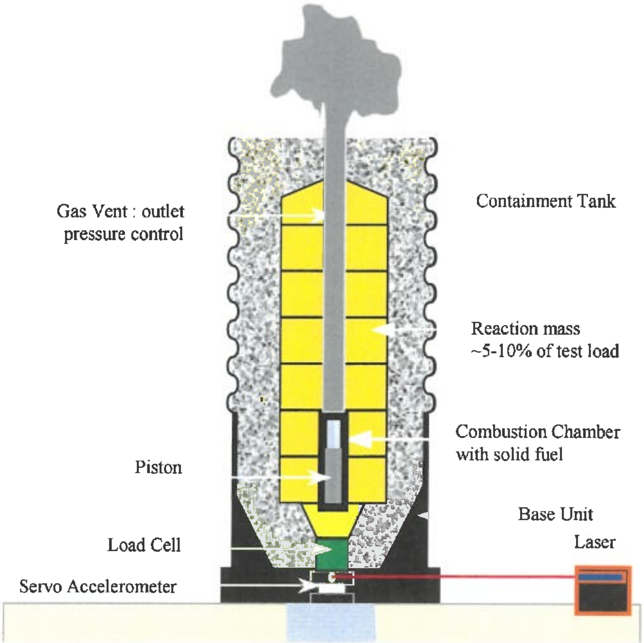


Figure 12 : Statnamic test schematic

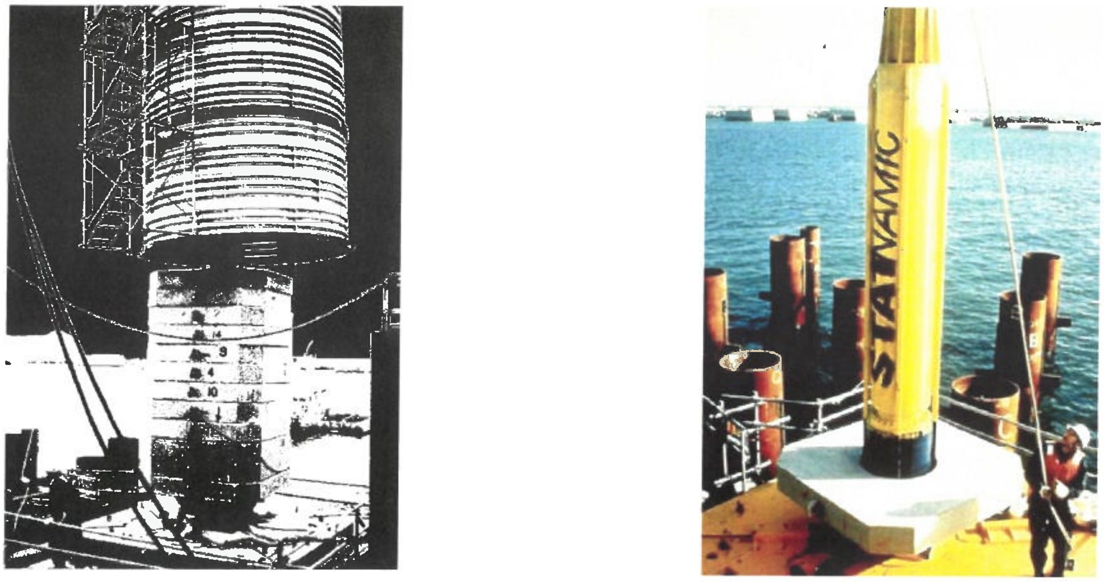


Figure 13. Views of Statnamic test set-up

Figure 14. Dynamic pile testing instrumentation.

Figure 15. Dynamic pile testing data acquisition unit

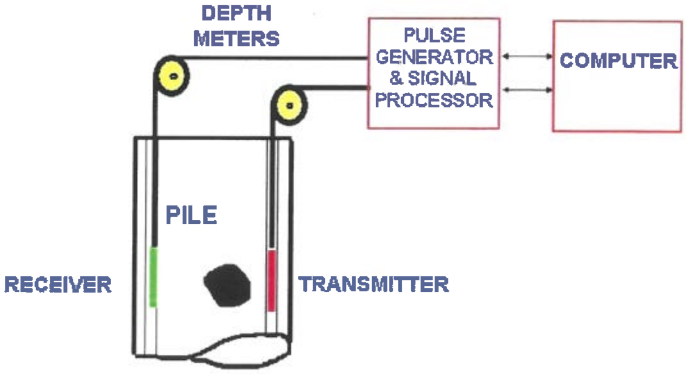


Figure 16. Schematic of cross-hole sonic logging

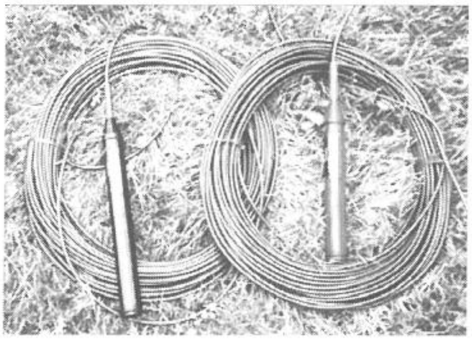


Figure 17. Cross-hole sonic logging probes

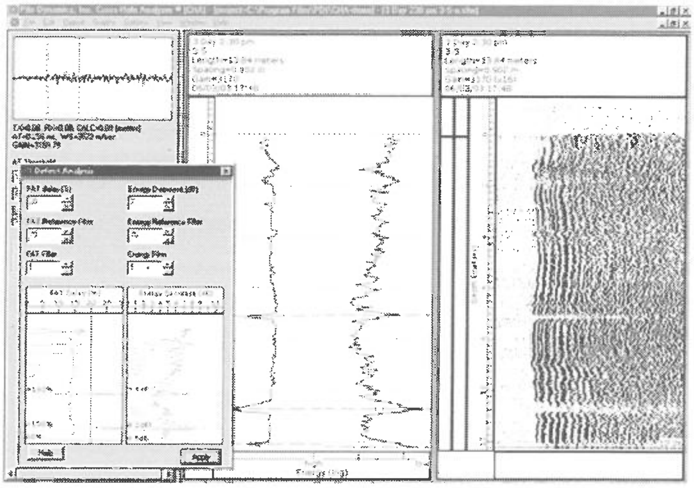


Figure 18. Plots of first arrival time (FAT), Energy and "waterfall" diagram

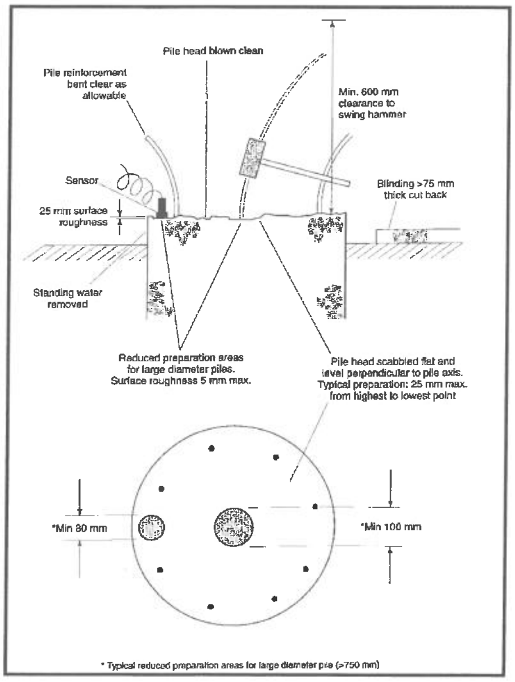


Figure 19. Pulse echo and Impulse Response method schematic.

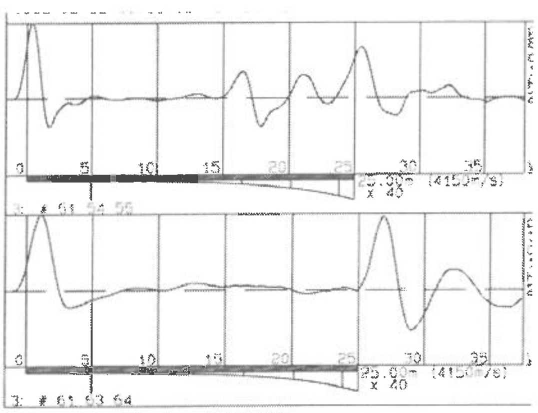


Figure 20. Pulse Echo tests showing responses for a defective pile (upper) and intact pile (lower)

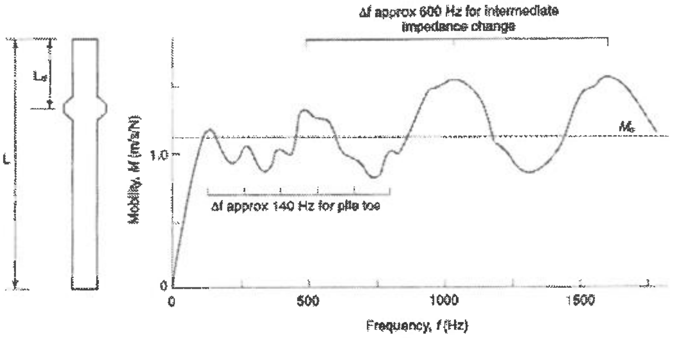


Figure 21. Impulse response test showing response for pile with a bulge in the frequency domain (after Turner, 1997)

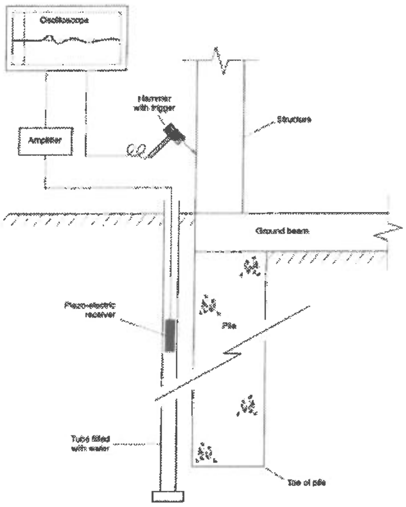


Figure 22. Schematic for Parallel seismic test (after Turner, 1997)

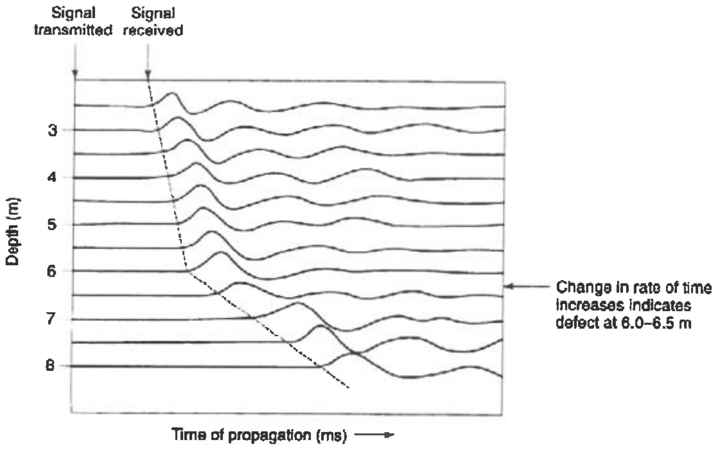


Figure 23. Classic response from parallel seismic test

Figure 24. The SID device – ground level equipment

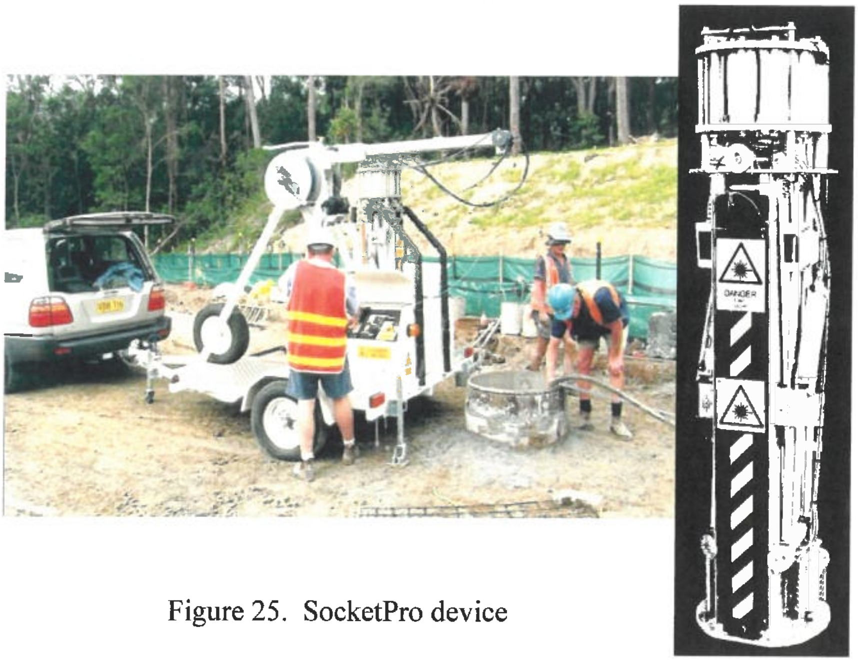


Figure 25. SocketPro device