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INTERNATIONAL ASSOCIATION OF ENGINEERING GEOLOGISTS

1 NOTICE OF MEETING AT GEOENG 2000 IN MELBOURNE

There will be an inaugural meeting of the Australasian IAEG Group (which has been formed under the umbrella of the AGS and the NZGS) at the Geoeng 2000 conference in Melbourne. The meeting will take place at 5.30pm on Tuesday, 21 November 2000 in a room at the Conference venue (the Melbourne Exhibition and Convention Centre). There will be an address by representatives of IAEG, a presentation describing the progress of the Australasian Group, and a meal and social evening afterwards.

2 DATA BASE OF PRACTITIONERS

An initiative of the Australasia IAEG Group Steering Committee is to compile a list of engineering geologists for the region. The objective is to establish a data base of practitioners, with a view to advancing the aims of the IAEG in the Australasian region, arranging informal meetings, forming working groups, networking and promoting a common interest in engineering geology.

It is intended that the list will be published in the Australian Geomechanics Society Journal and the New Zealand Geomechanics News; the list will also be circulated to all contributing engineering geologists following compilation. We look forward to your response, it will be to your benefit to complete this form and send it off, so put pen to paper now.

NAME____

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____

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YEARS EXPERIENCE____

FIELDS OF EXPERTISE____

____

We would appreciate it if you could pass this on to other engineering geologists, or alternatively, provide the names and address of others who may be interested: Please return the completed forms by 30 November 2000 to:

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10/272 Hay St
SUBIACO 6008 WA
Phone (08) 9381 9498
Fax: (08) 9382 1564
email: fredb@iinet.net.au

Front Cover (Andrew Leventhal): View of an operating coke oven in Goa, India, immediately after the pushing of coke. The heat in the underflue (beneath the floor) produced up to 0.6m of settlement in this oven battery. Details of the project are presented in the proceedings of the John Booker Memorial Symposium (Nov 2000), with abridged details in the proceedings of GeoEng2000 (Nov 2000). *(Courtesy of GHD-LongMac Pty Ltd)*

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UPCOMING AGS EMINENT SPEAKER TALKS – SYDNEY, PERTH, ADELAIDE

"Application of Limit State Principles to Geotechnical Engineering"

Speaker: Brian Simpson (Ove Arup & Partners - UK)

SYNOPSIS

Eurocode 7, on Geotechnical Design, is one of a suite of nine European codes for the design of buildings and civil engineering structures. Work on Eurocode 7 started in the early 1980s, and it was published as a draft for development in 1995. A final version is due for publication in 2001.

It is intended that the Eurocodes will be mutually consistent documents, and this has set a major challenge to geotechnical design, since the interface between geotechnical and structural design codes has been confused in many nations, including the UK. The Eurocodes are based on a limit state method, with safety margins generally provided in the form of partial factors.

Eurocode 7 sets out the use of a partial factor approach to geotechnical design in a context of sound geotechnical engineering. It attempts to define the process by which parameter values will be selected for calculation, from the usual sparse and scattered data available to engineers. In separate sections, it then details the application of this approach to typical foundation types, retaining structures and slopes.

This talk will describe the contents of the code and present some of the more important, and controversial, features of its methods. These will be illustrated by examples of both foundations and retaining structures. Likely changes between the 1995 draft and the issue planned for 2001 will be discussed.

BRIAN SIMPSON

Brian Simpson is a director of Ove Arup & Partners and a principal of Arup Geotechnics. He joined Arups in 1971 after researching the use of finite element methods in geotechnics at Cambridge University. He has worked on a wide range of geotechnical and ground-structure interaction problems, with particular interests in numerical modelling, retaining structures and tunnels, and in 1992 presented the British Geotechnical Society's Rankine Lecture on the subject "Retaining structures - displacement and design". Since the early 1980s, he has been involved in the development of Eurocode 7 (Geotechnical Design), having been a member of its drafting panels and vice-chairman of the CEN (Comité Européen de Normalisation) committee on Eurocode 7. He was lead author of a Commentary on Eurocode 7 published by EMAP, and is currently Editor of the journal Géotechnique.

TIMES LOCATIONS AND CONTACTS

SYDNEY: Monday 13th November, 5.30 for 6pm.
IEAust, Harrick Auditorium, Milsons Point.
Contact: craig.covil@arup.com.au

PERTH: Tuesday 14th November, 5.30 for 6pm.
IEAust, 712 Murray St, West Perth.
Contact: derek.pennington@arup.com.au

ADELAIDE: Thursday 16th November, 5.30 for 6pm.
IEAust, 11 Bagot St, North Adelaide.
Contact: mjaksa@civeng.adelaide.edu.au

EDITORIAL

This is our ninth issue of Australian Geomechanics. While it is now at least twice as frequent, it is still late. It seems I am never going to be able to boast about getting an issue out on time. This time I have a once in a millennium excuse - the Olympics are to blame! No way am I letting such an excuse go to waste.

This issue is an exhibition of the work of the young members of the Society. The first seven papers and Glastonbury have been drawn from those published in the Fourth Australia New Zealand Young Geotechnical Professionals Conference, held in Perth in February 2000. These highlight the research and practice of members nominated by their organisations to attend the conference. The proceedings contain thirty six papers and shows that the Society has many capable young members. John Small, Nasser Khalili and I organised the first of these conferences in 1994, it was considered a success and it was decided to organise future conferences at two yearly intervals. Since then the organising committees have generally consisted of young members and the process has become self perpetuating. The content of these papers and the fact that the conferences are organised by our young members indicates that the Society's future is in good hands.

The papers by de Ambrosis (the younger), Gourlay and Moyes are drawn from ones presented during a Young Geotechs evening in Sydney – again these point to the secure future of the Society. If the better of our monthly speakers around the country could write up their presentation in this manner, the Journal would provide an invaluable repository of local knowledge for our members. Each chapter should consider this.

Finally, there are several pages devoted to discussion and letters regarding the two papers (Fell and Risk) in the March issue – it is good to see that the geotechnical community has sprung to life on these matters. Several views are put and replies provided.

Garry Mostyn
Editor

AGS National Committee

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From The Chairman's Desk

I am indebted to a colleague for pointing out to me a marvellous mixed metaphor that he attributed to a Manchester United football player. I think it aptly describes how I feel about the current state of our Society: "I can see the carrot at the end of the tunnel".

Incorporation of AGS

With this thought in mind, it is important that I provide a brief update on a significant issue that has been a concern of many of us, and occupied the time of a number of members of your national committee over the past few years.

You will recall that as a Society we voted on and approved a Constitution not so long ago. The motivation behind this development was always to place our Society in a stronger position than it had previously been, and in particular to provide us with more autonomy and an unequivocal measure of financial independence. In addition to drafting and approving a formal constitution, we subsequently applied as a Society for incorporation as a non-profit organisation within the ACT. Unfortunately, this application for incorporation hit a number snags in the ACT Registrar General's Office. These stem essentially from several ambiguities and omissions in our Constitution, as presently approved by the membership. In order to overcome these difficulties, we have taken advice from lawyers experienced in this field, and I understand that the outcome of our consultations will be a number of suggestions for revision of the Constitution. Importantly, our lawyers are liaising with the ACT Registrar General to ensure that we "get it right" this time. The process is not yet complete, but once the necessary amendments are known I anticipate that they shall be widely circulated and put to all our Chapters for approval at either their next Annual General Meeting or at a Special General Meeting. So please be prepared to consider these important changes in due course.

In this regard I should acknowledge the wonderful efforts being made on our behalf by Tony Phillips, the father of our original Constitution. He remains our trusted Henry Parkes, and his continuing assistance to me on this matter should not go unacknowledged.

GeoEng 2000

In the last issue of AG I mentioned that the highlight of 2000 was GeoEng2000. I believe it is most appropriate that I remind you yet again about this major international event being organised and hosted by your AGS. As I have stated previously, this conference has all the hallmarks of being a landmark in geotechnics, perhaps the best and most comprehensive in a lifetime. We expect a large number of overseas delegates, and this will be a rare opportunity to rub shoulders with many of the luminaries in our discipline. I am advised that to date local registrations have been a little slow. Perhaps this is due to the distraction caused by the international sports carnival being held in Sydney during September. With that out of the way, I ask that you now turn your mind to registering for GeoEng. It is THE main event for 2000!

Best wishes and adieu!

John Carter

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Acknowledgement in front section of every issue of Australian Geomechanics

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Contact Val Lee on 02 6270 6558 or fax 02 6273 4825 for application forms, etc.

AUSTRALIAN STANDARDS COMMITTEE CE/9
HANDBOOK – TESTING OF SOILS FOR ENGINEERING PURPOSES

Standards Australia Committee CE/9 is embarking upon a project over the next 12 months to produce and publish a handbook to explain the application and performance of the AAS 1289 series of tests on soils for engineering purposes. This will be in relation to Codes such as AS 1726 – *Geotechnical Site Investigations*, AS 3798 *Guidelines on Earthworks for Commercial and Residential Developments* and AS 2159 *Piling – Design and Installation.* The handbook will be developed under a sub-committee of CE/9.

The aim of the handbook is to assist engineering staff in the selection of suitable tests for the work involved and to provide a training tool for educational institutions, technicians and engineers.

The sections or chapters of the handbook will follow that of the AS 1289 groups of test methods (ie. Sampling and Preparation of Soils, Soil Classification Tests. Soil Compaction and Density Tests, etc). They will provide background information, additional explanations and technical advice on the test methods and identify some of their limitations as well as referencing documents relating to the test methods. It is envisaged that this handbook will become a significant publication in the area of soils testing for engineering purposes.

Those wishing to make contributions to the handbook or wanting further information regarding the handbook should contact Mr Adam Fitzhenry at Standards Australia:
Telephone: (02) 9746 4700
Email: adam.fitzhenry@standards.com.au

THE DEVELOPMENT OF WATER BALANCE MODELS FOR TAILINGS MANAGEMENT

S. J. Watson

Summary The management of water on a tailings storage facility (TSF) is an important aspect of the overall management of a TSF. A useful tool to assist with TSF water management is a water balance model that tracks water inflows and outflows, and the change in surface water storage on a TSF. This paper discusses the components of a TSF water balance model and the uses of such a model. A case study illustrating the development of a water balance model for the TSFs at Kalgoorlie Consolidated Gold Mines is then presented.

1 INTRODUCTION

In recent years the management of tailings storage facilities (TSFs) has received increasing attention from government regulators and operators within the mining industry. One important aspect of tailings management that relates to the safe operation and efficient management of a TSF is the control of water within the storage area.

Water management in a TSF largely involves controlling the size and position of the decant pond through careful management of slurry deposition and decant water return to the processing plant. External influences such as rainfall, evaporation and seepage complicate the overall management of water on a TSF.

A useful tool to assist with the water management in a TSF is a water balance model, which accounts for all of the water inflows and outflows, and the change in water storage within the TSF. TSF water balance models can be developed for individual paddocks within a TSF, for an entire TSF, or for a series of TSFs that are independent or interconnected.

Two types of TSF water balances can be developed to assist with water management. An overall water balance can be developed for a TSF by accounting for the change in surface water storage and interstitial void storage. This requires all of the internal and external water flows to be accounted for in the water balance model.

Alternatively, the model can be simplified to account for the change in surface water storage only. In this case some of the external flows, such as seepage from the facility, and the processes that cause a change in interstitial void storage can be neglected. This has the advantage of simplifying the model by reducing the number of parameters and processes that have to be included in the model. A disadvantage of this approach is that the overall water balance for the TSF facility can not be evaluated.

This paper focuses on this second approach for TSF water balance model development, although most of the information presented is also applicable to the first approach.

2 COMPONENTS OF A WATER BALANCE

2.1 GENERAL

A water balance is an account of all quantities of water added to, stored within, or removed from a system over a specified period of time, such as a day, week, or month. In general, a water balance for a TSF is dynamic, which means that the components of the water balance vary continuously with time.

The main components of a TSF surface water balance are classified as storages, inflows or outflows, and are presented in Table 1.

POND INFLOWS	POND OUTFLOWS
Slurry Bleed Water	Seepage / Drainage
Seepage / Drainage Return	Return Water
Rainfall	Evaporation
Additional Pumped Water	Spillway Overflow
Consolidation Water	

Table 1 Components of a TSF Surface Water Balance

The difference between the inflows and the outflows is the quantity of water that is added to (positive difference) or removed from (negative difference) water storage upon the TSF.

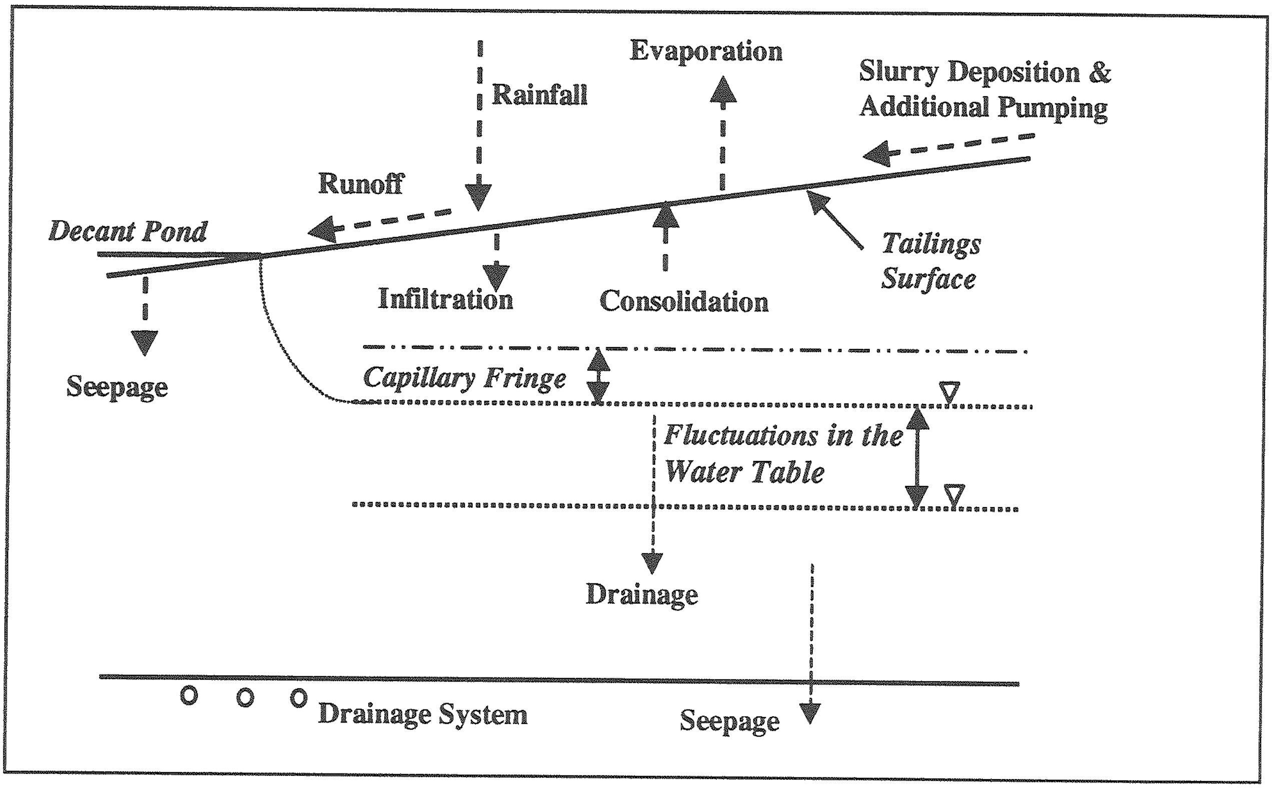

Figure 1 Schematic representation of the water inflows into a TSF and outflows from a TSF.

A schematic representation of the internal and external water flows is given in Figure 1, and a description of the inflows and outflows is given below.

2.2 SLURRY DEPOSITION

Mine tailings are generally deposited in TSFs as a slurry from single or multiple discharge points. Slurry deposition is therefore an external inflow of water into the facility.

In TSFs under active deposition, the rate at which slurry water is discharged into the facility is generally the largest of the water inflows into the facility. Slurry water deposition is also the water inflow that is most easily managed by the operators of a TSF.

Using the simplified (surface water) approach for TSF water balance model development, the volume of slurry water that reaches the surface water pond (decant pond) has to be evaluated. The volume of slurry water being deposited into the facility can be readily calculated from the tonnage being deposited, the slurry solid content, the specific gravity of the tailings solids and the density of the slurry water. However the volume of water discharged into the facility is greater than the volume of water that reaches the decant pond because some of the slurry water is held in interstitial voids, some infiltrates into the underlying tailings, and some is lost to evaporation.

A rough estimate of the volume of slurry water that inflows into the decant pond can be obtained by assuming that a fixed percentage of the slurry water reaches the pond. A more accurate approach is to account for the processes that have a *significant* effect on the volume of water that reaches the decant pond. These processes include sedimentation of the tailings with release of slurry water (termed bleed water), followed by loss of the bleed water to evaporation and infiltration, as a function of the area under active deposition.

2.3 TAILINGS CONSOLIDATION

Water that is released to the tailings surface during consolidation of the tailings is an inflow of water into the decant pond. The volume of water released by consolidation depends on:

- the mineralisation, physical and chemical properties of the tailings
- the depositional history of the tailings, particularly the rate of tailings deposition
- the thickness of tailings
- the permeability of the TSF foundations.

Generally, in a facility that has a small decant pond, the majority of water that is released to the surface during consolidation of the tailings evaporates. However, if the tailings has been deposited with a rapid rate of rise, or the majority of the tailings surface is covered by the decant pond, then a larger proportion of consolidation water is likely to inflow into the decant pond.

An estimate of the volume of water released into the decant pond by consolidation of the tailings is best obtained from a consolidation model of the facility.

It should be noted that if an overall water balance is being developed for a TSF, water release due to consolidation not only results in an inflow into the decant pond, but also reduces the volume of water stored in interstitial voids.

2.4 RETURN WATER

Most TSFs are constructed with a decant facility to remove surface water off the TSF. The majority of operations return water to the processing plant, whereas some allow for evaporation or disposal of the water. Returning process water to the plant reduces overall raw water and reagent consumption, thereby reducing costs. It has the added advantage of minimising the volume of contaminated slurry water that requires treatment or storage.

At any point in time, the volume of water returned to the process plant from a TSF depends on a number of factors including:

- the volume of decant water available on the TSF
- the pumping capacity of the water return system
- the maximum volume of process water able to be used by the processing plant.

Plant return water is generally the outflow from a TSF that is most readily controlled by the TSF operator.

2.5 SEEPAGE AND DRAINAGE

Seepage and drainage from a TSF is an external outflow from the facility into either an underdrainage collection system, if such a system has been installed, or into the surrounding environment.

It is important to minimise seepage from a TSF for both environmental and economic reasons, to minimise adverse environmental impacts, such as groundwater contamination and salinisation of the root zone, and to minimise the loss of process water and chemicals.

In some cases the seepage that is collected in seepage collection dams or from recovery bores is returned to the plant. In others, the seepage water is returned back into the TSF from which it has emanated. In the latter case, the seepage water that is recovered becomes a surface inflow into the facility.

If the water balance is being developed to account for changes in surface water storage only, the seepage from the decant pond into the underlying tailings is more important than seepage from the base of the TSF. The rate of seepage from a decant pond is a function of the tailings depth, tailings permeability, pond size and to some extent the facility size. The rate of seepage from a decant pond can either be estimated using Darcy's law, or can be evaluated using 2-dimensional sectional seepage models.

2.6 RAINFALL AND EVAPORATION

Rainfall is an external water inflow and evaporation is an external water outflow from a TSF, but unlike slurry deposition and water return, these water flows are largely unable to be controlled by the operators of the TSF.

The volume of rainfall that reaches the decant pond of a TSF depends on the amount of runoff and evaporation that occurs from various regions of the decant pond catchment.

A decant pond catchment can generally be divided into five main regions, discussed below:

- Wet Beach - is the area of tailings beach under active deposition. In this area the tailings are already saturated so 100% of the rainfall is likely to become runoff.
- Drying Beach - is the area of tailings beach that has recently undergone deposition, but is no longer an active beach. The tailings in a drying beach are also close to saturation, so 100% of the rainfall can be assumed to runoff. There will be some evaporative losses from the runoff.
- Dry Beach - is the area of tailings beach that has dried to the point that the voids are no longer fully saturated. In this case some (or all) of the rainfall is lost to infiltration and evaporation, depending on the intensity and duration of the rainfall event. There may also be significant cracking of the tailings surface in a dry beach area, which can affect the volume of water lost to infiltration and evaporation. The remaining volume of rainfall can be assumed to report to the decant pond.
- Decant Pond - rainfall falling directly on the decant pond is a direct inflow into the pond. Water loss from the decant pond due to evaporation also needs to be included in the water balance.
- Surrounding Catchment – Some TSFs located within valleys may receive runoff from the surrounding catchment during a rainfall event if drainage channels or bunds are not constructed to divert the catchment runoff.

If a TSF water balance model is being developed to account for changes in surface water storage, then only the volume of rainfall and runoff reaching the decant pond, and the volume of evaporative loss from the pond need to be accounted for in the model.

Alternatively, if an overall water balance model is being developed to account for changes in surface and interstitial storage, then the volume of rainfall infiltrating into the residue and the volume of water lost to evaporation in the various regions of the TSF must be included in the model.

2.7 CLIMATIC DATA

One aspect of TSF water balances that requires special consideration is the climate data to be used in the simulation. It is not possible to predict future rainfall and evaporation rates, so various techniques are used to obtain climate data for the simulations.

One approach is to use measured historical daily rainfall and evaporation data for the area. A long period of historical record is preferential because it allows a number of simulations to be run, starting at different times in the historical records. This allows a range of climatic conditions to be applied to the simulated deposition strategy. The simulation results can then be interpreted to give probabilistic results, such as daily water return volumes and decant pond elevations. An advantage of this approach is that it can be used to examine the effect of climatic extremes by simulating a series of wet or dry years.

A less accurate approach is to use average monthly rainfall and evaporation rates. The main disadvantage of this technique is that the averaging procedure smooths out the rainfall, moderating the effects of irregular rainfall events.

2.8 SPILLWAY OVERFLOW

Some TSFs are designed with spillways to remove excess water from the decant pond when the elevation of the water surface in the decant pond exceeds the elevation at the base of the spillway. Typically spillways are used to transfer excess water into other TSFs (or other water storage facilities) to maintain sufficient freeboard within an interdependent system of TSFs.

2.9 TSF GEOMETRY

To account for the changing geometry as slurry is deposited into a TSF, a water balance model must incorporate the following:

- The depth of the decant pond as a function of pond volume
- The surface area of the decant pond as a function of the pond volume (or depth)
- An approximate rate of rise of the tailings during the period that tailings is being deposited.

These geometrical relationships are extremely important as they affect many of the water balance components, including evaporation and seepage from the decant pond, rainfall runoff from most of the beach areas, rainfall recharge of the decant pond, and consolidation water flow into the decant pond.

3 USES OF A WATER BALANCE MODEL

A TSF water balance model, which accounts for the storages, inflows and outflows discussed in Section 2, can be used to:

- Improve deposition strategies in the short or long term to maximise water recovery or maximise water evaporation, depending on the water return requirements
- Estimate the likelihood of exceeding the maximum operational water elevation for a range of climatic conditions
- Estimate the freeboard necessary to contain extreme rainfall events, such as a 1:1000 year storm event
- Estimate the likelihood and extent of a return water shortfall under average or extreme climatic conditions
- Manage the water in TSFs containing acid generating tailings that may require a permanent water cover
- Provide inputs for a qualitative and quantitative risk assessment of a TSF.

4 DEVELOPMENT OF A TSF WATER BALANCE MODEL FOR KCGM

4.1 BACKGROUND

KCGM is Australia's largest gold producer from one operation, with annual gold production in excess of 23 tonnes. Approximately 13.7 million tonnes of ore are processed annually to produce the gold, which results in approximately 13.7 million tonnes of waste tailings being generated annually.

The waste product is a sandy (fine-grained) silt tailings which is discharged as a slurry into the KCGM TSFs. In excess of 23,000 kL of slurry water are discharged daily into the TSFs. A proportion of this slurry water reports to the decant ponds and is available for return to the processing facilities.

KCGM spends several million dollars a year on process water for the plant. It was therefore considered desirable to develop a water balance model for the KCGM TSFs to enable reasonably accurate prediction of return water from the TSFs.
Another purpose of the model was to enable comparison of differing tailings management strategies and/or TSF construction options to maximise water recovery.

4.2 KCGM TAILINGS STORAGE FACILITIES

KCGM has three active TSFs called Fimiston I, Fimiston II and Gidji. Tailings from the Fimiston processing plant is predominantly discharged into Fimiston II, with excess tailings directed to Fimiston I. A much smaller volume of tailings is discharged into the Gidji TSF from the Gidji roaster.

Fimiston II is the largest TSF covering approximately 3.6 km^2. Tailings discharge is cycled between three paddocks, with each paddock receiving approximately 2 months of deposition, followed by 2 months of drying and 2 months of wall raising.

Fimiston I and Gidji are also divided into paddocks so that tailings deposition can be cycled between paddocks to allow for drying of the tailings. This allows for upstream raising of the perimeter walls with tailings borrowed from the adjacent beach.

An important consideration in the development of the water balance model was the incorporation of a flexible deposition strategy to enable the cycling between facilities to be modelled, and also to enable different deposition strategies to be evaluated.

4.3 MODEL FRAMEWORK

The model was developed in Microsoft Excel as it was desirable to have a model that could be readily adapted in the future, and which allowed for the incorporation of add-in programs, such as @RISK, at a later date.

The model structure incorporates a graphical menu to direct the user to various sheets in the model and a control menu to enter simulation parameters and run the water balance simulations. There are a number of data entry sheets setup in the model, including an operational data sheet which allowed for the entry of different deposition strategies. A climatic data sheet has also been included in the model, containing 25 years of historical rainfall and evaporation data to be used for the model simulations. The model also contains a master calculation sheet for the water balance calculations and various report sheets to view the simulation results.
The model was developed to track water flow into and through each TSF paddock on a daily basis and daily return water availability.

The model was also developed to track three water quality parameters, namely pH, magnesium ion concentration and the concentrations of total dissolved salts (TDS). This functionality was included so that the model could simulate the quality of the water returned to the processing facilities.

The chemical species of interest (hydrogen ions, magnesium ions and other salts) were assumed to be conservative solutes. This allowed the chemical concentrations to be tracked using a mass balance approach.

4.4 WATER BALANCE COMPONENTS

The following components were incorporated into the KCGM TSF water balance model:

- slurry deposition
- rainfall and evaporation
- water return to the plant
- seepage losses.

Consolidation water release to the decant pond was not included in the water balance model, as this inflow was considered negligible given the overall water flow volumes for the KCGM TSFs.

Some other important processes that were incorporated into the model equations include:

- the loss of slurry (and therefore slurry water) to borrow pits around the perimeter of the tailings surface following upstream raising of the perimeter embankment
- loss of rainfall to cracks
- effect of salinity on evaporation

- runoff factors for each month for a range of rainfall events
- varying rates of seepage from each decant pond as a function of the pond size, residue depth and paddock area.

4.5 MODEL SIMULATIONS

The model has been developed so that different water balance simulations can be run once all of the operational and tailings data has been entered into the model. Water balance simulations can be run for a specified period of deposition using either wet, dry or randomly selected sets of daily historical rainfall and evaporation data. If the climate sets are set to be randomly selected, up to 25 different climate scenarios can be run against the specified depositional strategy.

4.6 MODEL CALIBRATION

Although historical and operational data has been used during the development of the model, many of the lookup tables upon which the model relies have been estimated from experience, or using surface or groundwater models with little hard data available for their calibrations.

The model is now in a stage of calibration to real data to refine various parameters, such as runoff coefficients for the various beach areas, seepage rates from the decant pond, beach drying times and dry beach crack volumes.

4.7 MODEL OUTPUT

The results from a water balance simulation are presented in tabular and graphical form in the KCGM TSF model. Results are generated for each TSF paddock and for user specified paddock combinations to obtain total processing facility results.

The output results generated for each paddock include:

- the volume of water returned to the process plant
- the percentage of total slurry water that is returned to the plant
- the TDS and magnesium ion concentration of the return water
- the pH of the return water.

The output results generated for each paddock are tabulated and produced graphically against the simulation time. An example of a paddock graph showing the volume of water available for return to the processing plant from a paddock in the Fimiston II TSF is given in Figure 2. This example has been generated from a simulation using 5 different climate sets (CS1 to CS5). As the model has not yet been calibrated, the volumes presented in Figure 2 are not an actual indication of water return volumes for the single paddock.

The output results generated for each paddock for each climate set are combined to generate processing facility results. The paddock results are evaluated statistically to give the mean and 90% confidence interval (90% CI) values for the parameters listed above (for the paddock results).

The statistical results for the processing facilities are tabulated and produced graphically against the simulation time. An example of a facility graph showing the mean and 90% confidence interval limits for the volume of water returned to the Fimiston processing facility is presented in Figure 3. The 90% CI curves give the upper and lower limits within which there is a 90% confidence that the predicted values will fall. As for Figure 2, the volumes presented in Figure 3 are not an actual indication of water return volumes to the KCGM Fimiston processing plant because the model has not been calibrated to site data.

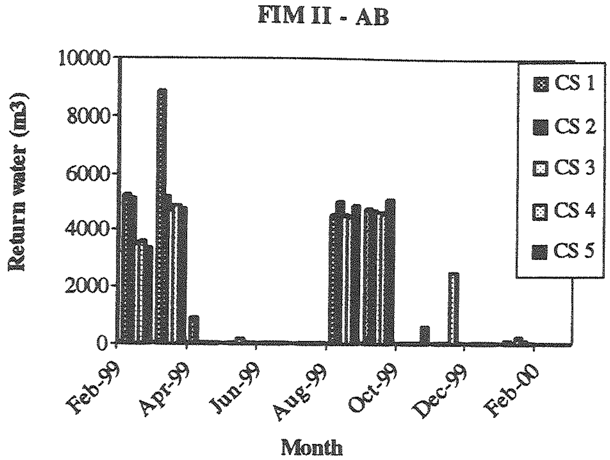

Figure 2 Example graph for a TSF paddock

5 CONCLUSIONS

A water balance model is a useful tool to assist in water management in and around a TSF, and to indicate how much water is likely to be available for return to the processing plant. A water balance model can also be used to assist in the development of deposition strategies that are appropriate to the water return requirements of the mining operation.

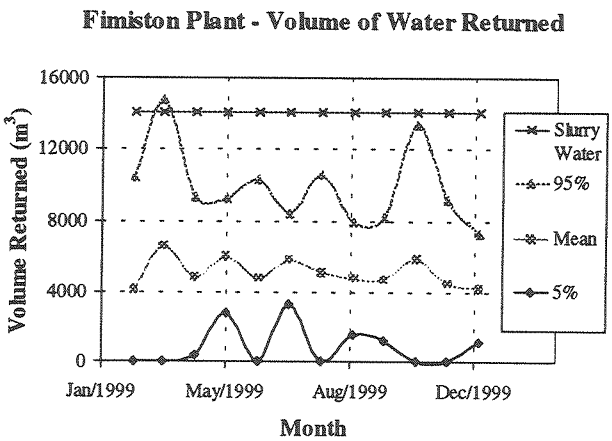

Figure 3 Example of a facility graph

It is generally not necessary to do an overall water balance for a TSF, as a surface water balance model provides reasonable estimates of surface water storage volumes, and the volume of water available for return to the plant.

The reliability of a water balance model is controlled by the accuracy of the input data. Some of the input data can be measured, but some data has to be estimated. In general, it is possible to make reasonable estimates of the input and output flows resulting in reliable output results from the water balance simulations.

6 ACKNOWLEDGEMENTS

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THE RESPONSE OF SUCTION CAISSONS TO CATENARY LOADING

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ABSTRACT

Research into the performance of suction caissons has developed in response to the demand from the offshore hydrocarbon industry for a versatile foundation solution capable of anchoring a range of alternative structures. Suction caissons are capable of providing large anchoring capacities in all directions. The simple installation procedure and high reliability has seen suction caissons employed in a range of water depths and within hydrocarbon fields from marginal to high potential. As exploration is directed toward increasing water depths, anchoring demands on the proposed structures become greater and subsequently a more detailed understanding of the limitations to caisson capabilities and performance is required.

Using the fixed beam geotechnical centrifuge facility at The University of Western Australia (UWA), the installation and response of a dimensionally scaled prototype caisson to inverse catenary chain loading was modelled with the objective of establishing a relationship between the caisson geometry, soil characteristics and the monotonic holding capacity. The installation and tensile resistances were recorded to determine the necessary installation pressures and uplift capacity of the caisson. Theory suggests that the lateral capacity is dependent upon the frictional resistance between the caisson and soil, which may be back derived through calibration of the theoretical and experimental response of the caisson to axial loading.

This paper presents the data from a series of centrifuge tests, comparing the results with the theoretical monotonic capacity of laterally loaded caissons. A smooth walled model caisson was installed and subsequently loaded with an anchor chain in normally consolidated kaolin clay. The data exhibited excellent repeatability between identical tests and a similar correlation with the adopted upper-bound plasticity solution for laterally loaded caissons.

1 INTRODUCTION

The versatility and cost effectiveness of suction caisson foundations has initiated significant research into the capabilities of and limitations to their applications.

In moderate water depths, suction caissons may be used within clusters as a mooring for such structures as floating, production, storage and offloading (FPSO) facilities. For these applications each caisson is attached to the structure with a chain that forms an inverse catenary profile within the soil between the mudline and the point of attachment, imposing a predominantly horizontal load on the anchor.

The first catenary moored structure using suction caisson foundations was at the Gorm field offshore Denmark (Senpere and Auvergne, 1982). The soil profile at Gorm comprises dense, fine sand overlying soft clay above stiff clay, proving the suitability of suction caissons in a diversity of soil types. Most recently, the first suction caisson installations within calcareous soils have been undertaken in the calcareous silty sediments of the Timor Sea for the Laminaria hydrocarbon field (Schrøder and Finnie, 1999).

As offshore hydrocarbon exploration exploits deeper waters, a greater understanding of the combined axial and lateral capacity of suction caissons is warranted.

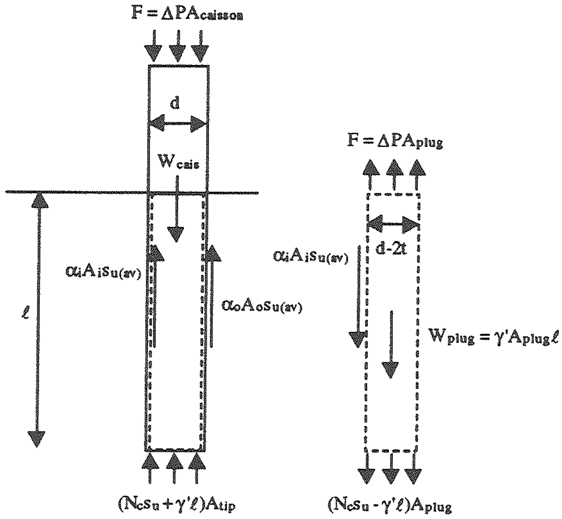
Very little experimental research has been published on the response of suction caissons to lateral loading. Experimental work in progress at The University of Western Australia has the objective of developing a design methodology capable of specifying an optimal caisson geometry for a given soil profile and design load configuration. This research involves the experimental modelling of suction caissons (of various geometries) subjected to loads ranging form purely horizontal to purely vertical in a range of typical offshore soil profiles. This paper presents the results of the first series of centrifuge tests on a caisson monotonically loaded within normally consolidated kaolin clay.

2 BACKGROUND

2.1 INSTALLATION

Of concern during the installation of suction caissons is whether the caisson will reach the target installation depth before upheaval of the internal soil plug. Soil plug failure will not be further discussed in this paper since the aspect ratio of the model caisson and the adopted soil profile suggest a limiting aspect ratio well in excess of the model geometry, (House *et al.*, 1999). Furthermore, the adopted experimental installation method (jacked) eliminates the likelihood of soil plug upheaval due to the absence of the uplift force on the plug experienced during suction penetration.

A free body diagram of the caisson and internal soil plug during suction installation is shown in Figure 1.



(a) Caisson (b) Soil plug

Figure 1 Free body diagram of caisson and soil plug

The required installation pressure is derived from the free body diagram of Figure 1(a) and is defined by

$$\Delta P_{caisson} = \frac{(N_c s_u + \gamma' \ell)A_{tip} + \alpha_i \overline{s_u} A_i + \alpha_o \overline{s_u} A_o - W}{A_{plug}} \tag{1}$$

where:
- N_c = bearing capacity factor (taken as 9)
- s_u = undrained shear strength (at caisson tip)
- γ' = bulk density
- ℓ = embedded caisson length
- A_{tip} = tip area of caisson
- A_{plug} = sectional area of soil plug
- A_i = internal area of caisson in contact with soil
- A_o = external area of caisson in contact with soil
- α_i = internal friction factor
- α_o = external friction factor
- W = submerged caisson weight

2.2 LATERAL CAPACITY

A least upper-bound to the undrained collapse load of a laterally loaded caisson in clay may be predicted using theory discussed in detail by Murff and Hamilton (1993) and refined for suction caisson analyses by Randolph *et al.* (1998). The iterative method (described below) varies three geometric parameters (defining the failure mechanism) and one optimisation parameter to solve for the minimum collapse load satisfying the kinematic constraints of the adopted failure mechanism.

1 Assume a failure mechanism (geometry)
2 Identify velocity field for failure mechanism
3 Derive function between plastic strain rate and energy dissipation
4 Set external work done by imposed loads to total plastic energy dissipation
5 Repeat steps (a) to (d) until minimum collapse load is determined

A schematic representation of the proposed kinematic failure mechanism is shown in Figure 2. The geometric parameters are the depth of the failure wedge, radial extent of the failure wedge and depth to the centre of rotation.

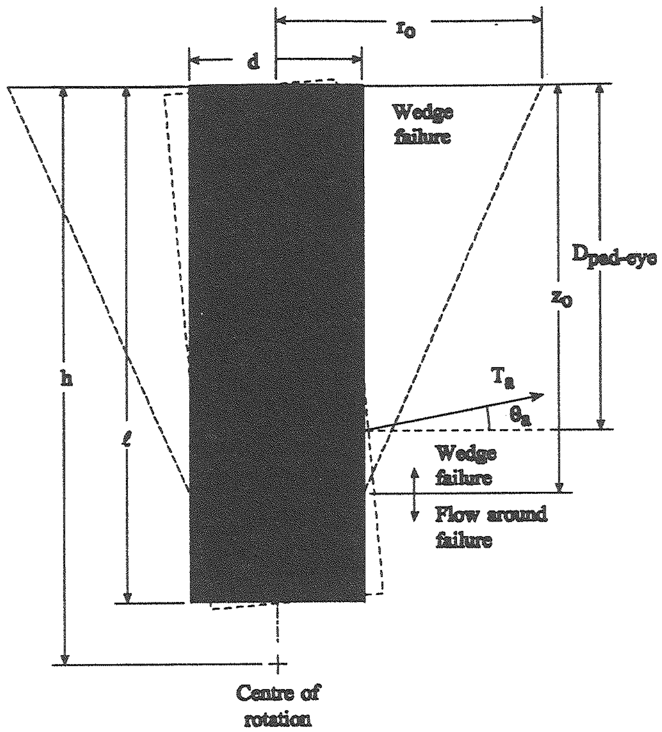


Figure 2 Upper bound failure mechanism

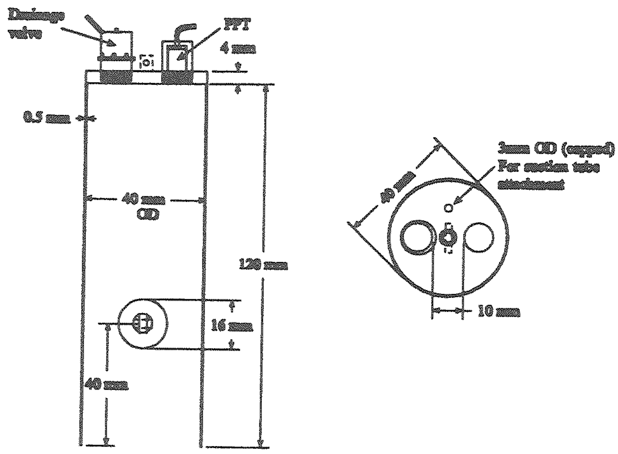
Subsequent to the experimental soil characterisation tests and using a mobilised friction coefficient calculated from the caisson installation data, the least upper bound to the caisson holding capacity is predicted using the aforementioned model.

3 EXPERIMENTAL MODELLING

A model suction caisson was fabricated in the UWA workshop from 6061 T6 aluminium and tested in the centrifuge within a sample comprising normally consolidated kaolin clay. The caisson had a dry mass of 32.3 g excluding attachments (representing a prototype submerged weight of approximately 550 kN), with a wall thickness of 0.5 mm and a stiffened region at the padeye of 1 mm wall thickness. The geometry of the model suction caisson is identified in Figure 2.

The normally consolidated kaolin sample was prepared by self-weight consolidation of a clay slurry on the UWA fixed beam geotechnical centrifuge (Randolph *et al.*, 1991). Commercially available dry kaolin clay powder was mixed to a slurry with a water content of 120 % (twice the liquid limit) and subsequently de-aired under a vacuum of approximately 100 kPa. A slurry depth of 280 mm was placed over a 10 mm sand drainage layer at the base of the strongbox and consolidated under an accelerated self-weight (120 g) for a period of approximately 2 days. Previous characterisation of the same clay at UWA (Stewart, 1991) suggested a fully consolidated model depth of 180 mm would be achieved with a target strength gradient (prototype scale) of approximately 1.1 kPa/m. Key properties of the kaolin clay are detailed in Table 1.

Soil characterisation tests were undertaken using the T-bar penetrometer (Stewart and Randolph, 1991) before each caisson installation associated with a catenary load test. The T-bar was penetrated and extracted at model rates of 3 mm/sec and 1 mm/sec respectively.



(a) Elevation (b) Plan

Figure 3 Geometry of model suction caisson

PROPERTY	VALUE
Specific gravity, G_s	2.60
Liquid limit, LL (%)	61
Plastic limit, PL (%)	27
s_u/σ'_v (Normally consolidated)	0.187
Consolidation coefficient, c_v (m^2/year)	1.3
Clay density, γ_c (kN/m^3)	5.9

Table 1 Kaolin clay properties, after Stewart (1991)

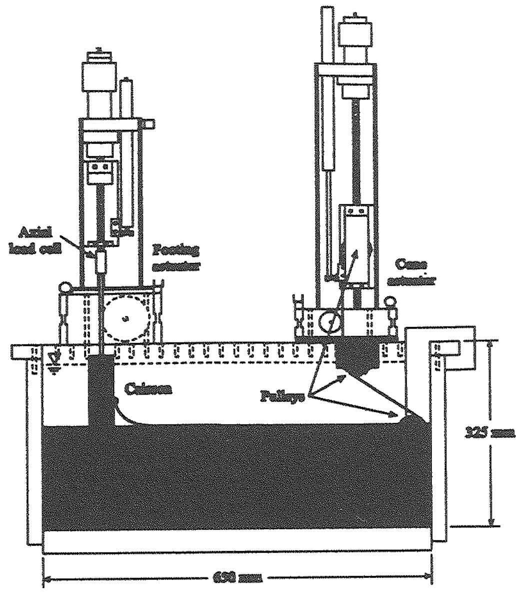
The undrained shear strength of the normally consolidated kaolin clay samples was determined using the T-bar penetrometer (Stewart and Randolph, 1991). The T-bar has a load cell attached to the shaft above the tip from which the installation resistance is directly determined. The bearing pressure, q, is derived by dividing the installation resistance by the bar area and the shear strength is subsequently estimated by dividing the bearing pressure by a bar factor N_b.

$$s_u = \frac{q}{N_b} \tag{2}$$

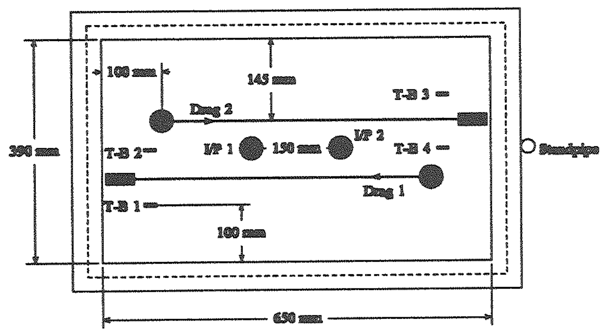
For the bar used it is standard convention to adopt an N_b value of 10.5, based on an average between the theoretical upper and lower bound plasticity solutions for a perfectly smooth (adhesion factor, $\alpha = 0$, $N_b = 9.14$) and a perfectly rough bar ($\alpha = 1$, $N_b = 11.94$), (Randolph and Houlsby, 1984).

The geometry of the experimental arrangement limited the modelling to two catenary load tests and a maximum of two installation / pull-out tests per strongbox. An elevation and plan schematic of the experimental arrangement is shown in Figure 4.

Each installation commenced with the caisson submerged and suspended above the mudline. The loading arm attached to the caisson was lowered into the sample with the drainage valve on the lid of the caisson open. Installation was undertaken at a model rate of 0.5 mm/sec. An axial load cell (3 kN model capacity) attached to the loading arm measured the penetration resistance throughout the duration of the jacked installation. The anchor chain was pinned alongside the anchor wall throughout the penetration phase. A miniature pore pressure transducer (PPT) attached to the lid of the caisson recorded the internal caisson pressure.



(a) Elevation



(b) Plan

Figure 4 Experimental arrangement

After the caisson was installed to the target depth, the centrifuge was stopped, the chain unclipped and attached to the pulley arrangement and the drainage valve on the caisson lid was closed. The loading arm attached to the caisson was removed before the centrifuge was ramped back up to the target normal acceleration and the sample allowed to re-consolidate.

The catenary (inverse) load tests were performed by vertically displacing the actuator associated with the chain and pulley arrangement. The pulleys were geared such that one unit of vertical displacement of the actuator represented a chain displacement of 2 units. A chain drag rate of 0.2 mm/sec (monotonic) was adopted to maintain load control yet also ensure an undrained response. The depth of the end pulley was set such that the catenary profile of the anchor chain agreed with the solutions of Neubecker and Randolph (1995), for the predicted ultimate lateral capacity.

4 EXPERIMENTAL RESULTS & DISCUSSION

4.1 SAMPLE CHARACTERISATION

The normally consolidated kaolin clay sample was characterised by two T-bar tests before each of the two anchor tests. The undrained shear strength profiles are shown in Figure 5.

A comparison of the first (T-bars 1 & 2) and second series (T-bars 3 & 4) of T-bar tests highlighted evidence of a strength increase of approximately 40 % over the duration of the first anchor test and re-consolidation period.

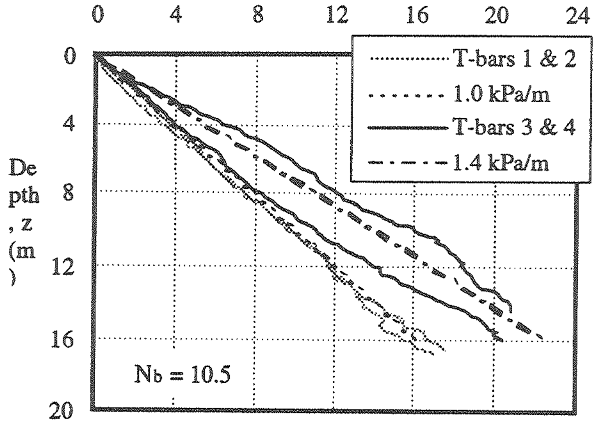


Figure 5 Undrained shear strength

The variability between the profiles of T-bars 3 and 4 suggests that the sites may have been subjected to soil disturbance upon extraction of the embedded end pulley used for monotonic drag test 1. T-bar pull out tests (immediately after penetration) showed a remoulded shear strength approximately 75 % that of the peak undrained shear strength.

4.2 INSTALLATION AND AXIAL LOADING

To ensure adequate drainage through the valve on the caisson lid, installation tests were undertaken at a constant model rate of 0.25 mm/s. The penetration resistances for the two installations preceeeding monotonic load tests are shown in Figure 6. Superimposed on the experimental curves is the theoretical installation response which gave best agreement with the experimental data using an average mobilised friction ratio of 0.3, and k is the shear strength gradient with depth. Note that minor corrections were made to account for the buoyancy effects in the initial stages of installation where the caisson was not completely submerged.

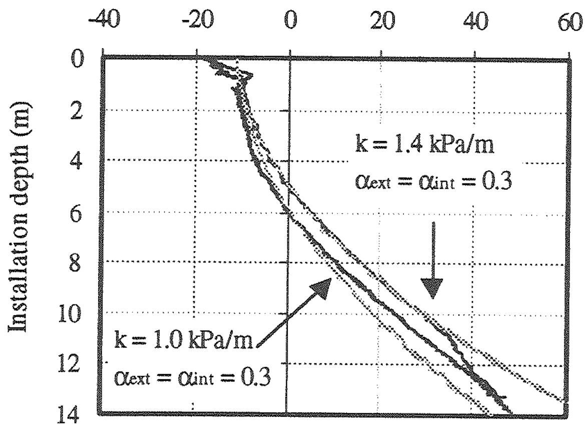


Figure 6 Installation resistances

The penetration resistance and internal caisson pressure rose sharply upon contact of the internal soil plug with the top cap of the anchor. Penetration was stopped at this point which for both installations was approximately 95 % of the target installation depth. Since the installation method was one of jacking, the internal soil plug upheaval may only be attributable to soil displaced by the caisson skirts.

Following the second monotonic load test, one installation / pull-out test was undertaken. The pull-out test was performed with a drained top cap at a constant rate (model) of 0.25 mm/sec. The experimental response is shown in Figure 7.

The caisson pull-out response is classically frictional, suggesting that the caisson-soil interface cohesion was mobilised with no contribution of reverse end bearing. The pressure response within the lid of the caisson showed that no negative pressures were developed between the top of the caisson and the internal soil plug, proving that the soil plug was not extracted with the caisson, verifying the expectations for a drained pull-out test.

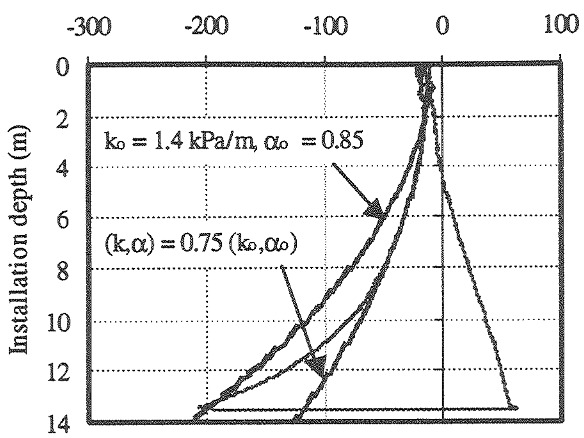


Figure 7 Axial pull-out response

The theoretical pull-out capacity is superimposed on the experimental data, showing that the experimental performance experiences a transition as the skirt friction apparently reduces quadratically. Best agreement is found by application of a reduction factor not only on the shear strength (as experienced in the T-bar pull-out tests) but inexplicably also on the mobilised friction ratio.

4.3 INVERSE CATENARY LOADING

The load development response of the suction caissons subjected to monotonic chain loading is presented in Figure 8. Load development is slow as the initial slack in the chain was taken up. Post-peak behaviour exhibits significant strain softening of the kaolin clay.

The pressure inside the lid of the caisson was monitored as the monotonic chain load was applied. It is observed that no significant pressure response was experienced up to the point of peak holding capacity, beyond which the differential pressure development is a response to the passive suction caused by caisson displacements.

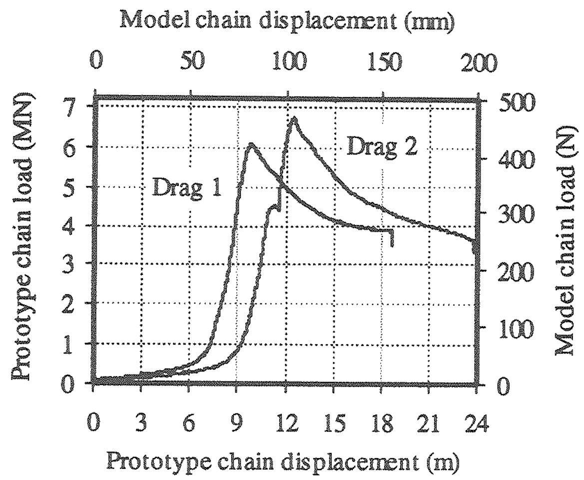


Figure 8 Load development (monotonic loading)

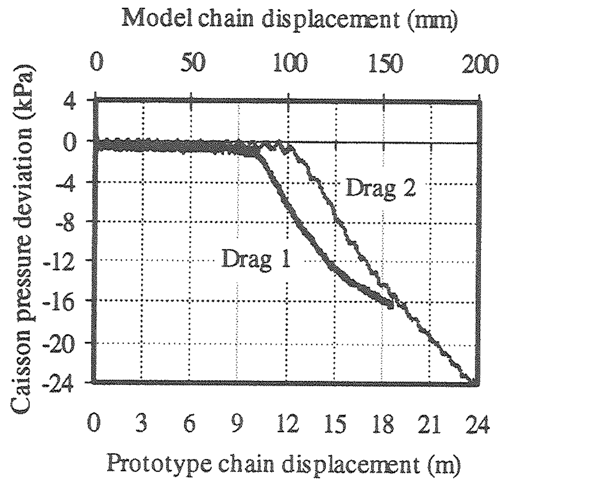


Figure 9 Pressure response (monotonic loading)

The total caisson displacement was approximately 1.5 and 2 diameters for drag tests 1 and 2 respectively.

Using the mobilised friction coefficient back derived from the installation data, the predicted least upper bound to the caisson holding capacity (2 sided failure mechanism) for each of the monotonic load tests is detailed in Table 2.

PARAMETER	TEST 1	TEST 2
Strength gradient, k (kPa/m)	1.0	1.4
Adhesion factor, α	0.3	0.3
Ultimate capacity, P_{ult} (MN)	4.7	6.7
Depth to centre of rotation, h (m)	14.4	14.4
Depth of soil wedge, z_o (m)	11.2	13.2
Radius of soil wedge, r_o (m)	10.1	11.0

Table 1 Predicted caisson holding capacity

Good agreement is observed between the experimental and predicted capacity for Test 2, although for Test 1 the theoretical model under-predicted the measured ultimate holding capacity by approximately 20 %. One possible reason for this under-prediction is that the sample may have experienced a strength increase over the period of reconsolidation between the caisson installation and lateral load tests. The reconsolidation period would also serve to increase the mobilised friction ratio, although as discussed in the following section this has a very minor influence on the predicted holding capacity for the experimental arrangement studied.

5 PARAMETRIC STUDY

To observe the sensitivity of the holding capacity model to the soil strength parameters and proportion of mobilised friction, a series of parametric analyses were undertaken using the experimental caisson geometry subjected to a purely lateral load. For normally consolidated soils (s_{uo} = 0 kPa) of various strengths, the predicted least upper bound to the caisson capacity is shown in Figure 10 for a two sided failure mechanism and a range of friction ratios.

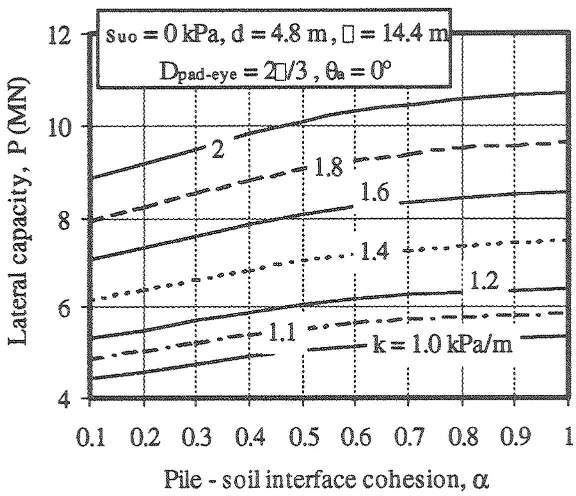


Figure 10 Influence of shear strength gradient

Similarly, the influence of mudline shear strength, s_{uo}, on the ultimate holding capacity of the same caisson within a 'typical' soil is shown in Figure 11.

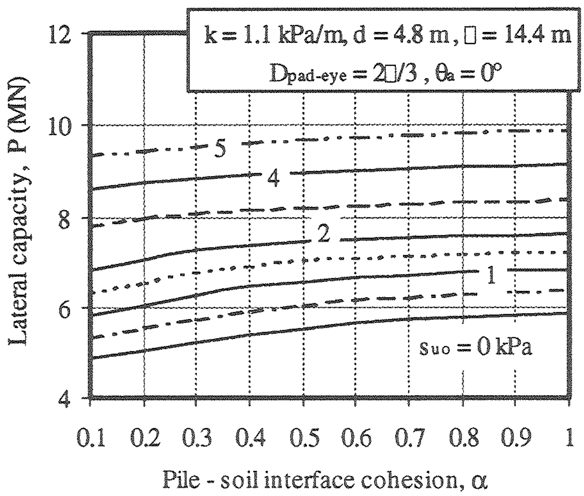


Figure 11 Influence of mudline shear strength

From the above two figures, it is apparent that for the experimentally modelled caisson, the predicted upper bound to the holding capacity is relatively insensitive to the caisson-soil interface cohesion ratio, α.

6 CONCLUSIONS AND FURTHER WORK

The results presented herein support the theory that the least upper bound plasticity solution provides a good prediction to the holding capacity of suction caissons subjected to quasi-lateral loading. Further refinement of the model will facilitate the prediction of caisson performance when subjected to load inclinations typical of those experienced when suction caissons are used for taut wire or fibre rope moorings.

More data are required on the performance of suction caissons in a range of soil profiles before a design methodology may be developed to assist in the optimal suction anchor selection for a specific requirement.

Kinematic data are essential in identifying the optimal pad-eye attachment depth on the anchor. Other proposed work at UWA includes a series of 3-dimensional finite element analyses to calibrate the theoretical load displacement response of caissons subjected to quasi-lateral loading and the predicted failure geometry of the soil with experimental data.

7 ACKNOWLEDGEMENTS

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A MICRO-MECHANICAL APPROACH INTO THE SHEAR BEHAVIOUR OF ROCK JOINTS

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Summary: Current methods to estimate the peak shear strength of rock joints rely predominantly on empirical relationships. The empirical nature of these models requires conservatism on the part of the designer. The Geotechnical Group at Monash University is currently developing a theoretical approach where the behaviour of the rock joint is being modeled. By understanding the mechanisms involved during shearing, it is hoped that the uncertainty of design can be minimized. This paper outlines the difficulties in correctly modeling rock joint behaviour and gives a brief outline of the theoretical approach being taken.

1 INTRODUCTION

Rock masses are inherently intersected by discontinuities that are typically the weakest part of the rock mass and hence may govern its strength. The presence of these discontinuities has been known to cause catastrophic failure in many engineering structures where the design has not incorporated accurate properties of these defects.

To understand the mechanisms of failure within the rock mass, it is first necessary to understand the behaviour of a single discontinuity. Considerable research has been undertaken during the last three decades to determine the behaviour of rock joints under the application of shear load. Much of this research has involved the use of empirical models due to the complexity of the joint interface and shearing behaviour.

This paper outlines the complexities of the joint shearing process and the many factors that may affect its behaviour. A theoretical approach to rock joint behaviour that is currently being investigated by Monash University is discussed.

2 FACTORS AFFECTING ROCK JOINT SHEAR BEHAVIOUR

To accurately predict the shear response of a rock joint, it is important to identify and understand the many factors that will affect the joint behaviour. These factors are often very difficult to quantify.

There are many stresses that act on a rock joint. These can be in-situ stresses; such as gravitational stresses, tectonic stresses, residual stresses from such processes as magma cooling, terrestrial stresses such as seasonal variation; or induced stresses from such processes as mining, drilling or pumping. These stresses are often difficult to accurately measure and with little to compare answers to, unavoidably require large confidence intervals.

The boundary conditions for a rock joint vary according to the deformability of the surrounding rock. The deformability of the surrounding rock can be measured in terms of its stiffness. If the rock surrounding the joint is deformable enough to allow dilation of the joint without reaction forces, then shearing will take place under zero normal stiffness. An example of this would be a rock slope where sliding along a joint occurs under the constant normal load of the block's weight. In most underground rock situations and in rock slopes where rock bolts or cables have been used, rock blocks cannot slide freely due to intimate contact with surrounding rock blocks. Dilation of the joint causes a reaction from the surrounding rock mass that applies additional stress to the rock joint. This is known as Constant Normal Stiffness. It is also possible to have conditions where the normal stiffness may vary.

Rock joint surfaces possess many irregular departures from the plane both on the large scale and on the small scale. These departures of varying angle and width (referred herein as asperities) are dependent on the rock's mode of origin, weathering and mineralogy. Due to the random and irregular nature of the asperities, it is difficult to assign a statistical value to represent the roughness of the surface. This is made more difficult as the roughness is scale dependent. Roughness measured on a 10 cm core sample may not indicate the overlying waviness visible when looking at an exposed joint surface from some distance. At very small stress levels or very small scales the grain size can even be seen to affect the shear behaviour. To further complicate a roughness analysis, the joint surface roughness will also typically decrease during shearing as steeper asperities are sheared.

Shearing of asperities may be dependent on the rock strength. If the rock joint is not weathered then the joint wall strength is that of the intact rock. However, weathering of the joint surface will alter the joint wall strength properties.

Weathering will typically weaken the joint wall strength although in some cases such as the occurrence of iron penetration, the joint wall strength can be stronger than the strength of the intact rock. Laboratory testing can be performed to determine the strength of the joint surface although very small depths of penetration often make this testing very difficult. A field tool, called the Schmidt Hammer, has been developed to provide characteristic data. It involves dropping a spring-loaded plunger onto the joint surface and recording the number of rebounds. From an average of at least ten tests the surface unconfined compressive strength can be estimated. Damage to the surface can cause an inaccurate measurement as can voids or compressive layers hidden in the rock mass and therefore, unless calibrated extensively, the results should not be relied upon.

Joint aperture, a measure of the perpendicular distance separating the adjacent joint faces, can affect the shear response as partially open joints only allow the higher asperities to come into contact and hence become involved in the shearing process. Measuring the aperture can be quite difficult as typical drilling techniques would tend to disturb the joint interface.

Many joint walls are separated by material that will typically reduce the shear strength of the joint (such as carbonaceous material, clay, silt, breccia or minerals). The effect on the shear behaviour will be dependent on the infill's thickness, composition, water content, extent of consolidation, previous shear history and roughness of the joint walls. Laboratory direct shear tests have indicated that a rock joint containing infill will produce two peak shear strengths – failure of the infill followed by failure of the asperities (Checcia de Toledo and de Freitas, 1995).

Laboratory testing has indicated that the shear displacement velocity can affect the magnitude of the shear resistance at stress levels applicable to engineering structures (Crawford and Curran, 1981). The extent of the effects appears dependent on the rock type and level of the normal stress.

The presence of water in the joint has been documented to either reduce, increase or have no effect on the shear strength of the rock joint (Barton, 1973). These results are dependent on the mineralogy and roughness of the joint surface and the resulting development of pore pressures.

In the determination of a realistic but useable rock joint model capable of accurately predicting the performance of the joint under the application of a shear load, an understanding and method of quantifying all factors affecting the joint is required.

3 TRADITIONAL APPROACHES TO ROCK JOINT SHEAR STRENGTH MODELLING

During the past three decades extensive research has been conducted on the behaviour of rock joints under the application of a shear load.

According to the classical law by Amonton, the shear resistance, τ, is related to the normal stress, σ_n, and the coefficient of friction, μ, by the following relationship:

$$\tau=\mu\sigma_n \qquad (1)$$

where $\mu=\tan\phi$
ϕ=friction angle of the material

By investigating over 300 slopes in the Rocky Mountains and performing laboratory tests on synthetic rock profiles with constant angled asperities, Patton, (1966) highlighted the presence of two failing mechanisms - sliding and shearing. He produced a bilinear failure model whereby under a predetermined transition stress, σ_T, sliding occurs and above this stress shearing occurs. He extended Amonton's shear stress relationship to include the asperity angle, i, as shown in equation (2).

$$\tau=\sigma\tan(i+\phi) \qquad (2)$$

Although this model highlights two basic mechanisms of rock joint behaviour, it is simplistic in that real rock joint surfaces are irregular, comprising many different asperity angles. This allows shearing and sliding to occur simultaneously and would produce curved failure envelopes.

Given the non-linear failure envelopes of natural rock joints, Barton, (1973) believed that empiricism was required to correctly describe the shear strength. He proposed the following relationship based on extensive testing and observations of rock joints:

$$\tau=\sigma_n \tan(JRC \log_{10}(JCS/\sigma_n)+\phi_b) \quad (3)$$

where τ = peak shear strength
σ_n = effective normal stress
JRC = joint roughness coefficient
JCS = joint compressive strength
ϕ_b = basic friction angle

This equation can be compared to the Patton model with the asperity angle, $i=JRC\log_{10}(JCS/\sigma_n)$.

The JCS is a measure of the joint compressive strength and can be roughly measured using the Schmidt Hammer. This takes into account any alterations to the joint surface.

The JRC represents a sliding scale of roughness where the roughness is visually assessed from ten roughness profiles. These profiles have been given the ranges 0-2, 2-4, etc up to 18-20 and are 100mm long. Scale effects are therefore not considered in the measurement of the JRC. A further empirical relationship was later added to the model to take into account scale effects.

The standard roughness profiles together with the empirical relationship has been accepted by the International Society for Rock Mechanics as a useful method to estimate peak shear strength in their Commission on Standardization of Laboratory and Field tests (ISRM, 1978). Due to this endorsement and ease of use, this method has gained significant popularity and is perhaps the most widely used approach today. However, its empirical nature may limit its applicability to all situations and rock types.

In an attempt to produce an understanding of the shear behaviour, Ladanyi and Archambault, (1970) used energy principles to extend Patton's bilinear model to take into account various failure modes of rock joints occurring simultaneously. They considered that the total shearing force would comprise of four components:

- component due to external force done in dilating against the external force, N
- component due to additional internal work done in friction due to dilatancy
- component due to the work done in internal friction if the sample did not change in volume in shear.
- component due to shearing through the base of the asperities

The summation of these components combined with several empirical constants was used to predict the total shear force.

Although introducing combined concepts of shear failure of asperities and dilation, the Ladanyi and Archambault model relies on several parameters that are either difficult to predict or rely on empirical methods. Its approach to boundary conditions, rigidity of asperities and elasticity of the rock is also overly simplified.

4 MONASH APPROACH

The Geotechnical Group at Monash University has for the past 20 years being developing a theoretical approach to the determination of rock socket pile performance. This work has concentrated on developing a micro-mechanical approach to the behaviour of the concrete-rock interface during shear. The model uses several of the energy principles adopted in Ladanyi and Archambault but is based on a more fundamental approach without the use of empiricism. The model is reported in detail in Seidel, (1993).

It was considered that the shear model developed for concrete-rock interfaces could be modified to represent rock joints. Initial testing has suggested that the main mechanisms are consistent (Fleuter, 1997). A simplified description of the model details will be briefly outlined.

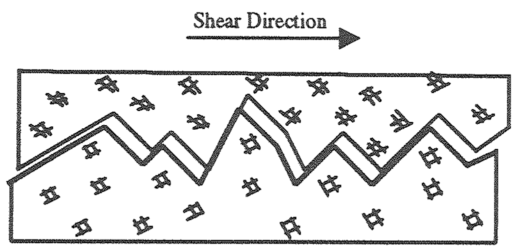
The model represents the roughness as a series of irregular triangular elastic asperities. When a shear force is applied, sliding will initially occur on the steepest asperity slopes. As the asperities are elastic (in particular with soft rock) sliding will also occur on several of the lower sloped asperities. Sliding on these asperities causes dilation and other asperities are lifted out of contact. This reduces the contact area causing an increase of the normal stress on areas still in contact. If the normal stress increases beyond the intact strength of the asperity then the asperity will shear. Further movements in the shear direction will cause a displacement of the sheared material. Due to the irregular nature of the surface, sliding, dilating and shearing can all occur simultaneously. With some materials the surface can also experience inelastic deformations where the surface is worn away. These simplified components are shown diagrammatically in Figure 1.

To date this model has been developed using simplified roughness profiles on either synthetic rock or reasonably homogeneous sandstone samples. To be able to realistically extrapolate the model to real rock joints of varying rock type and strength, further detailed laboratory testing is currently being undertaken.

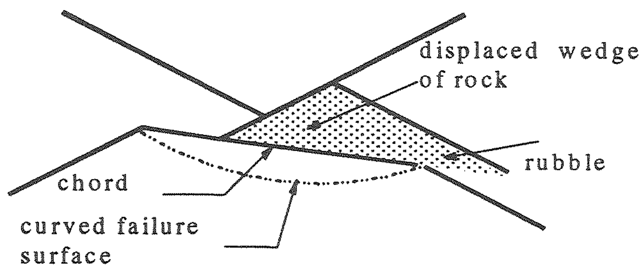
5 TESTING OF MODEL

Most of Melbourne is underlain by Silurian and Lower Devonian Age siltstones, sandstones and mudstones called Melbourne Mudstone. This formation can be encountered in a range of weathering states that can vary from very low strength rock to very high strength in its fresh state.

Due to its importance in the Melbourne area, initial testing was conducted on a synthetic rock called Johnstone that has similar properties to Highly to Moderately Weathered siltstone. This rock was developed specifically for testing of the concrete – rock interface in pile sockets (Johnston and Choi, 1986). It has an unconfined compressive strength of approximately 4MPa when its saturated moisture content is approximately 14%.



Sliding and Dilation



Shear Failure of Single Asperity

Figure 1 Idealised sketches of sliding, dilation, shearing

Direct shear tests conducted on intact and planar Johnstone samples obtained peak and residual friction angles of 35^0 and 24.5^0 respectively. Tests were undertaken on regular triangular 2-dimensional profiles to be able to observe and model the sliding, shear and wear behaviour. This was then extended into 2-dimensional irregular triangular profiles. All direct shear tests were conducted in a specially designed device that is capable of testing samples up to 600mm long under constant normal stiffness conditions.

These direct shear tests indicated that sliding was initially occurring on the steepest asperities and on some shallower asperities due to elasticity effects. This caused dilation and some asperities moved out of contact. Due to the increase in load on the steeper asperities, failure occurred transferring the load to lower asperities. Due to the irregular nature of the surface, sliding and failure was observed to occur simultaneously. Later inspection of the surface also indicated wear was occurring. With these observations, the basic model developed for concrete-rock interfaces could be extrapolated to rock joints (Fleuter, 1997).

Recent testing has extended this work into 3-dimensional profiles. These profiles were obtained by tensile splitting Johnstone blocks. Several 2-dimensional profiles were taken of each split surface using laser profilometry techniques. The laser device used was the Monash *Socket-Pro* that was developed for pile socket roughness inspections (Collingwood et al., 1999).

A visual comparison of the profiles from each surface indicated a close similarity across the face. Several of the 2-dimensional profiles for one split face are shown in Figure 2.

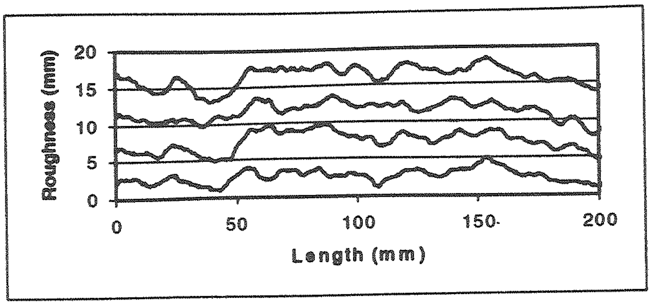


Figure 2 Several laser scans from a Johnstone split face

A statistical analysis was also completed by using the compass walking method. This method, which has been suggested as one method to estimate the fractal dimension of the joint roughness, involves walking a compass opened to a specified chord length along the profile. Considerable debate still rages over the application of this method, amongst others, in determining accurate fractal dimensions. As a result, analysis has been restricted to the determination of the variation of the standard deviation of chord angle with chord length. Correlation coefficients determined for the between each profiles range of standard deviation of chord angle with chord length were in the order of 0.99. This indicates excellent correlation.

These comparisons have suggested that the 3-dimensional surface can be modeled using a 2-dimensional profile. Limited laboratory testing has agreed with these comparisons although further work will be required to confirm this. By testing Johnstone, a synthetic rock, problems associated with natural variations in the material are avoided allowing the basic model to be developed. Testing is currently being conducted on natural Slightly weathered to Fresh Melbourne mudstone. Samples were obtained from the tunneling work conducted for the Melbourne City Link project. The rock obtained has an unconfined compressive strength of approximately 60MPa and saturated moisture content of approximately 1.5%. These samples are therefore considerably stronger than previous samples tested in the development of the rock socket model.

A series of direct shear tests have been conducted on regular triangular asperities of chord length 16mm at angles of 5^0,10^0 and 15^0. The profiles were cut using waterjet cutting techniques. Waterjet cutting involves spraying water mixed with fine sand through a nozzle under high pressure. The resulting jet is approximately 1.2mm in diameter. The shear test results indicated that under the normal stresses applied (up to 3000 kPa), sliding on the asperity faces occurred with no shearing and very little wear. An example profile and the resulting direct shear test results are given in Figure 3.

Tests were also conducted on 2-dimensional irregular triangular profiles. An example profile together with the direct shear test results is shown in Figure 4. Some shearing of the higher asperities can be seen in these results. Dilation caused some of the interface to move out of contact causing the stresses became highly localized. When these were greater than the strength of the intact rock shearing of the asperity occurred. The results from these tests can be used to confirm the basic model.

Several blocks of siltstone have been split parallel to their bedding direction These surfaces have been compared to natural bedding joints in the slightly weathered to fresh siltstone. The split surfaces were visually and statistically similar to the natural bedding joints. Direct shear tests will be conducted on these samples to extend the existing model into more realistic 3-dimensional profiles.

Siltstone 5^0, 16mm chord length, initial normal stress=800kPa, normal stiffness=800kPa/mm

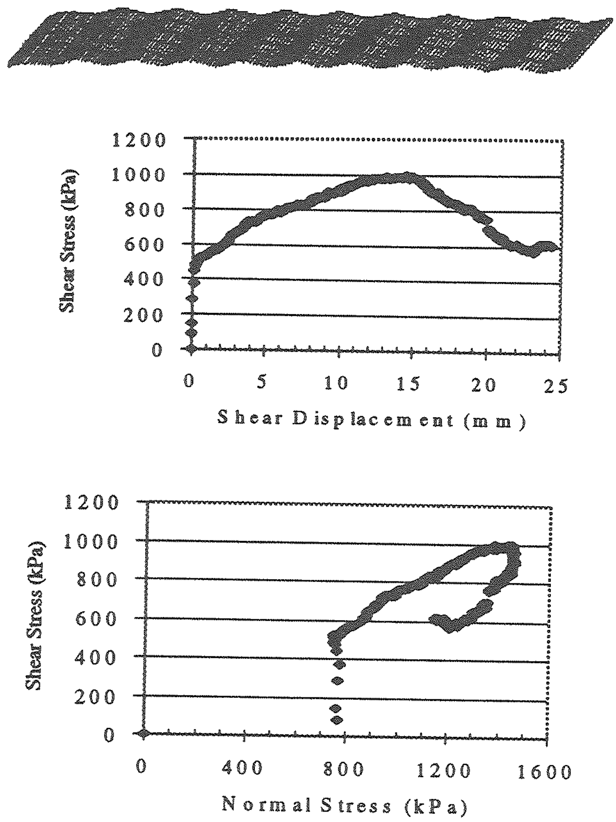


Figure 3 Shear results of 5^0 siltstone sample

6 CONCLUSIONS

Empirical models to predict the peak shear stress of natural jock joints fall short of explaining the behaviour of the joint. Although simple to use, they cannot be confidently extended to all rock joint situations thus requiring considerable conservatism.

The Geotechnical Group at Monash University is currently investigating a theoretical approach to model the behaviour of a rock joint under the application of a shear load. This work is being extended from previous investigations that were related to the performance of pile rock sockets. Testing to date has indicated that the model can be modified for rock joint analysis.

To maximize the potential application of the proposed model, confidence is required in its ability to model all situations. To provide this level of understanding, considerable testing of the many factors that effect the rock joints behaviour is still required. Once this level of understanding is achieved, it is hoped that a computer aided design tool can be developed.

7 ACKNOWLEDGEMENTS

The work described in this paper forms part of the ongoing research into the behaviour of rock joints under the application of a shear load funded through the Australian Research Council. The author is financially supported by the Australian Postgraduate Award Council. Grateful acknowledgement of the support provided by the Department of Civil Engineering, its technical staff and in particular my supervisors Chris Haberfield and Julian Seidel is also given.

Siltstone standard deviation angle = 5^0, 5mm chord length, initial normal stress=800kPa, normal stiffness=800kPa/mm

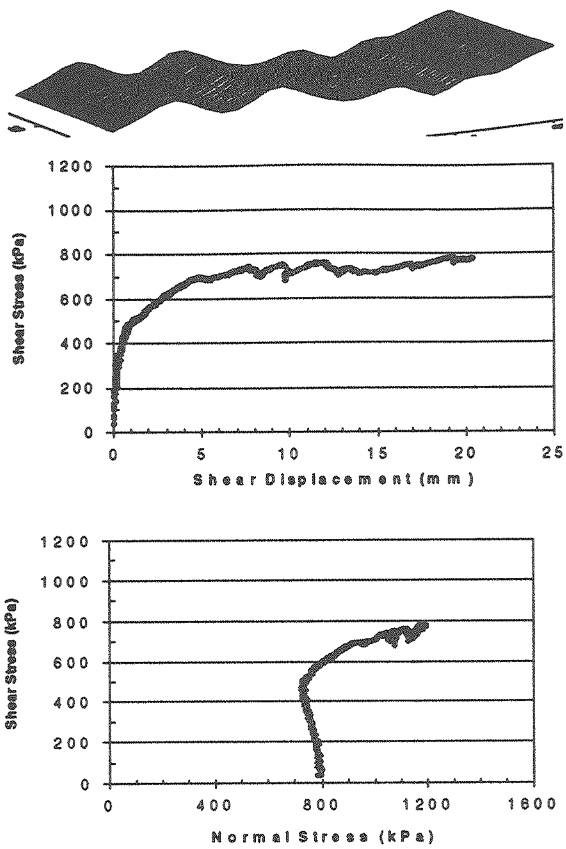


Figure 4 Shear results of s=5^0 siltstone sample

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A NEW MODEL FOR THE BEHAVIOUR OF GRANULAR FILTERS

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ABSTRACT

Filters are used in Geotechnical Engineering to control seepage and to prevent erosion of soil due to the drag forces of seeping water. Filters act as barriers to retain the base soil while allowing seepage flows to exit without causing high hydraulic gradients or pore pressures which may damage the structure. This paper describes a new analytical model of filtration. The model is based on a three dimensional network model of the filter pores, and the equations of conservation of mass and momentum which govern the rate of particle transport. The model has application in the design of granular filters for protecting non-cohesive base soils in embankment dams, retaining walls, drainage wells or road pavements.

1 INTRODUCTION

Filters are used, in geotechnical engineering, where it is necessary to protect soils from erosion due to seepage and groundwater. As water flows through a soil, particles of the soil can be washed out, leading to internal erosion (or piping) and eventual failure. A correctly designed filter will retain the eroded soil particles while allowing seepage water to flow; thus preventing piping and avoiding a build up of high internal pore pressures. Filters are used in embankment dams, road pavements, behind retaining walls, coastal protection, in landfills and wastewater treatment, sand beds in oil wells and chemical engineering filtration. This study deals predominantly with the problem of granular filters for embankment dams.

Filters are used where water seeping out of fine grained soils may cause erosion of the soil, by removing particles under hydraulic forces. To function correctly, filters must be:

1 Fine enough that the pore spaces between the filter particles are able to capture some of the larger particles of the protected material (see Figure 1).
2 Coarse enough to allow seepage flow to pass through the filter, preventing the build up of high pore pressures and hydraulic gradients.
3 Non cohesive, so that no cavities or cracks can form within the filter.

Figure 1 shows a stable base - filter interface. Seepage forces have washed some base soil particles into the filter. Initially, some fine base particles may be washed completely through the filter, but in a stable filter the larger base particles will be trapped by the void constrictions of the filter material. The void constrictions are the smallest cross section between two voids, hence, particles smaller than the constriction are unable to pass through to the next void. These trapped particles will then form smaller voids, retaining smaller base particles and the entire interface becomes stable. This process is called "self filtration". If a filter is too coarse, the base soil particles will be able to move through the pores of the filter material and self filtration will not occur. If a filter too fine, it may not have sufficient permeability to allow the seepage flows to leave the base soil and high pore pressures can develop. Also, manufacturing a fine filter is often considerably more expensive than a coarse filter; hence, the economic benefits of correct filter design are significant.

There is an increasing push to replace granular filters with geotextiles which perform the same function. The advantages of geotextiles are numerous, often they are cheaper to install than granular filters and they are manufactured and placed under strict specifications, so the uncertainties involved with using natural materials are removed. However, there is still a concern that the long term performance of geotextiles (remembering a dam usually has a design life in excess of 50 years) may be unsatisfactory. A particular concern is that a geotextile may tear due to differential settlement within the structure, or earthquake motion. Because of these concerns, granular filters are more commonly used in important structures such as embankment dams. This study will focus solely on the performance of granular materials as filters.

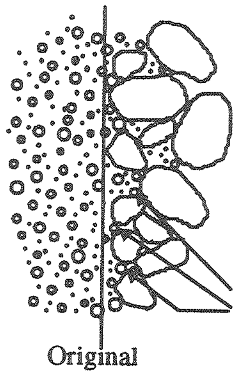

Figure 1 Stable Base-Filter Interface During Seepage (Indraratna and Locke, 1999).

2 EXISTING DESIGN METHODS

Terzaghi (1922) was the first to develop filter design requirements. He envisaged two requirements that must be fulfilled:
The filter should be many times more pervious than the base soil, to allow the free seepage of water, without causing excessive head loss (the permeability requirement). To ensure this he recommended:

$$D_{15F}/d_{15B} \geq 4 \tag{1}$$

The filter should be fine enough to prevent the washing through of the base soils and arrest piping (the retention requirement). To ensure this he recommended:

$$D_{15F}/d_{85B} \leq 4\text{-}5 \tag{2}$$

Where D_{15F} is the diameter of filter particles where 15% by pass of particles are smaller, and d_{85B} is the diameter of base particles with 85% of the particles, by mass, smaller. These requirements describe, basically, the conflicting requirements on grain size, of a suitable filter. Some engineers still use these criteria for designing granular filters.

Subsequent research into filter behaviour has been predominantly empirical; a series of experiments on sets of base soil - filter combinations, has lead the researcher to recommend an empirical relationship for a stable combination. Research has lead to empirical design criteria that provide simple to apply relations for stable base soil - filter combinations. The most widely accepted empirical criteria are those of Sherard and Dunnigan (1985). A review of the application of empirical methods can be found in Indraratna and Locke (1999). These empirical criteria are extensively used, in preference to other methods, for filter design. However, they are only applicable to the range of soils tested, and have certain laboratory bias due to different testing methods, definitions of failure etc. Applying empirical criteria does not provide an understanding of the mechanisms involved with base soil - filter interaction. Hence, these methods do not give the designer a clear picture of what may occur within the dam and the level of safety involved with design decisions.

Many researchers are now concentrating on numerical analysis of filtration, particularly modelling particle movement through filters. These approaches recognize that soil masses are made up of a random distribution of many sized particles. The most important part of base soil movement through a filter is the geometric requirement, that a base soil particle must be smaller than the pore void (and void constriction joining pores) through which it is passing (Silveira, 1965). Additionally, some researchers have considered the hydraulic conditions (seepage forces) necessary to carry soil particles through the filter (Indraratna and Vafai, 1997). The basis of the numerical analysis is:

to represent the filter by some form of a pore model, usually based on the particle size distribution of the filter material;
to simulate the movement of base soil particles by an analysis of the movement of individual base soil particles through the pores of the filter, caused by seepage forces, up to a point where the particle passage is blocked by a pore constriction, or the seepage forces are insufficient to move the particle further.

There are two general approaches to modelling particle movement, either: geometric - probabilistic, where the expected depth of infiltration of a particle, into the filter, is determined by probabilistic analysis of particle and void sizes (Schuler, 1996); or mass transport equations using flow laws and conservation of mass and momentum to examine the rate of particle movement (Indraratna and Vafai, 1997). Analytic methods provide detailed models of what may be occurring at a base soil - filter interface. They give an idea of the thickness of filter required and also can estimate a probability of failure. The assumptions used in developing the model are very important. Often the assumption of spherical particles or a certain particle size distribution curve shape etc. cannot be applied to a real soil. The models are often difficult to apply to real design situations because of their reliance on a number of empirical parameters or impractical mathematical models.

3 NEW ANALYTICAL FILTER MODEL

Existing numerical models of filtration have some limitations. They generally adopt simplified void models, and very few consider the time rate of formation of a stable filter interface. Indraratna and Vafai (1997) have developed a particle migration model, considering conservation of mass and momentum to model particle movement. This model is capable of showing the variation with time of particle size distribution, permeability and porosity of elements of the base soil and filter. Some criticism has been directed at the simplified void model adopted in this analysis. Hence there is some room for improvement and adaptation of this method. A new model for filtration is described below, based on the model of Indraratna and Vafai (1997), and a modified three dimensional pore void model, based on the work of Schuler (1996). The entire model includes:

Filter void model - based on a cubic network described by Schuler (1996). Void sizes are determined by an adaptation of the method described by Silveira (1965).

Particle infiltration depth - Schuler (1996) developed an equation, based on Monte Carlo simulation, for infiltration depth, dependent on particle size.

Particle transport equations - the equations of conservation of mass and momentum, developed by Indraratna and Vafai (1997), are used to determine the rate of particle transport.

3.1 FILTER VOID MODEL

There are many models of filter voids which have been adopted in modelling filtration. The commonly used models include; parallel channels of varying diameter (Indraratna and Vafai (1997)), layers of filter voids perpendicular to the direction of flow (Silveira (1965)), or pore networks (Schuler (1996)). The three-dimensional, random arrangement of pores and constrictions in a natural soil is most accurately modelled by a regular three-dimensional network of pores interconnected by constrictions. Schuler (1996) suggests that after examination of the pore voids of a real soil, there are on average 5.7 constrictions from every pore. Based on this, Schuler (1996) developed a regular cubic network model of pores and constrictions, shown in Figure 2.

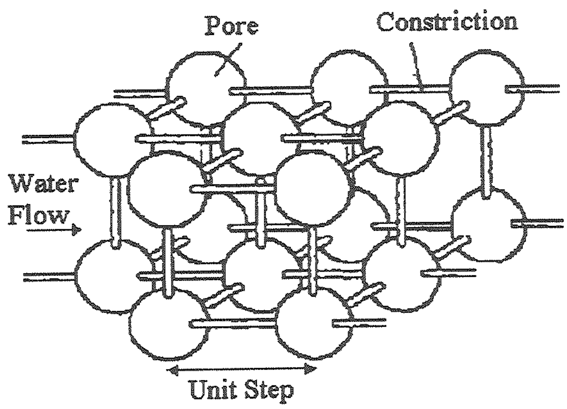

Figure 2 Cubic network void model (Schuler, 1996)

It remains then, to determine the size of the voids and void constrictions in the void model. The void constrictions, represented as bonds between the voids in Figure 2, form the smallest link between voids, capturing moving particles. Hence the important factor for modelling filtration is the void constriction size distribution, hereafter called the CSD. Schuler (1996) has examined the CSD of a soil at varying relative density and found that the CSD curves all have the same shape. Hence, if we find the CSD for most dense and least dense states, then the actual CSD will have the same shape and lie somewhere in between.

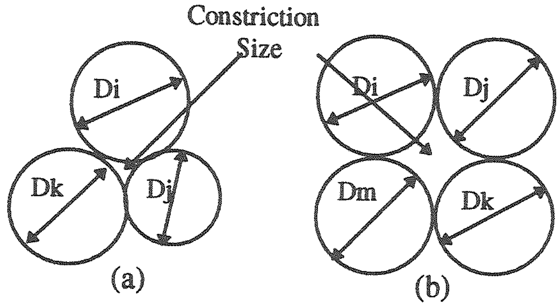

Figure 3 Void Constriction Size for a) Most Dense and b) Least Dense states (Indraratna and Locke, 2000).

The two geometric conditions to be considered, are shown in Figure 3. Humes (1996) presents a method to calculate the CSD for the most dense case (Figure 3a), based a method described by Silveira (1965) using the filter PSD. The PSD by mass (as determined by sieve analysis) tends to over-estimate the influence of larger particles, which form a large proportion of the mass of the soil, but are few in number and unlikely to meet to form large voids. Humes (1996) recommends using the PSD by surface area of grains, and has shown this to better represent the pore constriction sizes for well graded materials. Equation (3) can be used to convert the PSD by mass to a PSD by surface area, assuming all particles have the same specific gravity.

$$P_{i,SA} = \frac{P_{i,mass}/D_i}{\sum_{j=0}^{n} P_{j,mass}/D_j} \tag{3}$$

The PSD is divided into a number of discrete particle diameter intervals (D_0, D_{10}, D_{20}, etc.), so that it then represents the cumulative frequencies (P_0, P_{10}, P_{20} etc.) of the medians of these intervals. The theory of standard mean error can be used to find the void size, D_v, for the most dense particle packing (Equation 4). This is the case where the void is formed by three tangent spheres of diameter D_i, D_j and D_k, as shown in Figure 3a.

$$\begin{aligned}&\left(\frac{2}{D_i}\right)^2+\left(\frac{2}{D_j}\right)^2+\left(\frac{2}{D_k}\right)^2+\left(\frac{2}{D_v}\right)^2\\&=0.5\left[\left(\frac{2}{D_i}\right)+\left(\frac{2}{D_j}\right)+\left(\frac{2}{D_k}\right)+\left(\frac{2}{D_v}\right)\right]^2\end{aligned} \tag{4}$$

The probability of occurrence, P_v, of void size D_v, is a function of the probability of occurrence of the three particles, taken from the discretised PSD. P_v is calculated using (5), where r_i, r_j and r_k represent the number of times particle diameters D_i, D_j, and D_k appear in the combination of three particles being considered. Hence r_i, r_j and r_k = 1,2 or 3 and $r_i + r_j + r_k = 3$.

$$P_v = \frac{3!}{r_i!\,r_j!\,r_k!}(P_i)^{ri}.(P_j)^{rj}.(P_k)^{rk} \tag{5}$$

Silveira et al. (1975) present equations for the least dense packing of a granular material; where the void constrictions are formed by four tangent grains as shown in Figure 3b. Silveira et al. (1975) note that the geometry of the problem is very difficult to solve directly. An easier method is to assume the void is equivalent to a circle with the same area as that formed by four tangent particles as shown in Figure 4b and c. The Silveira et al. (1975) equations for the diameter of the equivalent circle are presented in Indraratna and Locke (2000), and will not be repeated here.

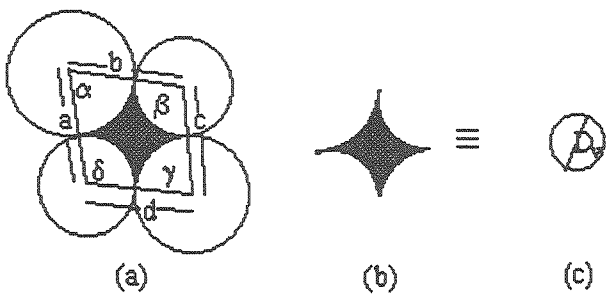

Figure 4a) Pore constriction formed by 4 particles b) Constriction Area formed by tangent particles c) Circle of equivalent area (after Silveira et al.,1975)

In the least dense packing arrangement, pore constrictions usually do not form on a plane through the centres of the four particles making up the constriction. Hence, it is suggested that the mean of all possible chords through the circular particle be used to represent the size of particles, rather than the diameter. Thus the particle diameter to be considered for determining constriction size, D_{model}, is given by Equation (6).

$$D_{model} = 0.82\ D_{actual} \tag{6}$$

The CSD of a granular material, at different relative densities, will have the same shape as the most dense and least dense CSD (Schuler, 1996). The assumption is made that the difference between two constriction size distributions will be directly proportional to the difference in relative density. Hence, using relative density, defined in Equation (7), then the actual CSD can be calculated from Equation (8). The CSD is divided into n discrete portions. The integer i represents these discrete portions of the CSD such that i/n is the fraction of constrictions finer than constriction diameter $D_{v,i}$. We then have a pore void model, consisting of a 3D cubic network of pores with six constrictions connecting each pore to its neighbours, as shown in Figure 2. The size of each constriction is randomly generated from the CSD.

$$RD = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{7}$$

$$D_{V,i} = D_{VMD,i} + \frac{i}{n}(1 - R_d)(D_{VLD,i} - D_{VMD,i}), i = 1,2,..n \tag{8}$$

where, $D_{VMD,i}$ and $D_{VLD,i}$ are the i/n% coarsest constrictions from the most dense and least dense constriction size distributions respectively.

3.2 PARTICLE INFILTRATION DEPTH

Having defined a model of the filter voids and void constrictions, it remains to examine how far a base particle can infiltrate into the filter before being captured. Here a deterministic equation is described for the expected infiltration of base particles, based on a probabilistic analysis of particle and constriction sizes. For a base particle of diameter d, the probability, p, of the particle passing a single random constriction is the cumulative probability of larger void constrictions from the CSD (ie. the percent larger than d on the CSD). Based on the probability p, the probability of a particle passing one layer in the direction of water flow, through the three dimensional network is given by Equation (9). This equation is developed based on the combined probability of a forwards step, in the direction of flow, through the model, after any number of movements perpendicular to the flow direction.

$$P(F) = p + \sum_{i=0}^{\infty}\left[1-(1-p)^4\right](1-p)p\left\{\left[1-(1-p)^3\right](1-p)\right\}^i \tag{9}$$

Silveira (1965) has used an absorbing Markov chain process to determine the number of confrontations, n, with randomly generated pore constrictions, required to stop a particle moving forwards through the filter, with a confidence level $\overline{P}$. This equation can be adapted to consider the number of layers, n, a particle can move through the pore network model (Figure 2), based on the probability of passing one layer P(F). The number of layers is given by

Equation (10). A confidence level of $\overline{P}$=95% has been adopted in modelling, as this gives a conservative estimate of the depth a particle may infiltrate into the filter.

$$n = \frac{\ln\left(1-\overline{P}\right)}{\ln P(F)} \tag{10}$$

The spacing between layers in the network is the *unit step* between confrontations. A particle encounters a constriction, and then moves into the next pore, where it will encounter another constriction at the exit of the pore. Since particles will meet a constriction and then pass approximately two half diameters to the next constriction, it seems reasonable to adopt the mean filter particle diameter (determined by number of particles, not mass), $D_{f,mean}$, as the unit step. Hence, the length of infiltration, L, is given by:

$$\mathrm{L} = \frac{\ln\left(1-\overline{\mathrm{P}}\right)}{\ln \mathrm{P(F)}}.\mathrm{D}_{\mathrm{f,mean}} \tag{11}$$

3.3 PARTICLE MIGRATION MODEL

Indraratna and Vafai (1997) have developed a particle transport approach to model particle movement. They also consider the hydraulic forces required to mobilise the particles. If seepage forces exceed the critical hydraulic gradient and the particle is smaller than the pore constriction, it will move. Moving particles are controlled by governing differential equations of conservation of mass (12) and momentum (13).

$$\frac{d(\rho_m u)}{dz} = \frac{d\rho_m}{dt} \tag{12}$$

$$\sum F = \rho_m V_m \left(\frac{du}{dt} + u\frac{du}{dz}\right) \tag{13}$$

By considering a number of elements at the base - filter interface, the movement of particles can be modelled by a forward step, finite difference analysis. The rate of particle erosion and movement is governed by (12) and (13). The geometric constraint to movement is modelled by the depth of infiltration into the cubic network (11). If the predicted infiltration, L, of a particular particle size is equal the length from the filter interface to the end an element then particles smaller than that diameter can pass through the filter element; larger particles will be captured. The base and filter particle size distributions can be recalculated at each time step and the procedure repeated. This analysis predicts the gradual change in particle size distribution of the base and filter elements and hence describes what is occurring at the base - filter interface with time for the entire particle size range. Indraratna and Locke (2000) describe the procedure in greater detail, including a method to estimate the change in permeability and porosity of the base and filter materials during filtration. Hence, the model is able to describe the time dependent changes in particle size distribution, mass transfer, flow rate, permeability and porosity of the base and filter materials.

3.4 APPLICATION OF THE MODEL

The model presented in this paper is intended to predict the time rate of particle migration of a non-cohesive base soil through a granular filter. No laboratory data or alternative models are available to verify the particle migration equations proposed by Indraratna and Vafai (1997). However, the geometric model of filter voids can be compared with existing laboratory data and model predictions. A number of models have been proposed for particle infiltration depths. The models considered here are that of Schuler (1996), and Humes (1996). Previous laboratory and analytical research has shown that a filter has a controlling constriction size, d_c* (Kenney et al., 1985). Base soil particles finer than d_c* can pass through a filter of large thickness. As base particles larger than d_c* are considered, their depth of infiltration into the filter decreases rapidly as the particle diameter increases. Witt (1993) also determined equations for the controlling constriction size. Hence, the particle infiltration models should predict a rapid increase in infiltration depth for particles finer than the controlling constriction size. The pore channel model adopted by Indraratna and Vafai (1997) predicts a minimum pore diameter, d_0. Particles smaller than d_0 will pass through the filter element, while coarser particles are retained. This minimum diameter is included in the comparison as a single value. Hence, d_0 can be compared directly with the controlling constriction size. The previous models are compared with the newly developed model for two cases: a uniform sand, with C_u=2 (Figure 5), and a well graded sand, with C_u=6 (Figure 6).

The pore channel model of Indraratna & Vafai (1997) predicted a minimum pore channel diameter larger than other models for the well graded filter. Vafai (1996) has pointed out that the validity of 'd_0' tends to decrease as the C_u value is increased above 6. The Indraratna & Vafai (1997) model assumes nearly spherical particles (shape factor α=6) which decreases accuracy when considering broadly graded filters. For broadly graded materials, the shape factor, α, should be calibrated such that the minimum pore channel diameter is equivalent to the controlling constriction size predicted by Witt (1993) or Kenney et al. (1985). As can be seen, in both examples the model of Schuler (1996) predicts a rapid increase in particle infiltration at diameters slightly coarser the controlling constriction size of Kenney et al. (1985) and Witt (1993). The model of Humes (1996) predicts a significantly lower infiltration depth for the same particle diameters. The current model predicts a rapid increase in infiltration depth (log scale) at a diameter close to the controlling constriction size determined by Kenney et al. (1985) and Witt (1993). Hence, the new geometric model is suitable for modelling infiltration into granular filters.

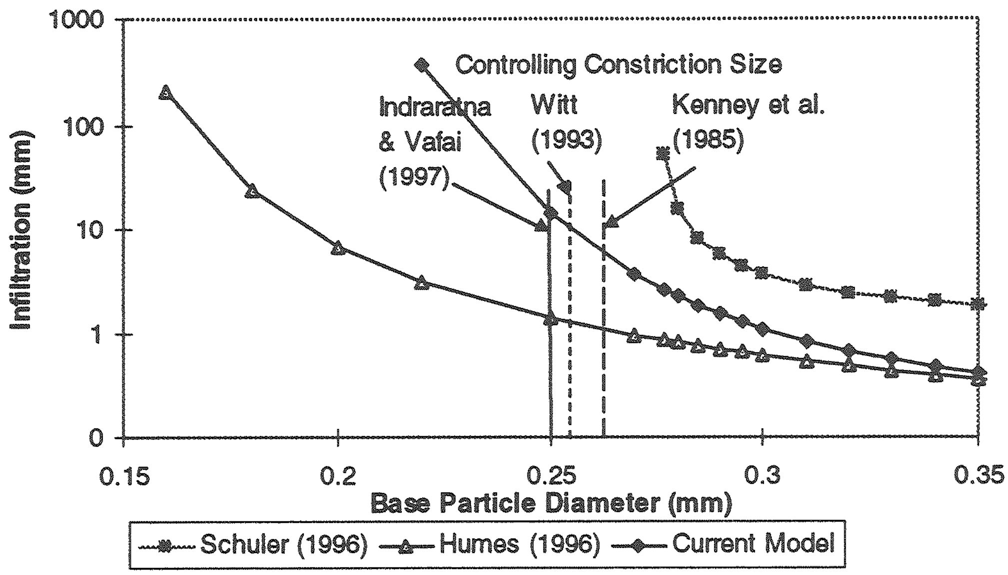

Figure 5 Comparison of Predicted Infiltration Depth of Base Particles into a Granular Filter - Uniform Sand, C_u=2, D_{15}=1.3mm (Indraratna and Locke, 2000)

The model can applied in practice to any geotechnical structure subject to seepage flow. First consider the steady state flow through a structure. Figure 7 shows the predicted flow through a simplified, rectangular (in two dimensions) earth structure with a hydraulic head difference across the structure.

Noting that flow occurs along flow paths, the filtration analysis can be simplified to a one dimensional flow problem. Along any seepage path, the corresponding core-filter system can be divided into a number of one dimensional elements with inclination matching the flow path. The hydraulic pressure and flow rate in each element are determined from the Laplace equation. Then the equations developed with the model described in this paper can be used to describe the movement and capture of particles. A finite difference procedure is used to predict the time dependent particle movements and flow rate, and changes in the base soil and filter permeability, porosity and particle size distribution. As the material permeabilities change due to particle loss and capture, the Laplace equation is applied again to the entire structure to re-calculate the flow rates and pressures.

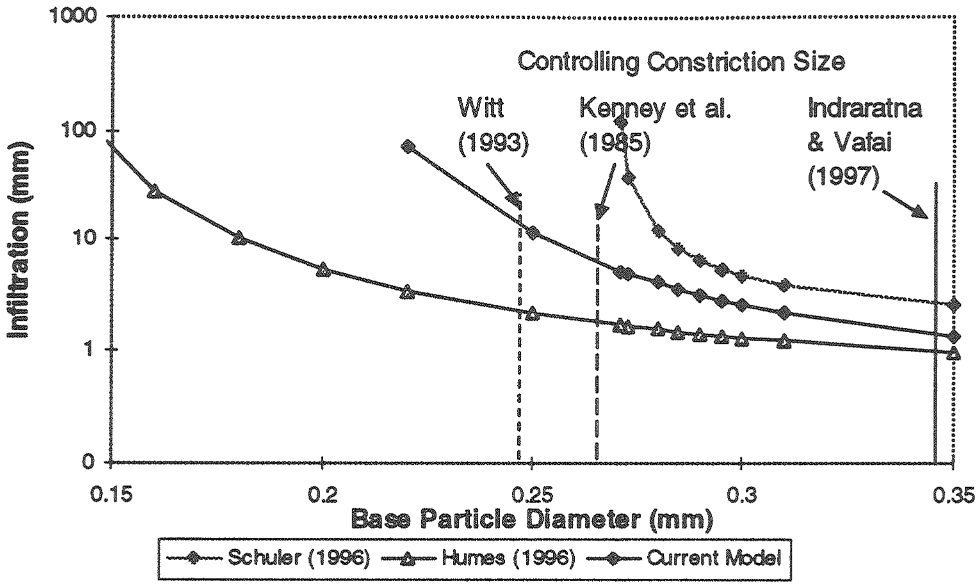

Figure 6 Comparison of Predicted Infiltration Depth of Base Particles into a Granular Filter - Well Graded Sand, C_u=6, D_{15}=1.3mm (Indraratna and Locke, 2000)

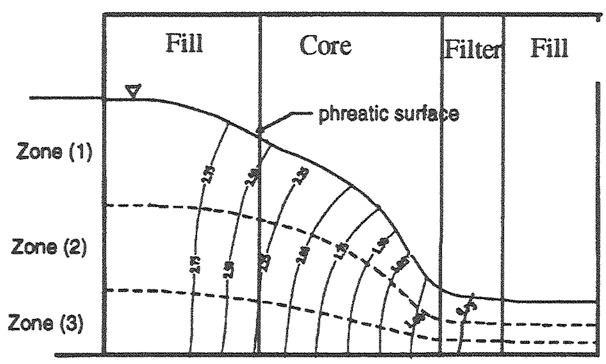

Figure 7 Flow net for a simplified earth structure (Vafai, 1996)

4 CONCLUSIONS

Granular filters are an essential element of earthfill dams, providing protection from erosion and piping to the dam core. The role of a filter is to retain any eroded particles, while allowing seepage water to drain from the base material. This requires careful selection of filter particle size; the pores of the filter must be small enough that the larger base soil particles are captured within the pore constrictions. A new analytic model has been presented which describes the time rate of infiltration of base soil into a filter. The filter pores are modelled by a three dimensional network of pores, connected by constrictions. The size of these constrictions is randomly generated from the constriction size distribution; which is determined from the filter particle size distribution and relative density. The rate of movement of particles is modelled by a finite difference approximation of the differential equations of conservation of mass and momentum. The model has been shown to predict infiltration depths of base soil particles similar to those determined by other researchers, from both experimental and theoretical work. The analytical model presented here has several advantages over previous models, including:

- A three dimensional pore model, more representative of real soil conditions than simple pore channel or layered void models.
- Description of the rate of particle movement. Hence, predicting the time to reach steady state or large scale piping of the base soil.

- Modelling of time dependent changes in the base soil and filter. As particles are eroded or captured they alter the particle size distribution of the soil or filter. By re-calculating the constriction size distribution, porosity and permeability of each element in the model at each time step, the time dependent changes are considered.

5 ACKNOWLEDGEMENTS

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UPLIFT CAPACITY OF SUCTION CAISSONS

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SUMMARY

Prediction of the uplift capacity of suction caissons is a critical issue facing design engineers and rational methods are required in order to produce reliable designs. Extensive theoretical investigations have been carried out of suction caissons subjected to vertical or inclined uplift loading for cases where the behaviour of the seabed soil is undrained, partially drained or drained. A brief literature review on foundations subjected to combined loading is included. Different analytical design models are discussed. Simplified methods for the estimation of the uplift capacity are described, based on the results of the finite element study. The simplified methods are then validated by upper bound theoretical solutions and experiment results. The expressions developed in this paper take into account the influence of the aspect ratio of the caisson, the point of application and angle of inclination of the loading, the undrained shear strength of the soil, the soil permeability and load rate.

1 INTRODUCTION

Compliant offshore structures, like mooring systems and tension leg platforms (TLPs), are usually subjected to considerable uplift forces. These structures require foundations that can anchor them to competent strata and it has been common in the past to use piles to provide such a foundation. However, there are some construction difficulties associated with the installation of the long piles usually necessary, particularly in large depths of water and in some soil type. Largely because of these difficulties a new type of foundation, the suction caisson, has been developed and used to provide uplift resistance, depending on the in situ conditions. A suction caisson, open at the bottom and closed at the top, is designed to penetrate to the sea floor by its own weight and sometimes by also creating an inside under-pressure relative to the outside water pressure. The latter is known as the active suction installation method. As soon as there is any tendency to pullout movement, the suction caisson mobilises significant pullout capacity through the development of negative pore water pressure inside the soil plug and at the bottom of the caisson. This is known as the passive suction condition. The main advantages of suction caissons over tension piles are: the ease of installation of the caissons with the active suction arrangement; the mobilization of passive suction forces at the caisson's bottom during uplift; and the possibility of placing additional ballast on the large diameter sealed top to provide increased pullout capacity. To date, the geometry of suction caissons has tended to involve relatively low aspect ratios, with length less than 3 times the diameter.

The main focus of this paper is on a) suction caissons subjected to vertical uplift loading for cases where the behaviour of the seabed soil is undrained, partially drained or drained, and b) suction caissons subjected to inclined uplift loading for cases where the behaviour of the seabed soil is undrained. The two cases are illustrated in Fig.1. The soil types assumed in this investigation are normally consolidated to lightly over-consolidated cohesive soils.

The theoretical investigations described in this paper have been carried out using a 3-D semi-analytical finite element approach (Taiebat, 1999) incorporated in the software package AFENA (Carter et al., 1995). Simplified prediction methods were developed based on the finite element results. Predictions of the ultimate capacity by the simplified methods have been compared to those obtained independently from upper bound techniques (Randolph et al, 1998) and experimental measurements in 76 individual tests from 12 independent studies. The solutions developed in this study are both for quasi-horizontal and quasi-vertical loading. The developed expressions have also taken into account the influence of the aspect ratio of the caisson, the point of application and angle of inclination of the loading, the undrained shear strength of the soil, the soil permeability and the loading rate.

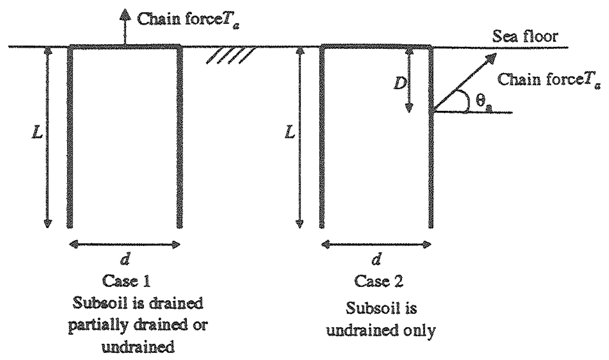

Figure 1 Study cases

2 FOUNDATIONS SUBJECTED TO COMBINED LOADING

The response of a foundation system subjected to combined loading has been a topic of much interest to geotechnical researchers and practitioners, particularly over the last decade. Research in this area has mainly involved investigation of bearing capacity problems. The bearing capacity failure locus for rigid shallow foundations subjected to combined loading has been derived on the basis of experimental results using curve fitting. For example, Butterfield and Gottardi (1995) investigated the behaviour of footings on sand under fully drained or partially drained conditions. Martin (1994) investigated the behaviour of a spudcan footing in undrained clay. In these cases, the footings either had a flat base or a spudcan shape and no consideration was given to the resistance to uplift loading. For these cases, it was found that maximum moments and horizontal loads are sustained in the presence of some vertical compressive load, i.e. typically when $V/V_o \approx 0.4$ to 0.5, where, V is the vertical component of the ultimate load and V_o is the ultimate load for cases of purely vertical loading.

It is evident from many of the previous studies of combined loading that a yield or failure locus relating the vertical (V), moment (M) and horizontal (H) loads at the ultimate condition can often be expressed in the form:

$$f\left(\frac{V}{As_u}, \frac{M}{Ads_u}, \frac{H}{As_u}\right) = 0 \tag{1}$$

where, A is the plan area of the foundation, d is the diameter of the foundation, s_u is the undrained shear strength of the soil at the base of the foundation. It is usually assumed that associated plasticity provides a reasonable description of undrained failure in the soil, so that this yield surface also describes a plastic potential defining the relative magnitudes of the incremental deformation during elasto-plastic yielding. To apply the normality relationships on the yield surface, the load and displacement definitions must form work conjugate pairs so that the normalised total system work, W, is written as:

$$\frac{W}{Ads_u} = \left(\frac{V}{As_u}\right)\frac{\delta v}{d} + \left(\frac{M}{Ads_u}\right)\delta\theta + \left(\frac{H}{As_u}\right)\frac{\delta h}{d} \tag{2}$$

where δv, δh and $\delta\theta$ are the incremental vertical and horizontal caisson displacements and its rotation at failure. These incremental displacements are measured at the same point at which the loads are assumed to act.

In a study of multi-footing foundation systems, Murff (1994) suggested a general form of the failure locus, which included some uplift capacity, given as:

$$f = \sqrt{\left(\frac{M}{d}\right)^2 + \Lambda_1 H^2 + \Lambda_2\left[\left(\frac{v}{v_c}\right)^2 - \left(1 + \frac{V_t}{V_c}\right)V + V_t\right]} = 0 \tag{3}$$

in which V_t V_c are the ultimate vertical compression and tension capacities, Λ_1, Λ_2 are constants and d is the footing diameter. It was assumed that this is a form of associated yield or failure surface. However, as indicated by Bransby and Randolph (1997), the locus described by equation (3) gives very poor agreement with numerical predictions of the collapse loads for strip footings when $M = 0$.

The response of a suction caisson (skirted foundation) to combined vertical (V), moment (M) and horizontal (H) loading has been studied for bearing problems by Bransby and Randolph (1997) using a two-dimensional finite element analysis. In this study, the caisson was considered as a long strip footing and one value of the aspect ratio, $L/d = 0.167$, was investigated in detail. On the basis of the plane-strain finite element predictions, they suggested a yield locus as:

$$f = \left(\frac{V}{V_0}\right)^{2.5} - \left(1 - \frac{H}{H_0}\right)^{1/3}\left(1 - \frac{M}{M_0}\right) + \frac{1}{2}\left(\frac{M}{M_0}\right)\left(\frac{H}{H_0}\right)^{5} \tag{4}$$

However, in practice suction caissons are often circular in plan. To obtain a more precise understanding of the behaviour of a circular foundation under vertical, moment and horizontal loading, a three-dimensional analysis is required. Further, the study by Bransby and Randolph (1997) produced a yield locus for problems involving only bearing (compressive) loads. No consideration of the resistance to uplift loading was included.

3 ANALYTICAL DESIGN MODELS

3.1 BOTTOM RESISTANCE FAILURE

When a suction caisson is pulled out at a rapid rate and when large deformation takes place, it may be completely pullout out with the soil plug inside. In this case, the bottom resistance (passive suction) and whatever tensile strength of the clay may have are fully mobilized at the bottom of the caisson. The pullout capacity in this case is given by the frictional resistance and the weight of suction caisson, plus the resistance at the bottom of the suction caisson (passive suction) (Fig.2). In this case the ultimate pullout force is given by:

$$P_u = F_s + W + R_b \tag{5}$$

where, Fs is the skin friction on the wall, W is the underwater weight of the foundation which includes the soil plug, and R_b is the bottom resistance (passive suction).

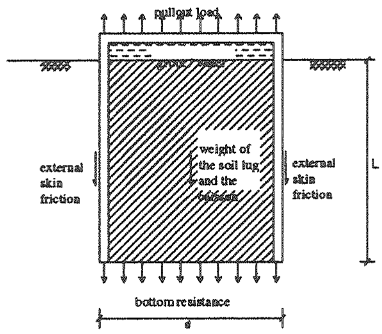

Figure 2 Bottom resistance failure

3.2 REVERSED BEARING CAPACITY FAILURE

Considering the failure in uplift as a reversed bearing capacity problem is a widely used approach for estimating the pullout capacity of suction caissons. This approach was firstly intro-duced by Finn & Byrne (1972) after performing laboratory model tests to understand the factors governing the pullout capacity of suction caissons. This idea was then further verified and enhanced by other researchers (e.g., Andersen *et al*, 1993). The failure mechanism of this model is shown in Fig.3.

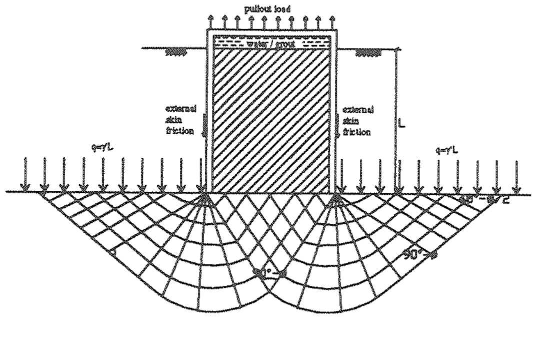

Figure 3 Reversed bearing capacity failure

In this case, the pullout capacity of circular suction caissons under vertical and static loads may be estimated by the following formula:

$$P_u = 1.2AN_c s_u d_c + F_{ts} \tag{6}$$

Where, N_c is the bearing capacity factor with respect to cohesion s_u of the soil. For $\varphi = 0°$ the theoretical value is 5.14 for N_c. d_c is the embedment factor. A is the section area of the caisson. F_{ts} is the friction between the external surface of the caisson wall and the soil.

3.3 SLIDING FAILURE

When a suction caisson is pulled out at an very slow rate, the pullout capacity can be given by the frictional resistance and the weight of suction caisson (Fig.4), such as:

$$P_u = F_s + W_f \tag{7}$$

Where, F_s is the skin friction on the wall, W_f is the submerged weight of the foundation. This failure mode assumes negligible bottom resistance and is more likely to occur under fully drained conditions.

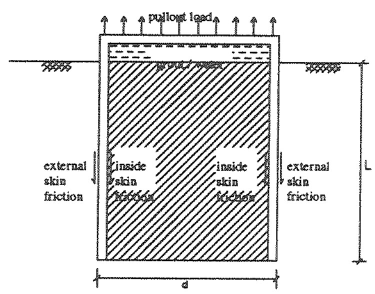

Figure 4 Sliding failure

3.4 CONICAL WEDGE FAILURE

A possible failure mechanism for suction caisson subjected to inclined uplift loading is illustrated in Fig. 5. r_0 is the radius of the caisson (d is the diameter of the caisson), R is the radius of the deforming wedge, Z_0 is the depth of deforming wedge, h is the depth of the centre of the rotation of the caisson, D is the lug depth, and L is the length of the caisson. Near the surface, a deforming conical wedge forms and is pushed laterally and upwards by the translating and possibly rotating caisson. Below the wedge, the soil is assumed to flow horizontally around the caisson. To accommodate this mechanism, the soil wedge must conform to the caisson at the caisson-soil surface, and must move tangentially to the right soil-caisson interface.

This failure mechanism was firstly developed by Murff & Hamilton (1993) for laterally loaded piles and was then used by Randolph et al (1998) for suction caissons subjected to inclined uplift loading. Similar mechanisms have also been detected in the finite element modelling.

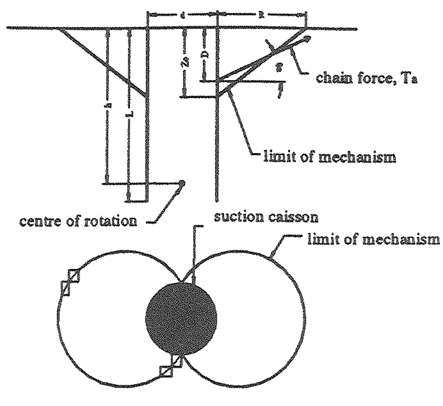

Figure 5 Conical wedge failure

4 THEORETICAL SOLUTIONS

Studies by 2-D and 3-D finite element modelling have been carried out for different aspect ratios of the caissons, different soil strength profiles, different soil permeability and different loading rate. The simplified expressions for the estimation of uplift capacity of suction caisson were then developed by curve fitting the finite element predictions. Details of the finite element study and the detailed derivation of the theoretical solutions have been presented elsewhere in Deng & Carter (1999a) and Deng & Carter (1999b).

4.1 VERTICAL UPLIFT CAPACITY - UNDRAINED

The vertical uplift capacity can be estimated by a modified form of the equations governing the reversed bearing capacity problem. The ultimate uplift capacity can be expressed as the ultimate value of the average uplift traction, p_u, applied at the top over the caisson area, $A = \pi d^2/4$. In this case p_u will be the sum of two terms, one representing the effective overburden pressure at the level of the caisson tip, and the other depending on the undrained shearing resistance of the soil, i.e.,

$$p_u = 1.2 N_p d_c s_{u(tip)} \tag{8}$$

where L is the embedded length of the caisson and d is its diameter. $s_{u(tip)}$ is the undrained shear strength of the soil at the depth of tip of the caisson.

The uplift capacity factor, N_p, which has taken account of the effects of the bottom resistance and the soil-wall friction, may be expressed as: (Deng & Carter, 1999a)

$$N_p = 7.9\left(\frac{L}{d}\right)^{-0.18}. \tag{9}$$

where L/d is the aspect ratio. In equation (8), d_c is the usual embedment factor for undrained bearing capacity, given by $1+0.4(L/d)$. The theoretical relationship between N_p and L/d (or s_{utip}/kd, where k is the undrained strength gradient with the depth) has been plotted in Fig. 6, together with the lower bound solutions for bearing capacity factor N_c suggested by Houlsby and Wroth (1983). There is good agreement between the two analytical estimates of N_p and N_c.

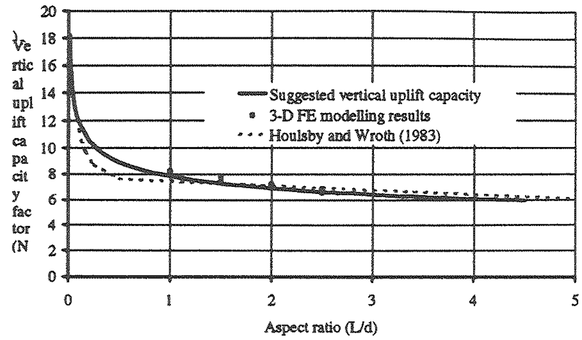

Figure 6 Vertical uplift capacity ratio

Clearly, the ultimate vertical uplift load can be expressed simply as:

$$V_u = p_u A \tag{10}$$

4.2 LATERAL CAPACITY – UNDRAINED

Previous investigators (e.g., Randolph and Houlsby, 1984; Murff and Hamilton, 1993) have carried out studies of the ultimate behaviour of piles in clays under lateral loading. These studies were of pile segments or complete piles subjected to lateral loading applied at the top of the pile, and hence the solutions do not take into account the influence of the relative depth to the loading point, D/L (where D is the depth of the load application point from the soil surface).

Studies by Deng and Carter (1999b) have shown that a good approximation for the ultimate horizontal load that may be applied to a suction caisson, H_u, can be expressed approximately as:

$$H_u = N_h A s_{u(2/3L)} \tag{11}$$

where, $s_{u(2/3L)}$ denotes the undrained shear strength at a depth equivalent to 2/3 of the caisson length. It was also found that N_h could be described by an expression of the form

$$N_h = \frac{\alpha}{\sqrt[4]{\left(\frac{\alpha}{6.3} - \frac{D}{L}\right)^4 + \left(\beta \frac{D}{L}\right)^4}} \tag{12}$$

The dimensionless parameters α and β are related to the aspect ratio, L/d, and they can be expressed as follows

$$\alpha = 7.02\left(\frac{L}{d}\right)^{-0.3785} \tag{13}$$

and

$$\beta = 1.58 e^{-0.775\left(\frac{L}{d}\right)} \tag{14}$$

The above solutions are applicable to the case of soil strength increasing linearly with depth (e.g., normally consolidated soils). The method may also be applied to cases where foundations are embedded in overconsolidated soils. In the latter case, the soil strength $s_{u(2/3L)}$ appearing in expression (11) should be replaced by the strength of the soil at the depth at which only horizontal displacement occurs when horizontal load is applied.

The above solutions are plotted in Fig. 7 for cases where the aspect ratio, L/d, is 1.0, 1.5, 2.0 and 2.5.

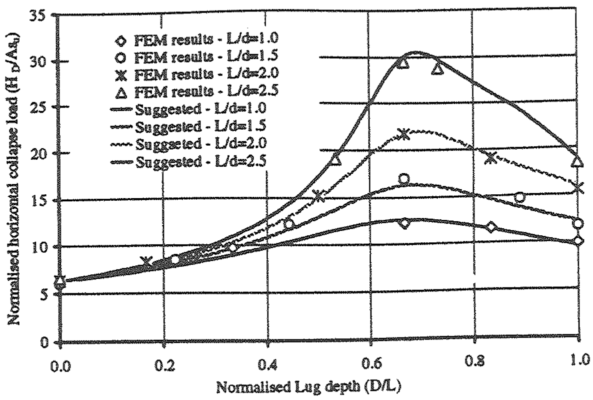

Figure 7 Lateral capacity factor for undrained conditions

4.3 INCLINED UPLIFT CAPACITY - UNDRAINED

Deng and Carter (1999b) found by fitting a curve to the results of finite element analyses that the ultimate inclined uplift load can be expressed to sufficient accuracy as:

$$\frac{V}{V_u}+\left(\sqrt{1-\left(\frac{H}{H_u}\right)^2}-1\right)^2-1=0 \tag{15}$$

in which V_u is the ultimate value for purely vertical load, given by equation (10), and H_u is the ultimate lateral resistance for purely horizontal load applied at a lug depth, D. The value of H_u is given by equation (11). V and H are the vertical and horizontal components of the ultimate inclined load applied to the axis of the caisson, and obviously,

$$\frac{V}{H}=\tan\theta_a \tag{16}$$

where, θ_a is the angle of load inclination (with reference to the horizontal plane). The solution given by equation (15) is plotted in Fig. 8.

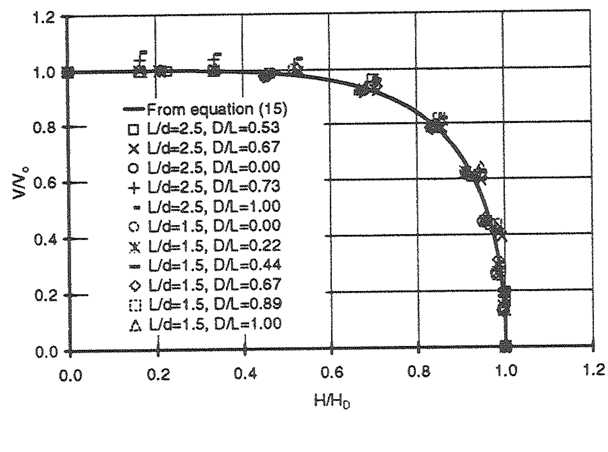

Figure 8 Inclined uplift capacity (undrained)

In practice, load is usually applied to anchor caissons by the attachment of a chain to a lug fixed to the sidewall of the caisson. A load applied directly to the caisson wall at a depth D is statically equivalent to the case of the same load applied at the centre line, at a depth of $D+(d/2)\tan\theta_a$. Therefore, the failure load for the more usual case of sidewall attachment can be obtained from equation (15), after making appropriate allowance for the location of the fixing point.

4.4 VERTICAL UPLIFT CAPACITY – FULLY DRAINED

In cases where the subsoil is fully drained, the pullout capacity of the caisson is equal to the total friction developed between the soil and the caisson wall. At any depth h, the friction between the soil and the caisson wall, f_s, can be expressed as:

$$f_s = \sigma_h ' \tan \delta \tag{17}$$

where, σ'_h is the effective horizontal stress acting on the soil element at that depth, δ is the interface friction angle. The interface friction angle may be influenced by the soil type (internal friction angle and OCR) and the aspect ratio. Therefore it has been suggested (Deng and Carter, 1999a) that the drained vertical uplift capacity of a cylindrical caisson can be expressed as:

$$p_{u(net)} = 9.1\left(\frac{L}{d}\right)^{0.5372}(1-\sin\phi')(OCR)^{\sin\phi'}\tan\phi'\sigma'_{v(bottom)} \tag{18}$$

where, γ' is the effective unit weight of soil, ϕ' is the effective friction angle, OCR is the over-consolidation ratio, and $\sigma'_{v(bottom)}$ is the vertical effective stress at the location of bottom of the caisson.

Equation (18) has considered the friction along both the external and the internal caisson wall. The effects of the the relationship between the interface friction angle, δ, and the soil friction angle, ϕ' and the correlation of horizontal effective stress in the soil to the original in situ effect stress have also been reflected on.

4.5 VERTICAL UPLIFT CAPACITY – PARTIALLY DRAINED

The pullout capacity of suction caissons subjected to uplift loading under partially drained conditions can be estimated by the following formula (Deng & Carter, 1999a):

$$p_{u(net)} = N_f\, p_{u(drained)} + N_b s_{u(tip)} \tag{19}$$

where

$$N_b = [0.13 - 0.446\ln(T_k)]1.6^{\frac{L}{d}} \tag{20}$$

$$N_f = 0.632 - 0.091\ln\left(\frac{L}{d}\right) \tag{21}$$

In expression (19), the first component is the friction resistance developed along the caisson wall and the second component is the resistance developed at the bottom of the caisson. Therefore N_f is called the friction factor and N_b is the bottom breakout resistance factor. $p_{u(drained)}$ is the drained pullout capacity and $s_{u(tip)}$ is the initial undrained strength at the tip of the caisson. T_k is a non-dimensional load rate parameter that can be defined as:

$$T_k = \frac{C_v}{vd} \tag{22}$$

where, C_v is the coefficient of consolidation of the soil and v is the load rate (steady velocity) at which the caisson is pulled from the ground.

It was found that when $T_k > 0.6$, the pullout behaviour is effectively fully drained, so $N_b = 0$. When $T_k < 0.002$, the pullout behaviour can be considered as effectively undrained. The upper limit of N_b is determined by the undrained condition. Hence, equations (19) to (21) are applicable whenever $0.002 < T_k < 0.6$.

4.6 LATERAL CAPACITY – FULLY DRAINED

Broms (1964) developed a simple but reliable method for the estimation of lateral capacity for suction caissons in sands. Broms' expression for the ultimate lateral capacity of a caisson embedded in saturated sand can be expressed as:

$$Q_u = \frac{\gamma' d L^3 \tan^2(45^\circ + \frac{\phi}{2})}{2(D+L)} \qquad (23)$$

where Q_u is the ultimate lateral load, ϕ is the angle of internal friction of the sand, γ' is the submerged unit weight of the sand, L is the embedded depth of the caisson, d is its diameter, and D is the distance from the point of application of the lateral force to the sand surface.

5 EVALUATION OF THEORETICAL SOLUTIONS

The theoretical solution for inclined uplift capacity can be verified by the solutions obtained by an upper bound technique using the theory of soil plasticity (Randolph et al, 1998). The theoretical solutions discussed above can also be examined in the light of available experiment data. The published experimental data include laboratory model-scale and centrifuge results, as well as some field test results. The test results include 76 individual tests from 12 independent studies, which are detailed in Deng & Carter (1999c).

5.1 EVALUATION BY UPPER BOUND TECHNIQUES

A three-dimensional upper bound technique was developed by Randolph et al (1998), based on the flow mechanism proposed by Murff & Hamilton (1993). This technique is focused on the capacity of suction caissons subjected to quasi-horizontal loading form a mooring chain. This method was extended to include quasi-vertical loading of suction caissons for TLP anchorages by Deng & Carter (1999b) by adopting the uplift capacity factor, as shown in expression (9), when considering the energy dissipation at the caisson tip due to soil movement in the vertical direction. Detailed comparison between the theoretical solution for inclined uplift capacity and the upper bound techniques can be found in Deng & Carter (1999b).

It was found that the differences between the theoretical solutions and the upper bound solutions for cases where the load is applied vertically along the sidewall are less than 5% for most aspect ratios. While for cases where purely horizontal load is applied the differences are 1% to 20%. A detailed comparison for inclined load cases is given in Fig.9.

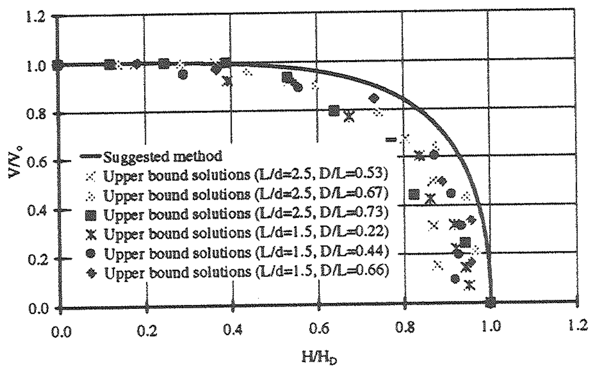

Figure 9 Comparison of theoretical solutions and upper bound solutions

As shown in Fig.9, the upper bound solutions for the ultimate loads are generally lower than the theoretical solutions. The reasons for this are likely due to:

- The theoretical solutions were derived from the finite element predictions. Generally finite element results tend to be over-estimates and it was also assumed in the finite element analysis that the interface between the soil and the caisson is perfectly bonded.
- The upper bound method is not entirely rigorous and has some limitations. One of the limitations is that this upper bound solution is not kinematically complete, since it ignores interaction between the top and the bottom of the 'flow' region (below the conical wedge) and the adjacent soil.

5.2 EVALUATION BY TEST DATA

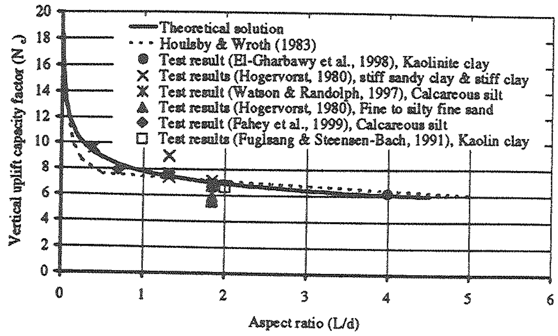

Figure 10 Comparison of theoretical solution and test data for undrained vertical uplift capacity (after Deng & Carter, 1999c)

Detailed evaluation can be found in Deng & Carter (1999c). Figures 10, 11, 12 and 13 show comparisons of the test results and the theoretical solutions for vertical uplift capacity (undrained), lateral capacity (undrained), inclined uplift capacity (undrained) and vertical uplift capacity (drained) respectively. From the comparisons, the following conclusions can be drawn.

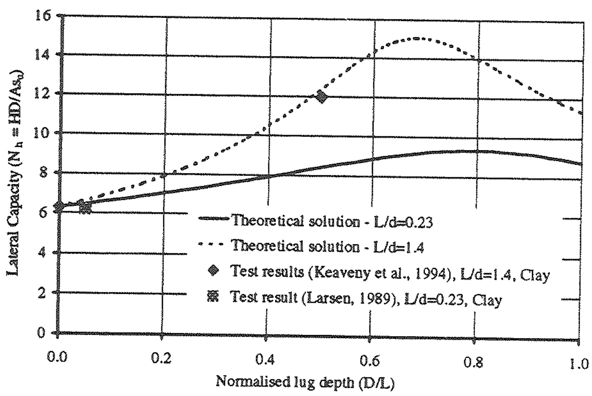

Figure 11 Comparison of theoretical solution and test data for undrained lateral capacity (after Deng & Carter, 1999c)

- The suggested theoretical method for estimating the undrained vertical capacity of suction caissons seems to apply reasonably well to a wide range of soils, including stiff clay, kaolinitic clay, sandy clay, calcareous silt, fine sand, and silty fine sand.
- For estimation of the lateral capacity of suction caissons, the suggested theoretical solution is reasonable for cases where the subsoil is clay or has significant cohesion and deforms under undrained conditions.
- The suggested theoretical method for the estimation of the inclined uplift capacity of suction caissons under undrained conditions appears to be reasonably accurate for design purposes.

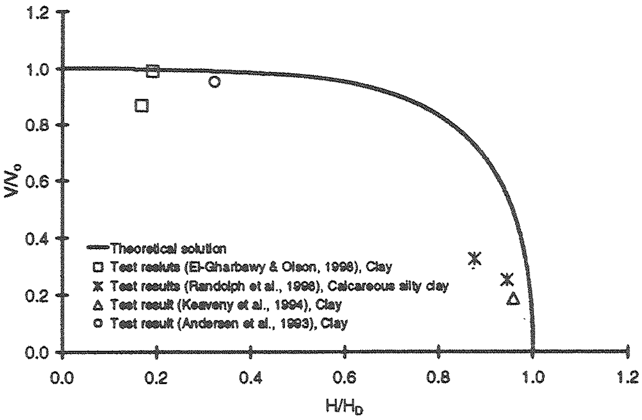

Figure 12 Comparison of theoretical solution and test data for undrained inclined uplift capacity (after Deng & Carter, 1999c)

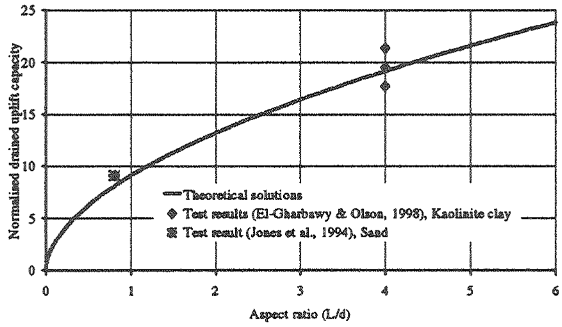

Figure 13 Comparison of theoretical solution and test data for drained vertical uplift capacity (after Deng & Carter, 1999c)

Six tests included measurement of the drained lateral capacity. The comparison shows that the differences between the prediction of Broms' formula and the measured lateral capacity are between 0.0 ~ 21.7%, which indicates that Broms' formula appears to be reliable for cases where the subsoil is sand or noncohesive soil. Twenty-three individual test results were recorded for vertical uplift capacity (partially drained). The differences between test results and estimation by expression (19) are 2.3 ~ 23.3% for aspect ratio L/d = 0.75, 4.5 ~ 10.8% for L/d = 1.0, 4.3 ~ 24.7% for L/d = 1.5, 10.7 ~ 47.7% for L/d = 2.0 and 114.6% for L/d = 4.0. It may be seen that for aspect ratios less than 2, the test results agree reasonably well with the theoretical solutions. It is also noted that the theoretical solutions generally overestimate the measured capacities. However, for those tests with an aspect ratio larger than 2, the theoretical solutions generally significantly overestimate the measured responses. Use of the suggested formula (19) in design together with an appropriate factor of safety is therefore recommended for aspect ratio less than 2 only.

6 CONCLUSIONS

In this paper, theoretical solutions for estimation of the uplift capacity of suction caissons, for cases where the subsoil is drained, partially drained or undrained and the load is applied horizontally, vertically or inclined, have been presented. These methods have been verified by the results from a relatively large number of laboratory and field tests and solutions obtained from an upper bound technique using the theory of soil plasticity. The following conclusions were reached.

- Different failure mechanisms will be developed when the point of load application, the angle of load application or the subsoil's drainage condition is different.
- The suggested theoretical method for estimating the undrained vertical capacity of suction caissons, equations (8) and (9), seems to apply reasonably well to a wide range of soils, including stiff clay, kaolinitic clay, sandy clay, calcareous silt, fine sand, and silty fine sand.

- For estimation of the lateral capacity of suction caissons, the use of equation (13) and (14) is suggested for cases where the subsoil is clay or has significant cohesion and deforms under undrained conditions. The use of Broms' formula, equation (25), is suggested for cases where the subsoil is sand or is non-cohesive and deforms in a fully drained manner.
- The suggested theoretical method for the estimation of the inclined uplift capacity of suction caissons under undrained conditions, equations (17) and (18), appears to be reasonably accurate for design purposes.
- The suggested formula for the estimation of drained vertical uplift capacity, equation (20), can be used for a wide range of aspect ratios and for various kinds of soils.
- The suggested formulae for estimating the vertical uplift capacity for partially drained conditions, equations (21) to (23), appear to be reasonably accurate only for those cases where the aspect ratio of the caisson is less than 2.
- It should also be noted that the theoretical methods for the estimation of uplift capacity of suction caissons presented in this paper consider only two-sided (anti-symmetric) failure mechanisms.

7 ACKNOWLEDGEMENTS

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INVESTIGATION OF SLOPE MOVEMENTS IN A COASTAL SAND DUNE IN SOUTH-EAST QUEENSLAND

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SUMMARY

This paper presents the results of monitoring and investigation of slope movements which occurred in a coastal sand dune in South East Queensland, at a time of elevated groundwater levels. The magnitude and pattern of displacement suggest that a Factor of Safety of very close to 1 developed as a result of the rise in groundwater levels. Subsurface investigations identified the presence of a layer of former swamp deposits (sandy clay and clayey sand) beneath the dune, which has been interpreted as the layer in which shear displacement was concentrated. Finite element and limit equilibrium analyses using a Mohr-Coulomb constitutive model yield consistent results, and indicate that an angle of friction of 11° is required for a Factor of Safety of 1 for a failure mechanism which passes through the former swamp deposits. Such a low friction angle is inconsistent with limited laboratory testing on the material. Further work is being undertaken to better understand the mechanisms which led to the development of large scale movements, in order to better predict the potential future development of such movements.

1 INTRODUCTION

Results of monitoring and analyses carried out in relation to slope movements which occurred in a coastal sand dune in South East Queensland are presented in this paper. The sand dune is a relatively young dune, which extends over older swamp deposits. The crest of the dune is approximately 80 m above the toe, and the slope of the dune is 23° on average with steeper sections of up to 40° in places. A contour plan for the dune is illustrated in Figure 1, and a cross-section through the dune is illustrated in Figure 2. In late 1998, a downward displacement of 3.3 m developed near the crest over a period of approximately 17 days. The downward displacement at the crest was accompanied by the development of a 2.7 m scarp mid-way down the dune, and an upwards movement of 4.3 m at the toe of the dune, indicating horizontal movement of a lower wedge of sand and downward movement of an upper wedge.

2 DESCRIPTION OF DUNE MOVEMENTS

The slope movements developed at a time when dredging associated with a sand mining operation was being carried out at a distance of 400m behind the toe of the dune, in a dredge pond with a water level at 35m above the toe. The location and level of the dredge pond were within the normal operational guidelines for the mine.

For several days preceding the first observed formation of cracks in the slope, loud rumbling noises were heard emanating from the dune. Such behaviour has been previously observed to precede large slope movements in similar terrain in the area. Four days after the first rumbling sounds were heard, a crack was observed in the ground along the crest of the north-west dune, and crude measurements of slope movement were instituted at that time (measurement using a tape measure between two stakes). More detailed monitoring using surveying equipment was established shortly thereafter, and measurements of the vertical displacement at the crest were made every fifteen minutes initially.

The cumulative displacement measured at the crest is illustrated in Figure 3. More than 1 m of vertical movement developed at the crest over the first 48 hours of movement. A total movement of approximately 3.3 m developed over a period of approximately 17 days, after which time the movement slowed to less than 0.5 mm/hour. The slope has remained stable since that time. Detailed information regarding the rates of displacement in the first seven days of movement are illustrated in Figure 4, with information on tidal water levels close to the toe of the dune. It can be seen that the rate of movement is closely tied to tidal fluctuations, and that peaks rates of movement occur approximately 1.5 hours after tide peaks. The mechanism of tidal influence is yet to be considered in detail.

In addition to the movement at the crest, a mid-slope scarp developed (which was first observed the day after the first observation of the upper crack). At the completion of monitoring, a relative displacement of 2.7m had developed at

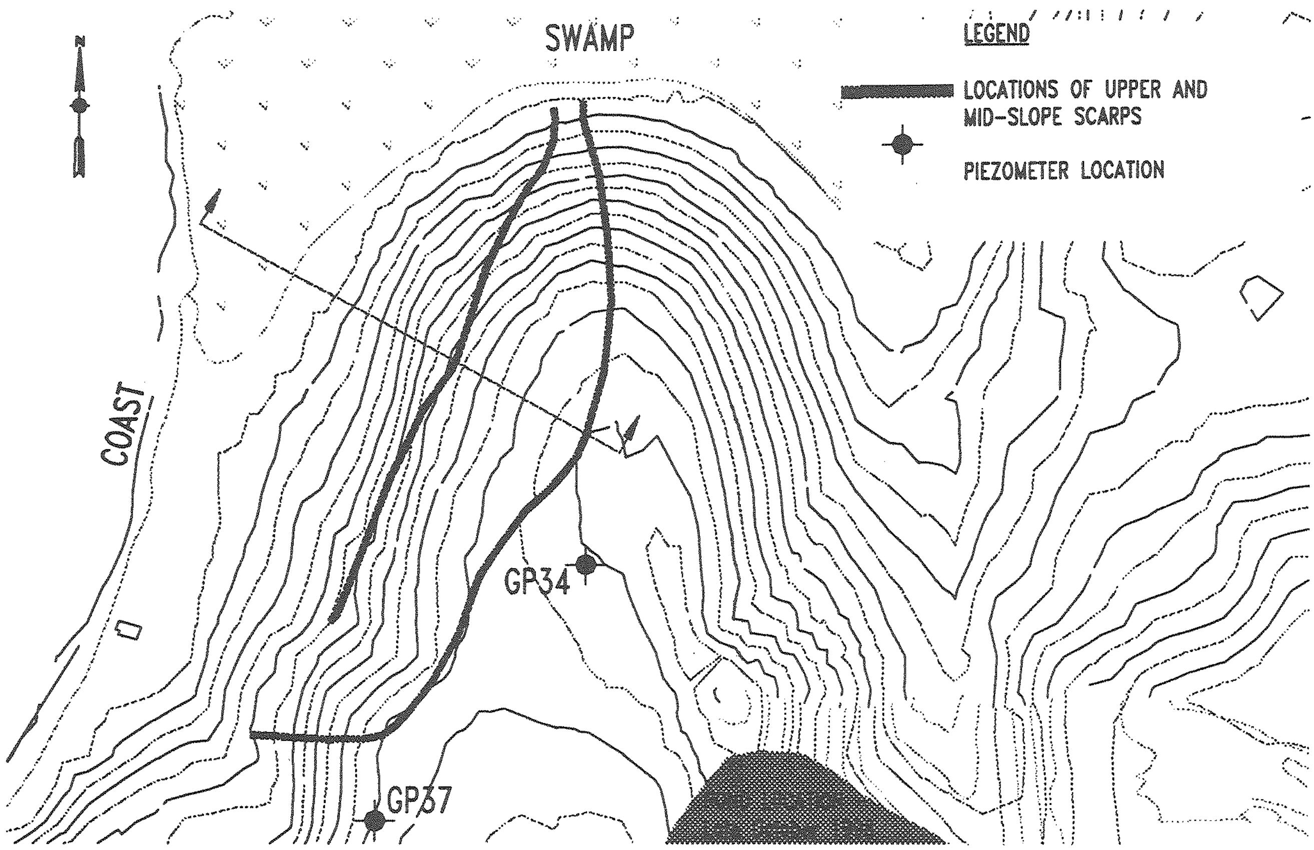

Figure 1 Plan of sand dune

the mid-slope scarp. The location of the upper and mid-slope scarps and the recorded movement for these features are illustrated in Figures 1 and 2 (the cross-section illustrated in Figure 2 is for the section on which the largest slope movements were measured). Apart from some leaning trees near the toe, there was no readily apparent evidence of mounding in this area. However, significant displacements developed in this area, as indicated by surveying carried out prior to and following the movements (refer to Figure 2).

3 GROUNDWATER CONDITIONS

The water table within the dune rose over a period of approximately 8 months preceding the slope movements, in response to seepage from the mine dredge pond. The pond moved towards the dune from the east, and on 30 October 1998 was located as shown in Figure 1.

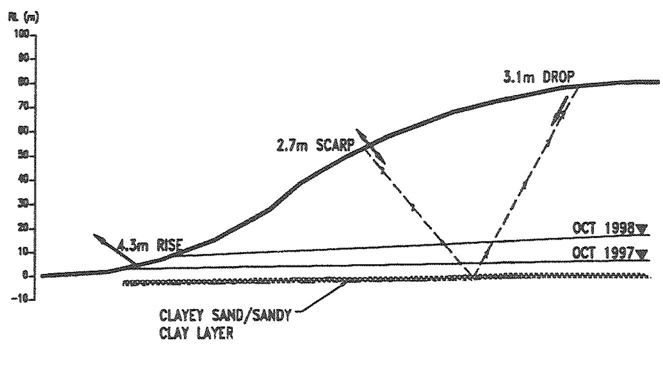

Figure 2 Cross-section through dune indicating measured displacements

The results of water level monitoring at two piezometers in the vicinity of the dune are illustrated in Figure 5. The locations of these piezometers are shown in Figure 1. It can be seen that the water table at the rear of the dune was at an elevation of approximately 7 m AHD in the period prior to direct influence from the dredge pond, and that peak levels in excess of 18 m AHD were recorded on 6 November 1998. Water levels dropped relatively rapidly following 6 November, as the dredge pond was moved away from the area of slope movements. The results also indicate that similar groundwater levels were present along the length of dune where movements were observed.

4 SUBSURFACE INVESTIGATIONS AND LABORATORY TESTING

Drilling and cone penetrometer testing was carried out at a number of locations along the toe of the dune, and at three locations near the crest of the dune, to assess the nature of materials beneath the dune, and to assess the extent of the suspected swamp deposits beneath the dune. As indicated in Figure 1, swamp deposits were present around the base of the dune to the north and west, and the shape of the dune indicated that it had been blown out over pre-existing swamp deposits.

Drilling and cone testing results identified the presence of a layer of variably sandy clay/clayey sand beneath the dune, with a variable percentage of organic material (refer to Figure 2). Some samples of this material were essentially peat. The layer extended along the majority of the toe of the dune, and was present in the boreholes drilled from the crest of the dune. Samples of clayey material from beneath the crest of the dune were very stiff to hard (as would be expected for clay normally consolidated beneath an 80 m high dune), and samples from beneath the toe were generally firm. It appears that the layer of sandy clay/clayey sand was deposited in a lake, over a previously deposited, relatively deep layer of weathered sand. The investigation did not identify the presence of any other layers on which movements could have potentially developed, and it has been assumed that shear displacement was concentrated in this layer.

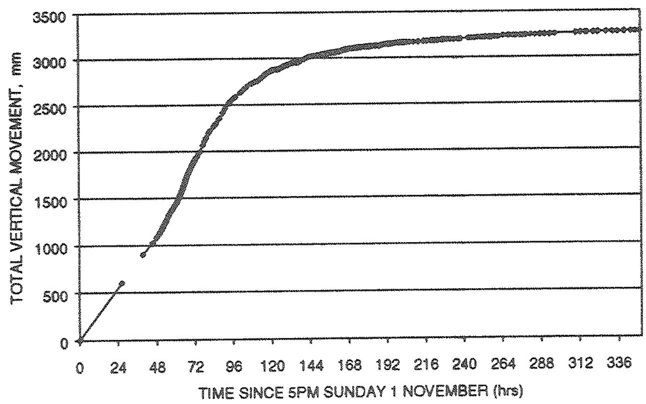

Figure 3 Cumulative displacement measured at crest

Shear box testing on a sample of clayey sand from this layer yielded an angle of friction of 27°. Additional laboratory testing on further samples recently obtained is currently underway.

5 BACK-ANALYSIS OF MOVEMENTS

5.1 GENERAL

Back-analyses to assess the conditions which led to the development of the slope movements described above have been undertaken using two approaches:

- non-linear stress-strain analysis using the finite element technique
- Limit Equilibrium methods

Finite element analyses have the advantage that they do not make a priori assumptions regarding stress distributions or locations of limiting soil stresses, and stresses and displacements are therefore determined by considerations of equilibrium, strain compatibility, and soil strength. Limit equilibrium methods, which are widely available and are relatively easy to use were also used for back-analysis, since such methods would ideally be used in any analysis of future potential problems of a similar nature.

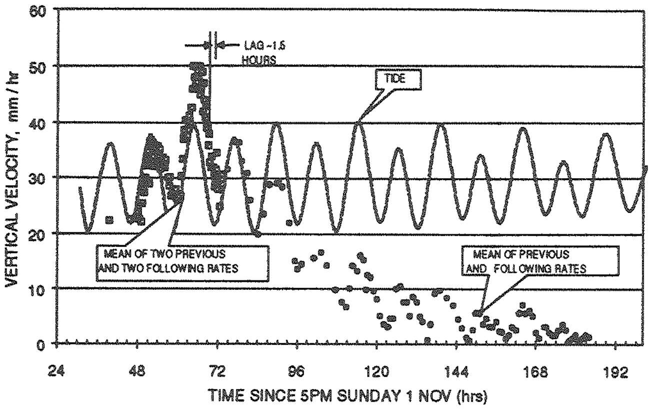

Figure 4 Rates of displacement at crest

Prior to commencement of the back-analyses, it was necessary to consider the choice between drained and undrained analyses. Loading of the slope took place over a period of approximately 8 months as the water table in the dune rose and increased the weight of sand above the potential sliding layer. In addition to the increase in total stress due to increased bulk density (possibly with consequent pore pressure increases depending on the rate of loading), pore pressure increases develop as a result of equilibration with the raised water table. In the extreme case of rapid increase in water level, the pore pressure increase in the lower permeability potential sliding layer would initially depend on total stress changes and the A and B pore pressure parameters. For typical values of A and B, the pore pressure changes which would accompany such an undrained loading would be less than the pore pressure changes which would develop as a result of equilibration with the raised position of the water table. Thus, in this case, drained conditions should be more critical than undrained conditions. Analyses for this study have therefore been carried out as drained analyses, with pore pressures that correspond directly to the rising level of the water table in the sand dune. It is possible that the dune movements developed prior to the full equilibration of pore pressures, in which case the actual soil strengths would be somewhat less than indicated by the back analysis.

5.2 FINITE ELEMENT MODELLING

In the finite element modelling carried out for this study, soil behaviour was described using an elastic, perfectly plastic constitutive law, with a limiting shear strength condition defined in terms of the Mohr-Coulomb failure criterion.

The finite element mesh used for analyses is illustrated in Figure 6. The lateral boundaries of the model were specified as zero horizontal displacement boundaries, and the basal boundary was specified as a zero displacement boundary. The lateral boundaries were chosen to lie outside the limits of observed slope movements. The formulation of the finite element model is for plane strain conditions, and therefore represents a slope which is infinitely long and of uniform cross section in the direction perpendicular to the section which was analysed. The cross-sectional geometry adopted for the model is that of the cross section on which maximal slope movements occurred. The implications of the assumption of plane strain conditions are discussed further in the following.

Properties adopted for the various slope materials are summarised in Table 1. Properties for the dune sand were based on previous testing. Variable strengths were adopted for the material in the clayey sand/sandy clay layer, in order to determine the angle of friction for which large slope movements were initiated in the model.

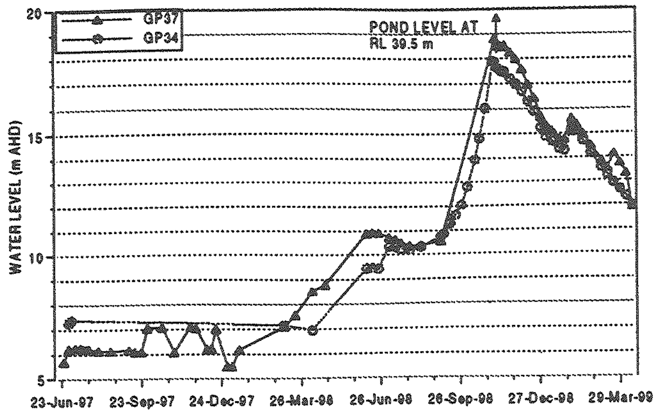

Figure 5 Measured groundwater levels at crest of dune

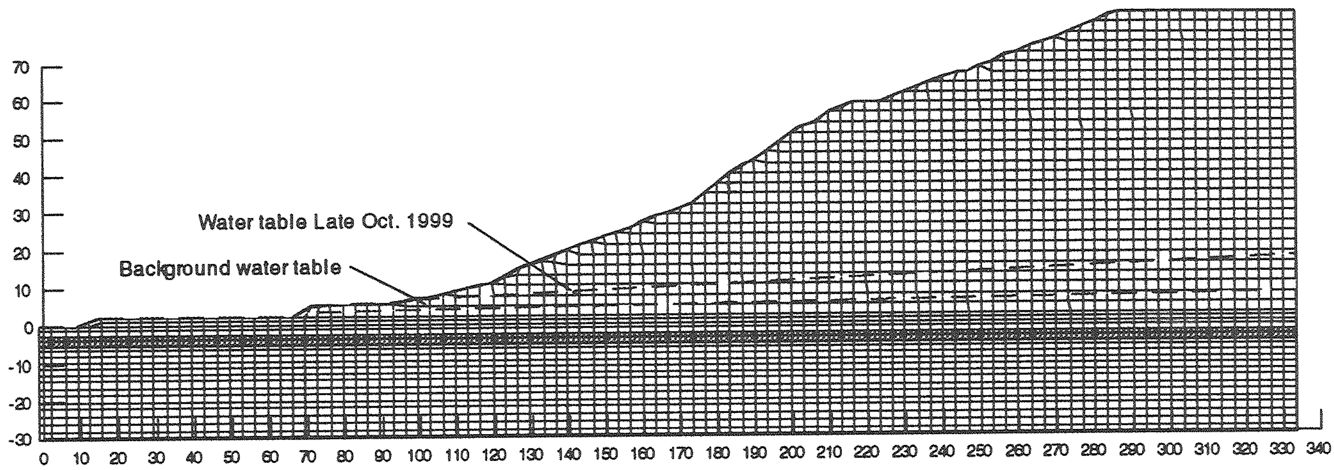

Figure 6 Finite Element Mesh

Material	Strength Properties	Density
Dune sand	$\phi' = 32°$	γ=19 kN/m3 below water table γ=16 kN/m3 above water table
Sliding layer	ϕ' = variable	γ=18 kN/m3
Underlying sand	$\phi' = 35°$	γ=19 kN/m3

Table 1 Material properties adopted in the Finite Element Model

Analyses were carried out for two water table elevations, as illustrated in Figure 1. The two water table conditions are based on the water table profiles for steady state conditions prior to the water table rise, and for the conditions at the end of October following a water table rise over the previous 8 months. At this stage, no attempt has been made to consider the effects of tidal fluctuations.

The stress history of the slope was imitated by commencing modelling with an approximately horizontal ground surface, and then sequentially adding layers of material to the model to simulate accretion of the dune over a period of time. Stress conditions in the slope were calculated in this manner for the case of steady state water table conditions, and changes from this condition were then calculated by increasing the water table elevation, and increasing the bulk density of the material in the zone of water table rise.

5.3 LIMIT EQUILIBRIUM ANALYSES

Limit Equilibrium analyses were carried out using the Generalised Wedge Method (Giam, 1989). The Generalised Wedge Method is a true upper bound solution, and thus the calculated Factor of Safety is strictly greater than or equal to the "true" Factor of Safety. Conventional Limit Equilibrium analyses are neither true upper bound or true lower bound analyses. The magnitude and pattern of slope movements, and their sensitivity to minor influences such as tidal fluctuations, indicates that soil strengths were very close to fully mobilised along a kinematically admissible surface. This suggests that an effective Factor of Safety of very close to 1 should be adopted for back-analyses with Limit Equilibrium methods.

As discussed further in the following, analyses with the Generalised Wedge Method were based on wedge geometries which were consistent with the observed displacements, and which were consistent with the pattern of displacement indicated by the finite element analyses. Analyses were carried our using parameters indicated in Table 1.

6 BACK ANALYSIS RESULTS

Horizontal displacement, calculated at the toe of the dune using the finite element method, is plotted in Figure 7 as a function of the angle of friction for the potential sliding layer. The displacements which are illustrated in this figure are for the load increment corresponding to the increase in water table.

Limited plasticity developed at the toe for a friction angle of 15°, with increasing displacement at lower friction angles. A converged solution was obtained for an angle of friction of 11° (with displacements of the order of 140 mm), whereas the analysis with an angle of friction of 10° did not converge. The pattern of displacement for the analysis with an angle of friction of 11° is illustrated in Figure 8. The pattern of displacement is roughly consistent with the observed pattern of displacement. However observations of the displacement at the upper scarp suggest displacement developed on a plane at a much steeper angle than that indicated by the finite element analysis. Similarly, the finite element analysis indicates the likely formation of a mid-slope scarp at a lower elevation and on a steeper plane than that observed.

Analysis using the Generalised Wedge Method was carried out using the wedge geometries indicated in Figure 8, and a friction angle of 11°. The calculated Factor of Safety for this case was 0.98.

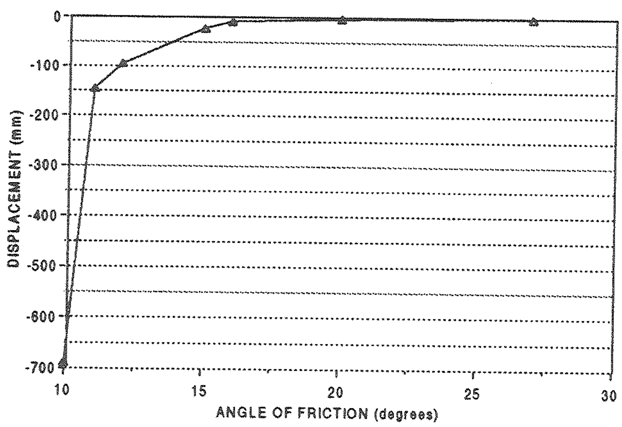

Figure 7 Predicted displacement at toe of dune

7 DISCUSSION OF BACK ANALYSIS RESULTS

The back analysis results indicate that an angle of friction of 11° would be necessary for large slope movements to develop, such as were observed. An angle of friction this low is inconsistent with the results of laboratory testing, and with values which are conventionally adopted for similar swamp deposits in South-East Queensland.

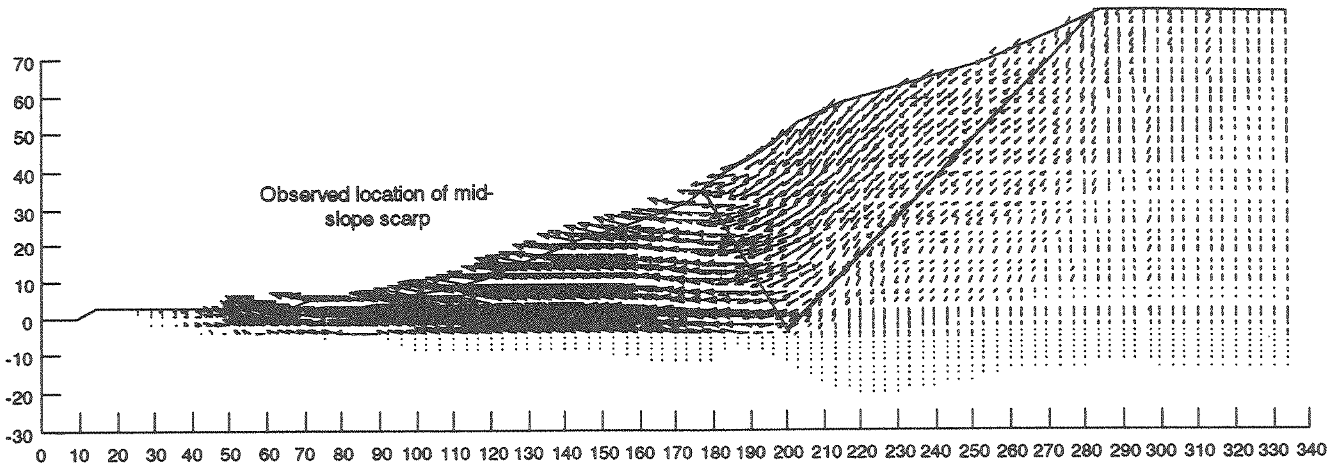

Figure 8 Pattern of displacement predicted by finite element analysis

One possible explanation is that large movements have previously developed at this site, and have caused residual strength to develop along the pre-existing slip surface. It should be noted, however, that laboratory testing carried out to date has not indicated that residual shear strength is substantially less than peak shear strength for the sandy clay/clayey sand layer beneath the dune.

The mechanisms for the development of a pre-existing slip surface are not evident. For a previous slip to have caused residual strength to develop on a discrete failure surface, it would be necessary for it to have developed after consolidation of the pre-existing swamp deposits beneath the dune (since failure on a discrete plane would not generally develop in a soft clay). A possible trigger for previous movements could be an earthquake.

It is worth noting that even for friction angles as high as 27°, the finite element analyses indicated the full development of plasticity in the clayey sand/sandy clayey layer, in almost the entire length from beneath the crest to the toe. Full development of plasticity was predicted in limited areas for the conditions preceding the water table rise, and the area of plasticity extended towards the toe as a consequence of the water table rise.

Although the full development of plasticity was predicted locally by the finite element analyses, large-scale displacements did not develop in the model since the local conjugate failure directions were not aligned horizontally. Relationships between local shearing on conjugate planes and overall shear displacements have been discussed extensively in the literature (for example, de Josselin de Jong (1971, 1988), Airey and Wood (1987), and Wroth (1987)). It has been postulated that, for example, displacements in a shear box test develop on vertical surfaces, and that overall horizontal displacements are made up of a combination of shearing on vertical surfaces and rigid body rotations (de Josselin de Jong, 1971). The references cited above indicate that shearing which develops as a result of such a mechanism provides less resistance than would be indicated by the angle of friction and the stress normal to the overall direction of shear displacement.

It appears possible that displacement at the toe may have been initiated by such a mechanism, leading to progressive development of movement through the remainder of the layer already at limiting conditions. The observed rumbling from the dune for a period of several days prior to the development of movement at the crest indicates a progressive development of movement commencing at the toe. Further analysis using alternative constitutive models (for example the constitutive models developed by de Josselin de Jong (1971, 1988)) may provide additional insight into the conditions which led to movement.

8 CONCLUSIONS

Large displacements which have developed in a coastal sand dune overlying former swamp deposits have been analysed using finite element and limit equilibrium techniques, in an attempt to understand the factors which led to the development of movement. Understanding the mechanisms which led to the movement is important, since future mine paths will pass within a similar distance of coastal dunes which are similarly located over former swamps.

The analyses yielded consistent results which indicate that for a Factor of Safety of 1 to develop as a result of an observed water table rise, an angle of friction of 11° is required for a layer of organic sandy clay and clayey sand which was encountered beneath the dune. The failure mechanism predicted by the analyses is similar to that observed in the field.

Laboratory testing on a sample of clayey sand collected from the likely sliding layer beneath the dune has indicated an angle of friction of 27°, which is consistent with values typically adopted for swamp deposits in the area. Mechanisms for the development of a plane of weakness with a friction angle of 11° are speculative only, but might include the prior development of plane with residual shear strength due to large scale movements caused by an earthquake. Geomorphological evidence of previous large scale movements would be difficult to discern, and evidence for the presence of a plane of weakness with very low angle of friction may be difficult to obtain from conventional site investigation techniques.

It is possible that analyses based on a Mohr-Coulomb failure criteria has led to under-prediction of the friction angle for the former swamp deposits beneath the dune, since such a model requires one of the conjugate failure directions to align horizontally for failure to develop. However, it is not clear that this requirement would necessarily hold in a system where soil plasticity is fully developed on inclined planes, but movement is essentially constrained to the horizontal plane by the presence of higher strength material above and below. If possible, further analyses will be carried out with alternative constitutive models, in order to assess the possibility that large displacements developed without the presence of a plane of very low strength material.

Further analyses will also be carried out in relation to similar large scale movements, which have previously developed at the mine in the batters of the dredge pond. Analysis of similar movements will hopefully assist in the assessment of whether such movements require the presence of very low shear strength materials.

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PUTTING THE GEO INTO GEOTECHNICAL - THE ROLE OF THE ENGINEERING GEOLOGIST DURING CONSTRUCTION OF THE EASTERN DISTRIBUTOR TUNNEL, SYDNEY, AUSTRALIA

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SUMMARY

The 1.5km long Eastern Distributor Tunnel, to the east of Sydney's CBD, is Australia's first piggyback tunnel. The tunnel is excavated through a highly urbanised area with limited rock outcrop, hence the geotechnical model formulated for design purposes was heavily reliant on borehole information. During construction, detailed engineering geological mapping served to 'proof check' the assumptions and interpretation made in the design model.

This paper briefly summarises the design geotechnical model, the role of the Engineering Geologist during construction and the Geologist's position in the overall context of the construction team. The post construction model, formed on the basis of in-tunnel mapping, is then presented.

It is concluded that the mapping undertaken during construction largely confirmed the preconstruction model. Where encountered tunnelling conditions were different to those anticipated in the design model, the results of engineering geological mapping allowed appropriate support redesign in 'real time'.

The paper therefore provides a working example of the concept of 'putting the "geo" into "geotechnical'.

1 INTRODUCTION

The 1.5km long Eastern Distributor Tunnel, located to the immediate East of the CBD of Sydney (Figure 1), will carry three lanes of Northbound traffic over three lanes of Southbound traffic in Australia's first piggyback tunnel.

The geotechnical model for design purposes was based on field investigations comprising a sum total of 103 cored boreholes, supplemented by geotechnical mapping in old quarries and along the Cahill Expressway, to the north of the tunnel. Hence the preconstruction or design geotechnical model was heavily reliant on interpretation between boreholes. Engineering geological mapping undertaken during the construction phase of the project was used to validate the preconstruction model.

This paper outlines the design geotechnical model, tunnel construction sequence and the role of the Engineering Geologist during construction. Elements of the post construction geotechnical model are then compared to the design model. It is concluded that mapping and database upkeep during excavation allowed for effective and quick support redesign where needed, served to largely confirm the design geotechnical model and has provided a permanent record of excavation conditions for the future.

2 PRECONSTRUCTION (DESIGN) MODEL

2.1 LITHOLOGY

The tunnel excavation was expected to occur entirely within subhorizontally bedded Hawkesbury Sandstone which forms the basement rock for the majority of the CBD. However, the upper part of the main south portal at South Dowling St and the ramp tunnels to Anzac Parade and Moore Park Rd were anticipated to be excavated through weathered Mittagong Formation. The boundary between the Hawkesbury Sandstone and the Mittagong Formation was known to be gradational and it was considered that the base of the Mittagong Formation formed a shallow basin structure near Taylor Square (Figure 1).

Three main sedimentary facies are apparent within the Hawkesbury Sandstone as outlined below.

1 <u>Massive Facies</u>: typically internally homogenous in particle size and either massive or displaying a poorly to well developed undulose layering. This facies usually displays a discordant, erosional lower layer and a planar

concordant upper surface, Herbert (1). Shale breccia commonly occurs within troughs above this erosional surface.

2 Sheet Facies: Sandstone in this facies consists of cosets of trough or tabular cross strata which are bounded by subhorizontal bedding surfaces. Cross bedded sets range from a few centimetres to more than 5m in thickness and commonly dip to the north to northeast. Syndepositional convolution and recumbent folding of foresets is common in this facies, Conaghan (2) and is probably due to mass movement events shortly after deposition involving the unconsolidated sediments

3 Mudstone Facies: This facies is laterally discontinuous and usually between 0.3 - 3m thick. It is composed of grey, fissile mudstone which in places is slightly carbonaceous and is often laminated with fine grained sandstone. The facies is often referred to by the terms “Shale lens” or “Laminite”.

2.2 BEDDING

The results of the site investigation indicated that bedding was typically subhorizontal and typically planer to undulose, with high horizontal continuities. Sand to Clay infill was encountered along approximately one third of the measured bedding defects.

2.3 CROSS BEDDING

Cross bedding was inferred to typically dip towards the northeast. In fresh or slightly weathered sandstone at the depths of the tunnel, cross bedding was not anticipated to form planes of weakness. However, in moderately to highly weathered sandstone the cross beds were considered likely to form surfaces of incipient parting or low shear strength.

2.4 JOINTING

Based on oriented hole core and the limited surface mapping data, the dominant joints were inferred to be subvertical and strike north-northeast, although occasional vertical joints orthogonal to the main north-northeast system were interpreted. The joints were anticipated to have substantial horizontal and vertical continuity, and be planar, rough and mostly clean.

2.5 FAULTING

Faulting in the Hawkesbury Sandstone is uncommon. However, a fault zone, termed the Woolloomooloo Fault Zone (WFZ) was encountered while drilling for the piers of the Eastern Suburbs Railway viaduct in the late 1960’s. It was expected that the WFZ would intersect the main tunnel between 160m and 240m in from the northern portal, and at the William Street Ramp Tunnel portal. Minor sub-parallel faults or shears were expected to intersect the main tunnel up to approximately 450m in from the northern portal.

The zone was inferred to be steeply dipping to the southeast; vertical displacement across the zone was interpreted to be about 5m, with the eastern (hanging wall) side up-thrown, ie an overall reverse motion.

Stress measurements were undertaken in a borehole just to the north of Stanley Street, using hydrofracture techniques. The results indicated that the horizontal stress was about five times the overburden pressure which was considered to also indicate reverse motion across the WFZ.

Borehole data suggested that additional minor faults or shears could possibly occur near the intersection of Flinders Street and South Dowling Street, ie approximately 200m in from the southern portal of the main tunnel.

Low angle thrust faults, which merge into bedding plane shears, are known to occur within the Hawkesbury Sandstone but were difficult to detect in borehole core. A thrust fault was observed in an exposure to the north of the tunnel. It was considered probable that thrust faults would be encountered along the tunnel route.

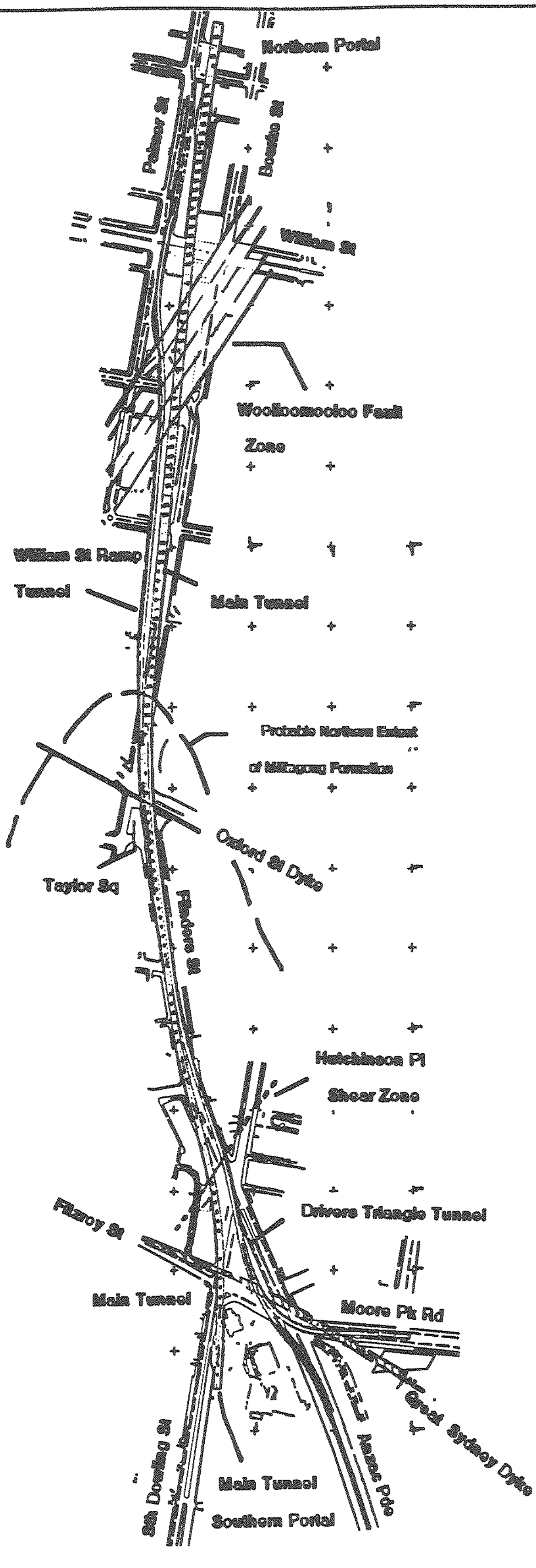

Figure 1 Tunnel Location and Encountered Geology

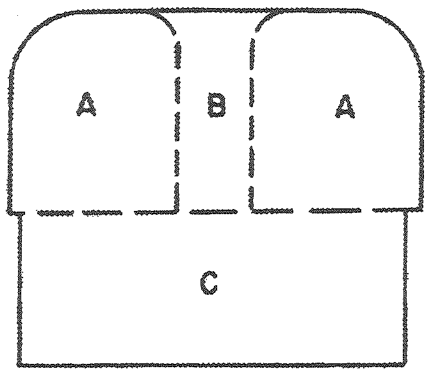

Figure 2 Tunnel excavation sequence. (See text for explanation.)

2.6 IGNEOUS DYKES

Two near-vertical igneous dykes were interpreted to intersect the tunnel route (Figure 1):

- a relatively thin (0.3 to 0.6m wide) dyke underneath Oxford Street (the Oxford St Dyke), and
- the Great Sydney Dyke which runs along the alignment of Fitzroy Street. The dyke varies in thickness from 5 to 7m and, at the depth where it is penetrated by the tunnel, was inferred to comprise stiff fissured clay (extremely weathered dolerite).

3 TUNNEL EXCAVATION

3.1 EXCAVATION SEQUENCE

Excavation was mostly by roadheader. Excavation of the main tunnel typically involved the following sequence (Figure 2):

1. Excavation of 5 to 6m wide parallel headings at Northbound level
2. Pillar stripping
3. Southbound excavation.

At the peak of excavation activity, six roadheaders were operational with a maximum advance rate of 80m per week for a 6m wide heading attained.

3.2 COMPANY ROLE AND PERSONNEL OBJECTIVES DURING CONSTRUCTION

Pells Sullivan Meynink Pty Ltd (PSM) had the responsibility for the design of roof and side wall support of the Eastern Distributor Tunnel. The basic philosophy for tunnel support involved the concept of a linear arch being formed in the rock above tunnel roof which was supported by rockbolts and shotcrete.

PSM's role on site was involved with the on-site "Surveillance" team, whose job it was to ensure that the support for the tunnel was installed as designed. Where ground conditions differed substantially from the design geotechnical model, it was the responsibility of Surveillance to provide support redesign. PSM's task was therefore to check that geological reality reasonably matched the design model and to undertake redesign when and where significant departures from the design model occurred.

Significant departures from the design geotechnical model would have included such scenarios as:

- relatively continuous moderately dipping defects,
- in conjunction with monitoring, evidence of low horizontal stress (eg. open joint planes),
- ground water inflows greater than expected,

- faults and shears other than predicted, and
- weathering conditions other than predicted.

The objectives of the Engineering Geologist's role were therefore to:

- modify and update the geotechnical models used for design purposes,
- forecast tunnel conditions, more particularly in twin headings, and through known structures,
- reduce the risk of 'geological surprises',
- allow quick response and localised redesign of tunnel support where 'geological surprises' were encountered, and
- provide a record of tunnel conditions for the future.

These objectives were achieved by:

- engineering geological tunnel mapping of all Northbound headings as well as Southbound walls,
- observation and interpretation of drilling for rockbolts and pipe canopy tubes to assess geotechnical conditions above the tunnel roof,
- routine sample collection for point load strength testing , and
- creation and management of a database containing the properties of mapped defects (orientation, length, persistence, roughness etc).

The major task was in-tunnel mapping. Detailed 1:100 scale maps were produced for all headings and included both sidewalls and the roof. When access allowed, the active face was logged. An example of a completed mapping sheet is shown in Figure 3.

Mapping of a particular heading typically lagged some 20 to 30m behind the tunnel face because the roadheader and haul trucks severely affected compass readings. Periodic logging of the face itself was necessary, however, as the face allowed a cross-tunnel section of the local geology to be viewed as well as assessment of the latest excavation conditions.

3.3 DATA FLOW

It was attempted where feasible to provide support redesign to the main contractor in what was termed 'real time'; or in other words with a minimum of delay. The theoretical data flow from in-tunnel mapping and monitoring through support redesign to support installation is shown in Figure 4.

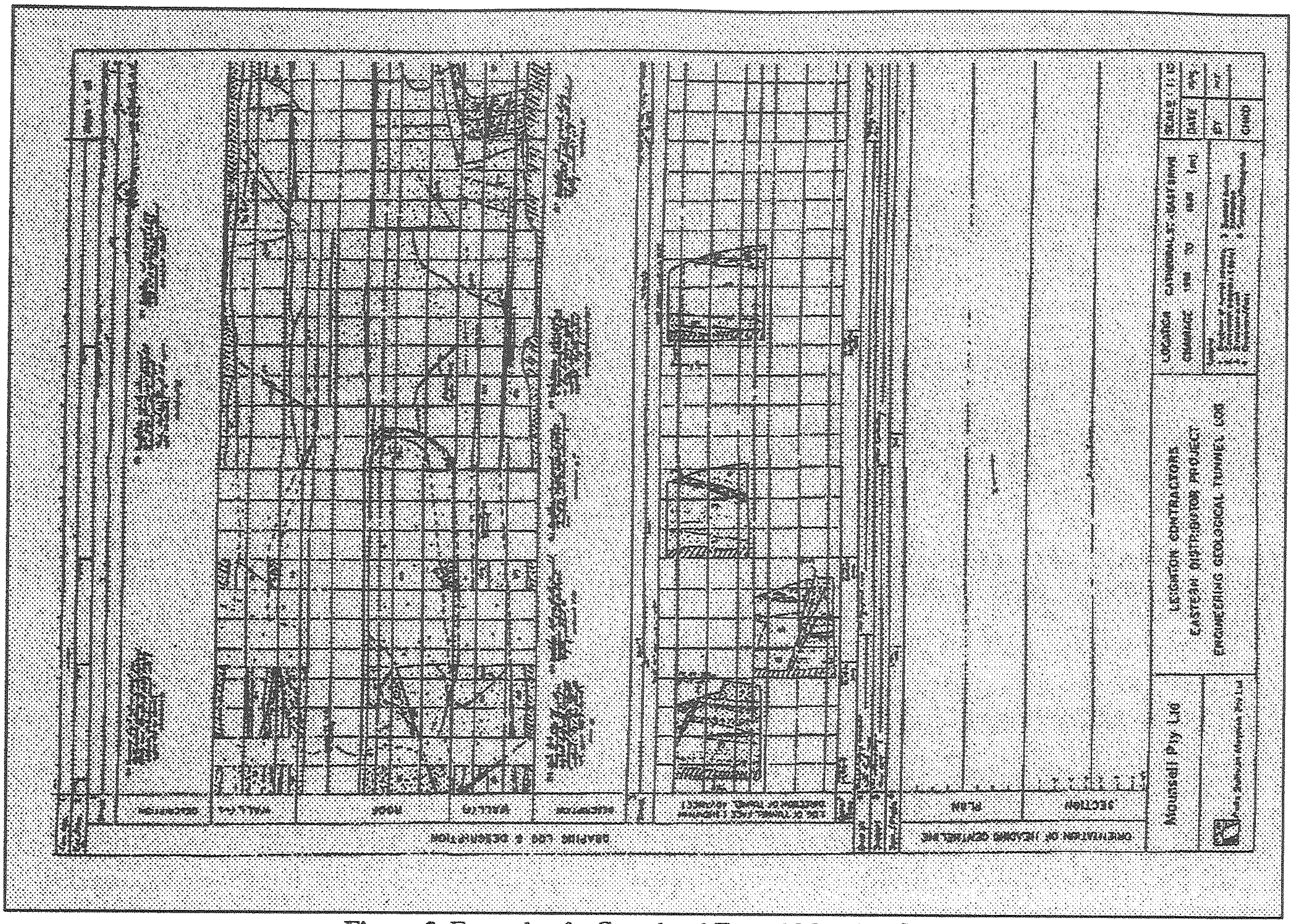

Figure 3 Example of a Completed Tunnel Mapping Sheet

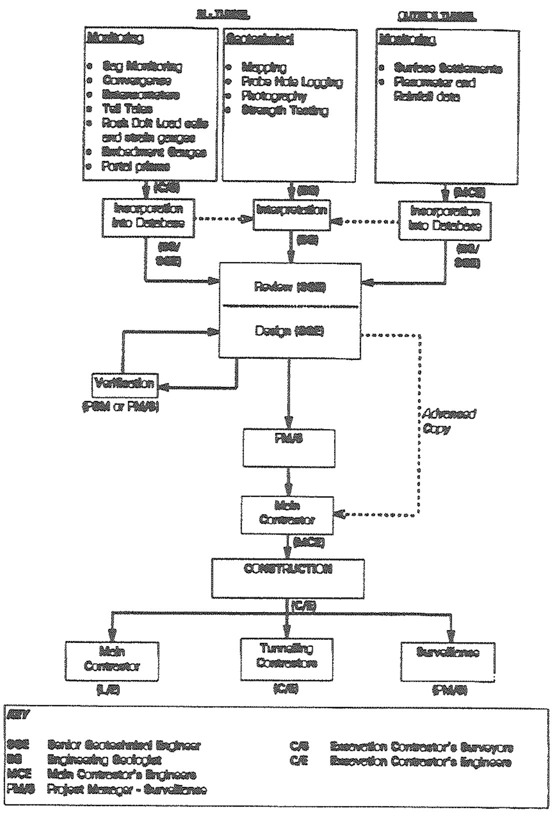

Figure 4 Theoretical Data Flow Sequence

Support review and redesign was carried out by a Senior Geotechnical Engineer, also from PSM, on the results of in-tunnel mapping and monitoring.

In reality, the roles performed by the Engineering Geologist and the Geotechnical Engineer were somewhat overlapping and became more so in the latter stages of the
project. The Geotechnical Engineer sometimes performed mapping, whilst the Engineering Geologist routinely undertook support design, particularly sidewall bolting. This had the advantage of 'decompartmentalising' both roles, ie the data flow sequence did not come to a grinding halt because of personnel unavailability etc.

The collection of data from in-tunnel monitoring was the responsibility of the tunnelling excavation contractor. Surveillance had overall control of where monitoring points were installed, and the frequency of reading. It was the responsibility of Surveillance to maintain a monitoring database.

3.4 DATA COLLATION

Data collation involved:

- daily updating of standard plans and sections in paper form,
- incorporation of structural data into a standard database from which defect stereoplots and defect histograms were produced, and
- inclusion of the results of Point Load, or other, testing into a testing database.

The histogram sheet and stereoplots were typically generated for 100 – 200m segments of the tunnel and allowed a quick visual check to be made between the design model and the encountered conditions.

3.5 SOUTHBOUND WALL BOLTING

The Eastern Distributor Tunnel is designed as a piggyback tunnel with Northbound traffic supported on a 600mm thick steel reinforced concrete decking, supported at each end on a 400 to 500mm wide sill beam.

Imposed loads of up to 827kN/m were calculated at the widest span of the tunnel. The orientation of the major joint set at an acute angle to the tunnel alignment and the imposed loads acting on the sill beam meant that potential "one sided" wedge failure, Pells and Dai (3), under the sill beam could occur. This failure mechanism involves release along an existing joint plane with shearing and tensile failure through intact rock (Figure 5).

Depending on the orientation of a joint beneath the sill beam and the local tunnel width, one of a series of four different patterns was installed adjacent to the joint. Support patterns were marked up during southbound wall mapping. This served to minimise the delay between mapping and support installation and allowed a 'one-pass' operation for the Engineering Geologist.

4 ELEMENTS OF POST CONSTRUCTION MODEL

For most of the length of the tunnel, the mapping confirmed the preconstruction model. The following sections provide a summary of the characteristics encountered for various features. Figure 6 summarises the post-construction engineering geological model.

4.1 HAWKESBURY SANDSTONE

Sheet and Massive Facies Sandstone as well as the minor Mudstone Facies were encountered during mapping. In general the Hawkesbury Sandstone encountered was typically classified as Class II with Class III near the portals (after Pells et al, Reference 4). A general profile can be summarised as:

- Mixed Massive and Sheet Facies Sandstone with few shale horizons (encountered in the southern half of the tunnel) overlying;
- Sheet Facies Sandstone with relatively common shale or shale breccia horizons (encountered in the northern half of the tunnel).

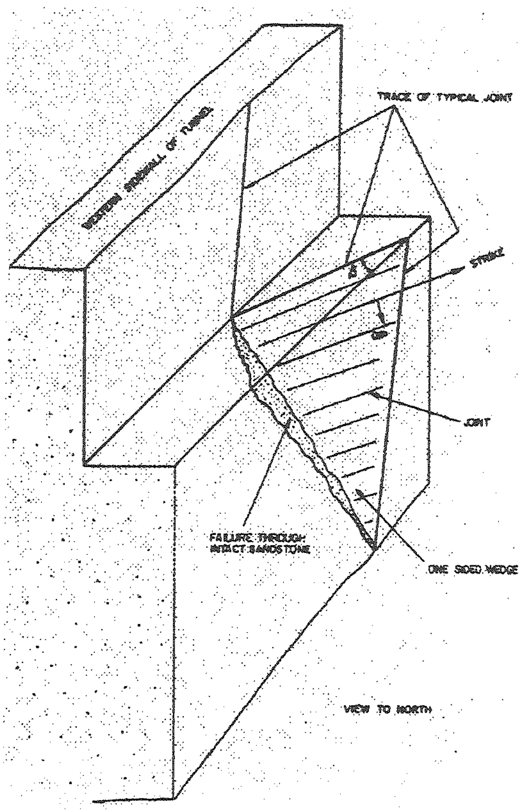

Figure 5 Geometry of One-sided Wedge Failure

The shale breccia was found to comprise zones of reworked and/or syndepositionally deformed shale within a fine to medium grained sandstone matrix. The horizons had a typical maximum thickness in the order of 1m and occurred at greater frequency than anticipated from borehole logging. Typically, the borehole logs indicated the horizons as one or two discontinuous shale lenses in a sandstone matrix, and were mostly noted as being minor features within the drill core. However, exposure within the tunnel indicated these apparently minor features typically formed part of a more extensive zone, with individual continuities of up to 100m.

4.2 BEDDING

Figure 7 presents the stereographic projections of the structure mapped during tunnelling and shows that bedding was mainly sub-horizontal. Analysis of bedding defects recorded during tunnel logging indicate the following defect characteristics:

- bedding spacing averaged about 2.0m;
- 85% dip less than 10°, 95% dip less than 20°;
- 60% had some infill varying from a stain to over 50mm;
  - 5% of which are Clay,
  - 50% of which are sandy Clay with or without some iron oxide,
  - 25% of which are clayey Sand,
  - 10% are Sand, and
  - the remaining 10% either iron oxide or carbonaceous material.

Clay dominated infill typically occurred near the surface whereas sand dominated infill occurred at the depth of the tunnel. This is interpreted to be a weathering related effect.

4.3 CROSS BEDDING

Figure 7 shows that cross bedding typically dips at 15° towards 045°. The mapping also indicated the following characteristics:

- cross bedding spacing was typically less than 0.2m;
- average length of cross bedding is approximately 4m;
- 66% dip between 10 and 20° typically to the northeast; and
- approximately 50% of the measured defects had minor infill of up to 3mm.

During logging, focus was placed on those defects that had some infill. Hence, the proportion of infilled cross beddings calculated above is much higher than reality.

In Class II sandstone or better, the cross bedding surfaces were not noticeably weaker than the rest of the rock material, and as such, did not tend to form planes of separation within the rock mass. However, in zones of Class III, or poorer, sandstone (ie within the WFZ and adjacent to the Great Sydney Dyke), cross bedding planes acted as release surfaces for roof failures. In these sections roof failures were typically defined by 'feather edging' on cross bedding with side release on joints.

4.4 MITTAGONG FORMATION

The Mittagong Formation formed the roof of the Drivers Triangle Ramp tunnel. The sequence comprised highly to extremely weathered interbedded sandstone and black mudstone (Class IV/V), and displayed a gradational relationship with the underlying Hawkesbury Sandstone which was assessed to be Class II/III.

4.5 JOINTS

The major joint set was confirmed to strike north-northeast and the minor set east-southeast (Figure 7). Both defect sets occurred as discrete features or as swarms of about 3 to 10 defects. Defect spacings within the swarm range from 20 to 500mm, whilst spacing between swarms was up to 20m. Typical joint characteristics are outlined below.

- 15% are continuous out of both the floor and roof of the tunnel, crossing several bedding planes, with a vertical continuity greater than 10m.
- 45% are semicontinuous, and transgress one or more bedding planes, with a vertical continuity of between 3 and 10m.

No clear correlation exists between sandstone type and joint occurrence or characteristics. However, joints were much less frequent in the central part of the tunnel compared to near the portals. This may be an effect of near surface stress relief.

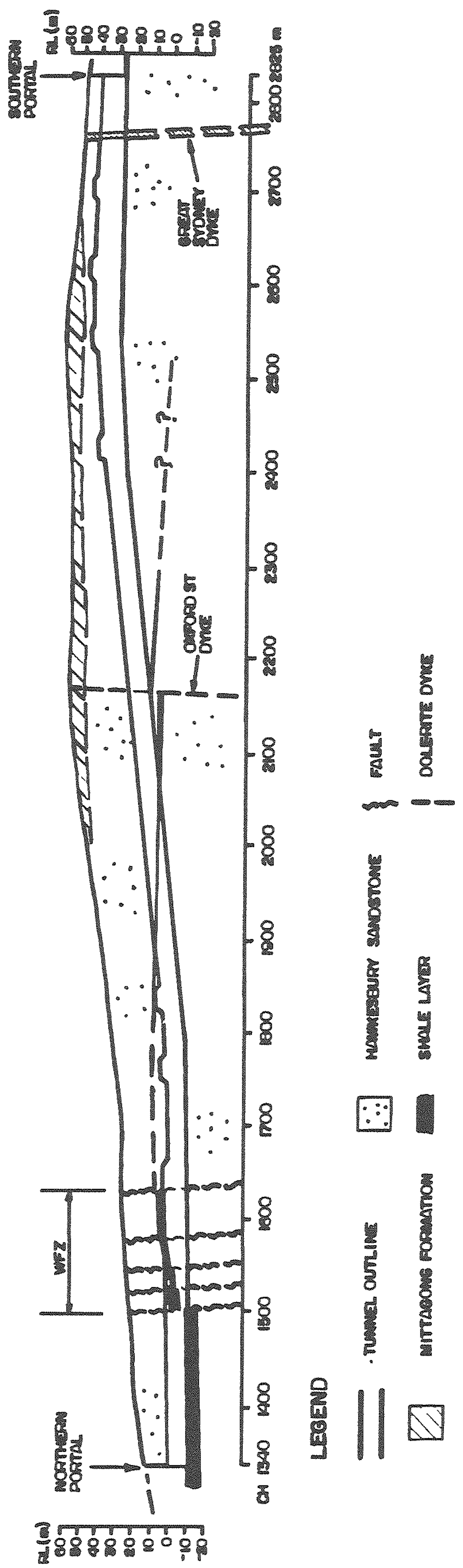

Figure 6 Post construction model

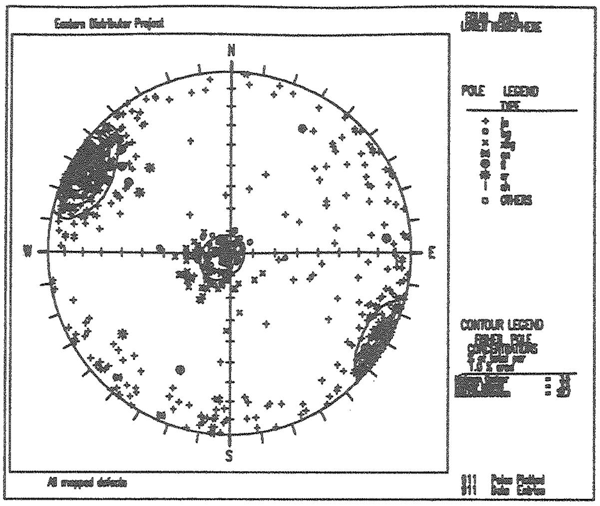

Figure 7 Stereoplot of defect data (relative to true north)

4.6 FAULTS

The WFZ and low angle thrust faults were encountered during tunnelling. Low angle thrust faults were recognised on both sides of the Great Sydney Dyke, and adjacent to, or as part of the WFZ.

The low angle thrusts ranged in length from about 1m up to approximately 20m. A sandy clay infill of up to 50mm was typical. The faults were often formed between adjacent bedding planes. This characteristic is interpreted to mean that:

- some amount of movement occurs along bedding planes, and
- the low angle thrusts serve to provide a movement surface, or 'connection path', between adjacent bedding planes.

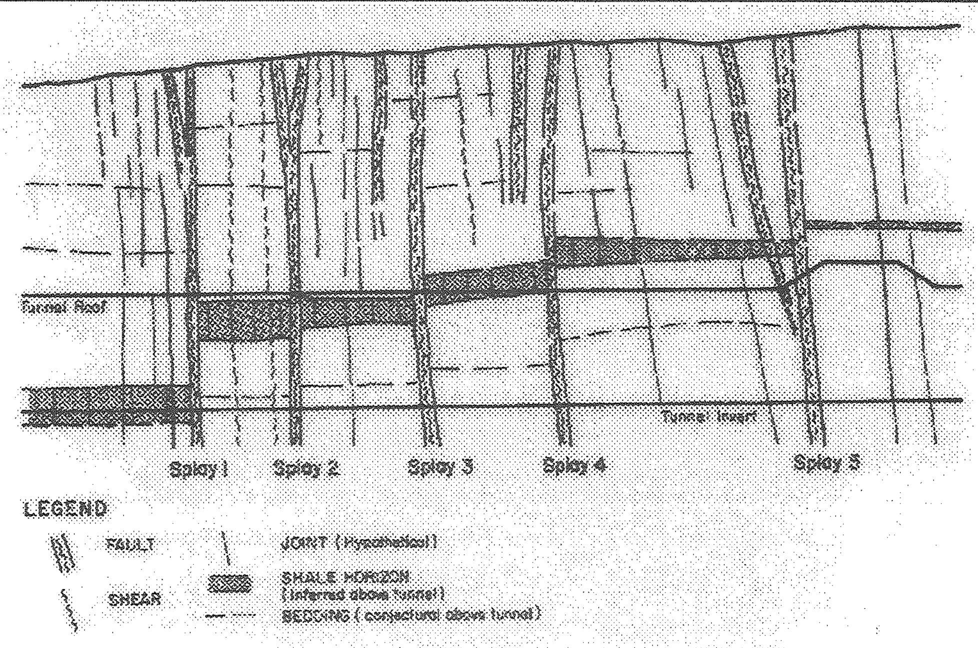

Figure 8 Interpreted Cross Section of the Woolloomooloo Fault Zone

The WFZ was intersected in the main northern tunnel between Ch1521-1625m with parallel faulting extending to 1700m, and at the portal of the William St Ramp tunnel. The WFZ is approximately 50m wide, dips at about 70° towards the southeast, and intersects the tunnel at about 25°. The WFZ comprises five major fault splays, (Figure 8) comprising 50 to 1000mm of crushed sandstone within a soft to firm clay matrix. Smaller faults and shears were apparent between the major splays. These minor faults had a maximum width of about 20mm. Rock quality between the fault splays improves towards the south, starting as poor between splays 1 and 2 and becoming very good between splays 4 and 5.

Overall movement of the zone is interpreted to be reverse (ie northwest side up relative to the southeast), although some of the fault splays are normal. Mapping indicated a displacement across the fault of approximately 8m. The component (if any) of strike slip motion was not able to be determined.

The gross characteristics and the overall sense of movement of the WFZ was accurately determined in the design geotechnical model, however, the number and position of the major fault traces was not realised.

The zone of minor faulting near the intersection of Flinders Street and South Dowling Street interpreted in the design model was intercepted during tunnelling, and was termed the "Hutchinson Place Shear Zone" (Figure 1). The zone was intercepted some 190m in from the main tunnel southern portal and comprised a number of closely spaced joints shears and faults. Vertical displacement of up to approximately 8mm was observed on some planes. The overall orientation of the zone is interpreted to be subparallel to both the WFZ and the major joint set, ie northeast - southwest.

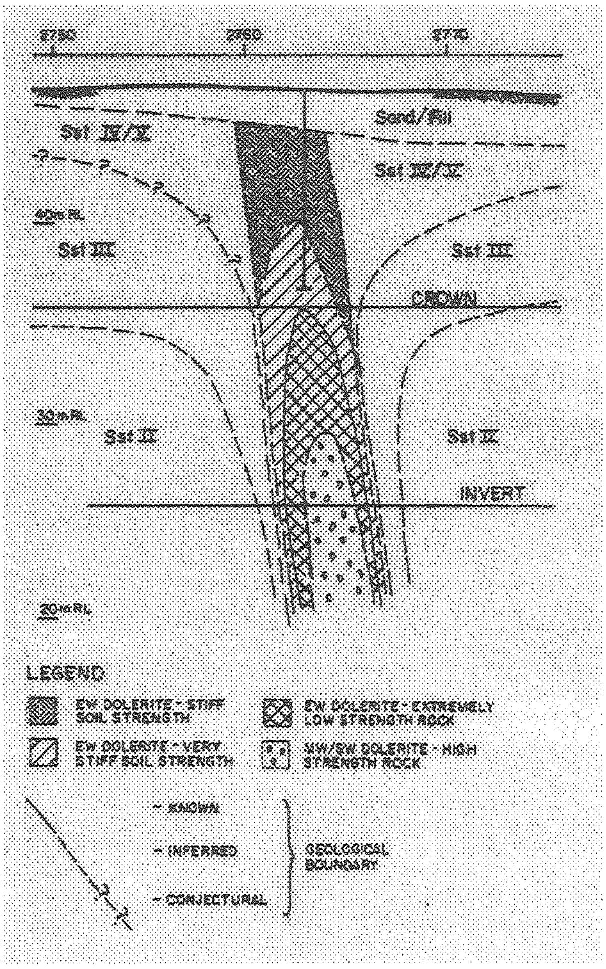

Figure 9 Insert caption here if at all possible at any stage

4.7 DYKES

Both the Great Sydney Dyke and the Oxford Street Dyke were intersected within 1m of their expected positions.

The post-excavation geotechnical model for the Great Sydney Dyke is shown in Figure 9. Key features of the model are:

- at tunnel level, the dyke comprises extremely weathered low strength rock with stiff clay near the sandstone contacts,
- the central core of the dyke is relatively less weathered than material near the contacts with the surrounding sandstone rock,
- the quality of the dolerite improves with depth to very high strength rock, and
- sandstone rock quality is degraded adjacent to the dyke. This is likely to reflect a combination of;
  - fracturing/faulting within sandstone prior to emplacement of the dyke,
  - baking of the sandstone immediate to the dyke during emplacement,
  - possible post emplacement faulting and weathering effects.

The design geotechnical model for the tunnel was able to quite accurately determine the location, thickness and material properties of the dyke at intersection depth.

5 CONCLUSIONS

Engineering Geological mapping and database upkeep during construction largely confirmed the design geotechnical model. In situations where the encountered excavation conditions varied from that predicted, data collected during mapping allowed for accurate redesign of tunnel support in 'real - time'.

The mapping sheets provide a complete record of the geotechnical conditions encountered during tunnelling. Given the potential traffic volumes through the tunnel over the 100 year design life of the tunnel, the effort put in to the engineering geological mapping and data collection during construction is considered appropriate.

This paper therefore provides a working example of the issue raised by Stapledon (5) of 'putting the "geo" into "geotechnical", by

- using graphic representation of data on excavation logs,
- presenting detailed logs of high standard, and
- regular and on-going comparison between the encountered tunnelling conditions and the design geotechnical model.

ACKNOWLEDGEMENT

The author wishes to thank Leighton Contractors Pty Ltd and Maunsell Pty Ltd for their permission to publish this paper.

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MODELLING REINFORCEMENT RESPONSE IN A SOIL NAILED EXCAVATION

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The main purpose of shotcrete facing in soil nailed excavations is generally held to be the control of local failure in the soil between the nails. This relatively non-essential role has meant that facing design has remained reasonably rudimentary. In this paper shotcrete facings in soil nailed excavations have been analysed using a purpose built three-dimensional finite element program. A full-scale experimental soil nailed wall, constructed as part of The French National Research Project Clouterre, has been modelled using the program and comparisons between calculated and observed behaviour are presented and discussed. The parameters used in the simulations are justified using published data. Good agreement is found between calculated and published displacements, nail forces, stresses and failure mechanisms.

1 INTRODUCTION

Ease of construction, flexibility of design and economic competitiveness have meant that soil nailed excavations have become increasingly accepted as a feasible means of excavation support. However, despite there being a large pool of published data describing actual walls constructed and at least three widely accepted design codes, (Japanese design code (JHPC 1987), recommendations of Clouterre (FHWA 1993), British standards (BS 8006:1995)), there still remains much to be learned regarding working stresses within the reinforcement system. In particular, the earth pressures experienced by the shotcrete facing are largely unquantified.

Facing design for soil nailed excavations is presently based upon a number of simplifications of soil behaviour, or empirical rules of thumb. The existing design methods utilise three main techniques:

- Assume the earth pressures at the face are equal to a percentage reduction in the Rankine active earth pressure.
- Assume that the effect of overburden is negated by the action of the soil nails and design the facing to retain an active soil wedge forming between the nails.
- Assume the maximum nail tension forces are equal, or proportional to the expected total load at the face.

Little is understood about which parameters have the greatest effect on facing loading.

This paper presents a three-dimensional (3D) finite element program, capable of directly calculating the reinforcement response in a soil nailed excavation. Results regarding the determination of earth pressures at the facing will not be presented, rather the programs ability to accurately model; deflections, nail forces and failure modes experienced by a soil nailed excavation will be verified through comparisons of observed and calculated responses.

2 PROGRAM DESCRIPTION

For the analysis, a 3D finite element program, capable of simulating construction of soil nailed excavations was developed. A 3D simulation was considered necessary, so that the nails could be modelled as bars, (a 2D simulation represents the reinforcements as continuous flat sheets). When considering the moments induced in the facing, it was important that the 'point' restraint provided by the soil nails be reflected in the simulation.

Three types of element are used by the program; a twenty noded elasto-plastic 'soil' element, an eight noded isoparametric Mindlin Shell 'shotcrete' element and a two noded beam 'nail' element. The reinforcement elements share nodes with the main soil elements. As such, slip between the soil and reinforcing elements is not modelled. Inclusion of separate soil nail and shotcrete elements has meant that induced moments and forces in the support elements can be directly calculated.

Soil behaviour is characterised by an elasto-plastic (elastic - perfectly plastic) response curve. The Sloan and Booker (1986), modified Mohr-Coulomb yield surface is used to describe the failure criterion, and a Von Mises' plastic potential surface is used to define strain orientation at failure. Use of Von Mises' plastic potential means that soil dilation is not considered.

Excavation is simulated in steps, using Brown and Booker's (1985) virtual work solution to calculate the induced forces. A skyline solver has been incorporated, in order to reduce the memory required to run the program.

3 EXCAVATION SIMULATION

For simulation of excavation in elasto-plastic soils, it is essential that excavation be carried out in steps that reflect the actual excavation process. There are two reasons why this is necessary in soil nailed systems.

- During a stepped excavation the soil will experience stress states, which are different to its final stress state. As such, the simulated excavation process needs to properly reflect the actual excavation process, so that any plastic deformation and resulting permanent strain, experienced in intermediate construction stages is incorporated in the total excavation response.
- Without a stepped simulation process it would not be possible to properly model the influence of the incremental construction process. The layered nature of soil nailed wall construction, means that each layer of reinforcement has a unique zero stress position. This phenomenon needs to be properly mirrored in any simulation for meaningful results.

The incremental nature of a soil nailed wall's construction, has some bearing on the applicability of any simulation of a soil nailed system. In reality, the shotcrete and nail annulus, are placed wet onto the deforming excavation. All stresses within the reinforcement are induced by displacements after the shotcrete and grout have hardened. In the simulation, the time dependent nature of the displacements is not modelled. The reinforcements are either placed before the current excavation step is completed or alternatively, they are placed after all the displacements associated with the current step have occurred.

Figure 1 illustrates the two simulations for one excavation step. These two cases represent the limits of the actual behaviour. For case A, the stresses in the reinforcements are induced by displacements associated with the current step as well as displacements induced by following steps. For case B, the displacements due to the current step are said to have occurred before the reinforcing is placed. As such, the stresses in the reinforcement are only due to displacements induced by following steps. The actual behaviour of an excavation most likely sits between these two limits.

For this simulation, case B is taken to represent the excavation's behaviour, because it is believed that this case more closely represents reality.

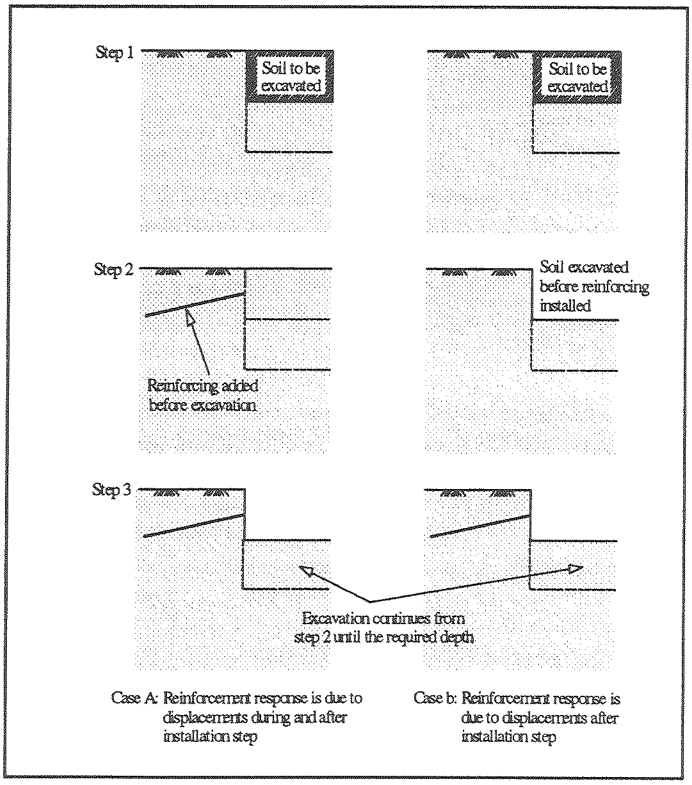

Figure 1 Excavation simulation

4 FULL SCALE EXPERIMENTAL WALL

Unterreiner et al. (1997), Schlosser et al. (1992) and Plumelle et al. (1990) present the results of a full scale experimental soil nailed wall constructed as part of the French National Research Project CLOUTERRE. Figure 2 shows a cross section of an embankment built for the project. The soil nail wall was constructed out of the embankment. Construction was incremental, with 1m high layers of the wall being built at a time. Nails were inclined at 10° to the horizontal, with nail length ranging from 6 to 8m. The facing was constructed of 100mm thick mesh reinforced shotcrete. The homogeneity and density of the backfill used to construct the embankment was tightly controlled. Excavation induced displacements were measured using inclinometers, positioned as shown.

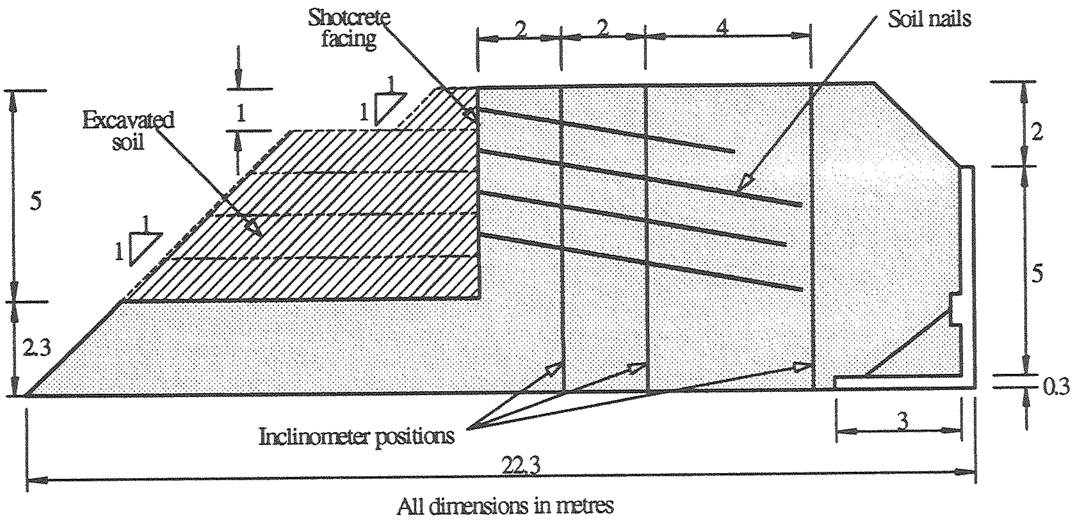

Figure 2 Embankment dimensions

Extensive testing was used to determine soil strength parameters. A Menard pressuremeter was used to calculate soil modulus and triaxial testing to determine the friction angle.

4.1 SIMULATION OF EXPERIMENTAL WALL

The large amount of testing conducted as part of the Clouterre project, and the controlled nature of the embankment fill in which the soil nailed wall was built, meant that a high degree of confidence could be placed in the soil parameters reported by Clouterre. As such the simulation makes use of these parameters wherever possible.

Table 1, shows comparisons of reported and simulated soil parameters. Figure 3, shows a comparison of the measured and simulated Young's Modulus. Initial stresses were considered to be geostatic, with K_o calculated using the relationship shown in Table 1.

	Reported	*Simulation*	*Method of determination*
φ	36°-40°	38°	Triaxial tests
C	3-4 (kPa)	3 (kPa)	Triaxial tests, water content
K_o	0.38	0.38	$K_o = 1\text{-}\sin\varphi$
γ	16.1 (kN/m^3)	16.1 (kN/m^3)	Avg. unit weight

Table 1 Soil parameters used

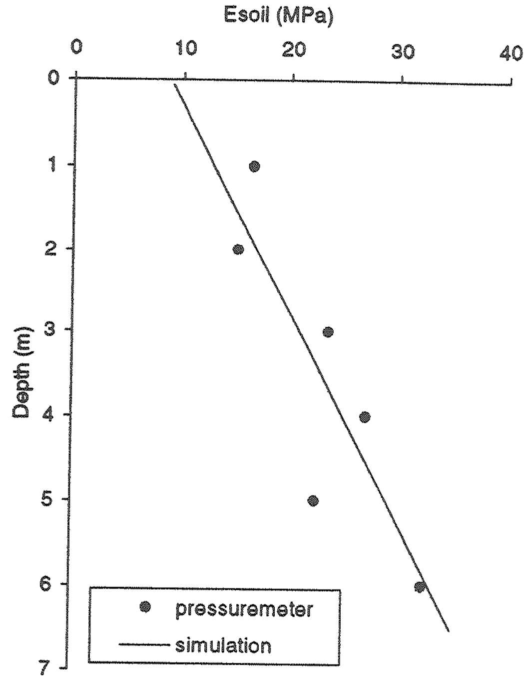

Figure 3 Measured and Simulated E_{soil}

4.2 COMPARISON OF RESULTS

Figures 4 (a)-(d) show the comparison of calculated and observed horizontal displacements. Results for two excavation stages (wall heights, 3m and 5m) are shown in each figure. Figures 4(a) and (b) (displacements for the facing and 2m behind face) show excellent agreement. In Figures 4(c) and (d), differences appear between the calculated and observed readings. Inconsistencies in the observed data suggest that some of the inclinometer readings are erroneous, particularly the observed displacements for the 5m deep excavation in Figure 4c, where the inclinometer data shows the soil moving away from the excavation.

Figures 5 (a)-(d) show comparisons of observed and calculated nail forces. A fair agreement is found for the size of the nail loads. Figures 5(b) and (c) in particular show that the program can over estimate the size of the nail forces. This is most likely attributable to the assumption of full fixity between the nails and soil. There is however a good agreement between the shapes of the observed and calculated nail force distributions. The position of the maximum nail force gives an indication of where the failure plane intersects the nail. As such, the good correlation between the positions of observed and calculated nail force maximums, shows that the program is properly simulating observed failure mechanisms.

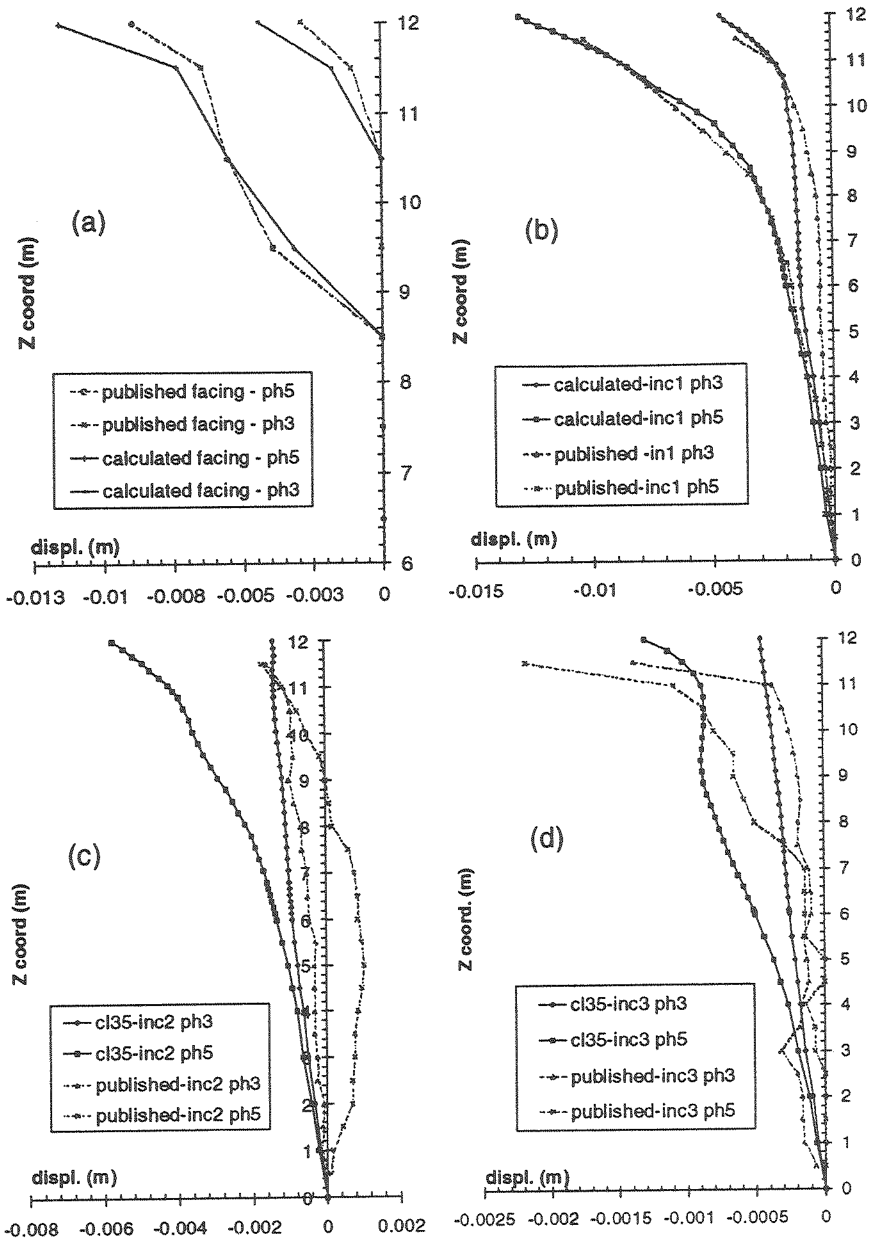

Figure 4 Horizontal displacement (a) at face (b) 2m from face (c) 4m from face (d) 8m from face.

5 DISCUSSION

Comparisons between observed and calculated; displacements, nail loads and failure mechanisms show that soil nailed excavations can be simulated using the presented 3D finite element program.

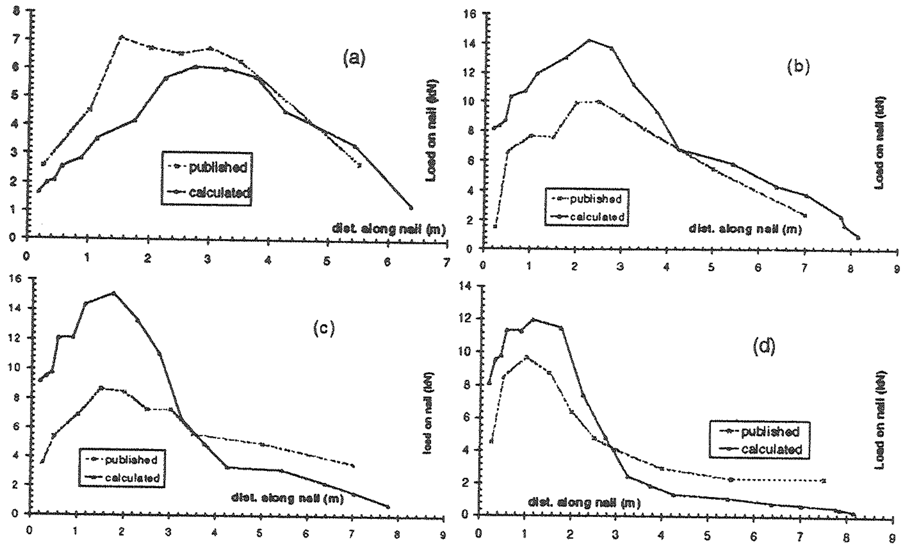

Figure 5 Force distribution in nails (a) top nail (b) second nail (c) third nail (d) bottom nail.

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PHILIPPINES NATIONAL RAILWAY MAIN LINE SOUTH REVITALISATION PROJECT PHASE II

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A series of railway embankment slope stability failure located on the Main Line South between Gapo and Bongalon, 8 km to 10 km north of Legaspi City on the south-eastern corner of Luzon Island in the Philippines, was identified by Ove Arup & Partners in 1996. This paper describes the mechanism and reason for failure of one of the embankments (TP9383), the results of the ground investigation and the preliminary design of stabilising measures.

1 INTRODUCTION

In 1995 the contract for Phase II of the Main Line South Revitalisation was awarded by the Philippines National Railway (PNR) to John Holland Constructions (Asia) using funding made available by AusAID. TMG International in association with Kinhill and Ove Arup & Partners were commissioned by the PNR in May 1996 to assist the client project management his duties and provide engineering advice as required. The Main Line South runs from Manila Central Terminus for 478 km to Legaspi City on the south-eastern corner of Luzon Island.

Construction of the remedial works commenced in June 1995. Ove Arup & Partner's geotechnical involvement started (August 1996) with a walk-over survey of the existing track to identify areas of concern in terms of the overall geotechnics, but concentrated on earthworks and embankment stability. Five major railway embankments failures were identified between Gapo and Bongalon, 8 km to 10 km north of Legaspi City which led to a series of geotechnical investigations being carried out in 1996 and 1997. This paper specifically discusses the failure at one of the embankments (TP9383).

2 BACKGROUND

Embankment TP9383 is situated in an area which is prone to earthquakes, volcanic eruptions, typhoons and floods. The embankment is approximately 15 m high and comprises three 5 m high slopes battered at about 1:1.5 (V:H) with 3 m wide benches. With regard to the railway alignment, the embankment is located on a pinnacle of high ground connecting a bridge which spans a meandering river. On the western side of the embankment the river is located at the toe of the slope which is therefore prone to scour and erosion. The eastern side of the embankment is located in a rice field which separates the toe from the river.

3 SITE OBSERVATIONS

The reconstructed embankment at TP9383 was inspected in August 1996 and found to have one open crack along the centre-line, 10 to 20 mm wide. The crack could be traced down the uppermost of the two batters on the embankment, but there was no indication of movement below this level at the time of the inspection. There was no sign of vertical displacement.

At the time of the site investigation (October 1996), the embankment had already slipped quite considerably. Measured displacements of up to 3 m (vertical) and 2 m (horizontal) were recorded at the back scarp of the failure surface at the top of the embankment (See Figure 1 below). Both vertical and horizontal displacements, as well as large open cracks, were observed down the batters of the failing embankment. A mound of soil was also observed at the toe of the embankment which probably represented the failed material from previous slips as well as this most recent failure. The length of the slip is approximately 22 m at the top of the embankment which then fans out to a length of approximately 47 m at the toe of the embankment.

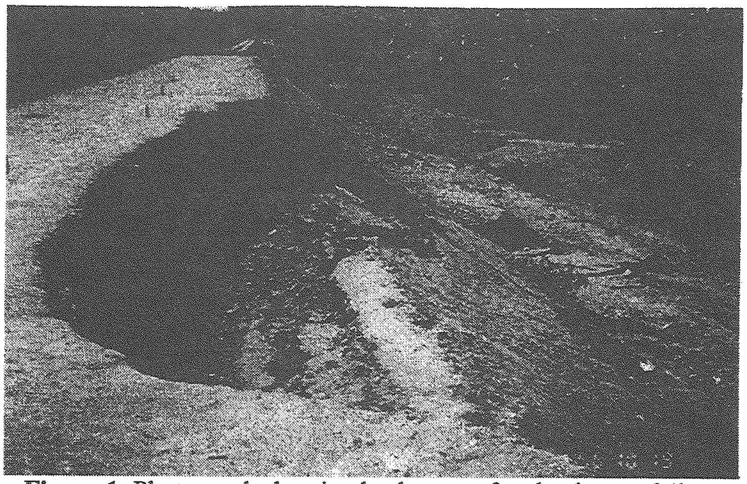

Figure 1 Photograph showing backscarp of embankment failure

During the site investigation, additional cracks opened up at the top of the embankment and down the eastern face of the embankment to the north and to the south of the slip under investigation. No signs of vertical movement were detected on these cracks.

Photographs of the original embankment prior to reconstruction indicate that there were three distinct slips which had occurred on the eastern side of the embankment in the past. The locations of these old slips coincide quite well with the locations of the current slips. This would tend to suggest that the newest slips were simply a reactivation of the old ones. It is understood that the remedial works involved excavation and reconstruction of the upper parts of the embankment, but did not extend to the location, excavation and replacement of the slip plane itself

4 SITE INVESTIGATION

A total of 3 boreholes were drilled down the face of the failing section of the embankment in September 1996 using wash boring techniques. SPT's were carried out in all boreholes with undisturbed 75 mm diameter thin wall tube sampling in the clays. Pocket penetrometer readings and shear vane tests were carried out in the tube samples to determine the undrained shear strength. A single standpipe piezometer was installed in each borehole to monitor the groundwater pressures in the bedrock. During the course of the investigation 4 survey monitoring points were installed down the face of the embankment to monitor surface movements. Monitoring was carried out during the investigation and showed that the embankment continued to move during this period which resulted in the loss of one length of drill casing.

Laboratory testing comprised indicator tests including natural moisture content (6 No.) and Atterberg limits (6 No.) which were carried out by an independent laboratory at the Philippines National University in Manila. Sophisticated triaxial testing was not undertaken as a reliable laboratory in the Philippines with the capability of carrying out such tests could not be identified at the time of the investigation.

5 RESULTS OF INVESTIGATION

5.1 GEOLOGICAL PROFILE

The subsurface profile adopted for analysis is shown in Figure 2. The profile generally comprised up to 9 m of fill overlying 3 m to 4m of soft colluvium (medium-high plasticity silty CLAY) which in turn overlies 2.5 m to 4 m of soft flood plain alluvium (medium-high plasticity silty CLAY) near the toe of the embankment.

The colluvium and alluvium are located above a layer of residual soil, relatively consistent in depth, overlying a fine grained igneous bedrock (Basalt & Dacite). The bedrock appeared to rise toward the top of the embankment, suggesting the embankment was originally constructed on top of a rock outcrop carved by the action of a meandering river.

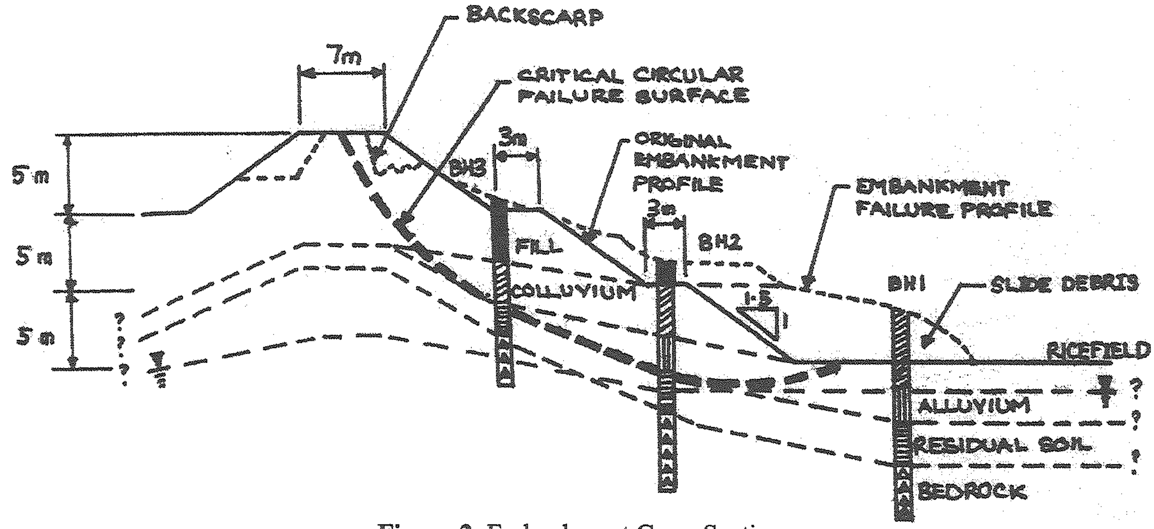

Figure 2 Embankment Cross Section

5.2 GROUNDWATER CONDITIONS

The maximum recorded water levels were used to back-analyse the slope failure. It should be noted that embankment TP9383 was reconstructed during the "wet season" and failure of the embankment occurred during the same period.

5.3 LABORATORY RESULTS

In BH1 the alluvium encountered at the toe has a natural moisture content (84%) in excess of the liquid limit (55%) indicating that the material is very soft to soft which is relatively consistent with the SPT (N=2) and pocket penetrometer (PP=50 to 100 kPa) results. Towards the top of the embankment (BH2 and BH3), the natural moisture content (43% to 53%) of the colluvium approached the liquid limit (46% to 61%), suggesting that the colluvium is soft to firm throughout the embankment. This is relatively consistent with the SPT (N=5 to 8), pocket penetrometer (25 to 100 kPa) and shear vane (S_u=60 kPa) results.

6 STABILITY ANALYSIS

6.1 GEOTECHNICAL MODEL OF SLIP FAILURE

The in-situ geometry, soil profile and groundwater data collected in the field were used to back-analyse the failure. Numerous circular and non-circular failure planes were analysed to determine the limiting equilibrium conditions (Factor of safety=1.0) which produced a slope failure similar to that observed on site. For embankment TP9383 it appeared that the recent failure had been the result of the reactivation of an old slip. Surface observations were indicative of a circular type failure through soft clay.

6.2 SOIL PARAMETERS

Although the immediate failure of the newly reconstructed slope suggested an undrained (c_u) failure in the underlying clay layers, a failure along a previous slip indicates a drained failure (ϕ', c') during periods of prolonged high water levels. Both of these possibilities were considered in the analysis and design of remedial works. The soil parameters initially assumed for analysis were determined from the borehole logs, laboratory test results and site observations. Soil parameters determined from the back-analysis are given in the table below.

Material	((kN/m^3)	c_u (kPa)	ϕ' (°)	c' (kPa)
Fill	20	---	25	5
Colluvium	18	30	25	0
Alluvium	18	30	25	0
Residual Soil	19	70	30	0

6.3 METHOD OF ANALYSIS

The slope stability analysis was undertaken using the two-dimensional slope stability analysis package "*SLOPE*", from the *Oasys GEOsuite* written by Ove Arup & Partners. Both circular and non-circular failure planes were analysed using the Bishop method with variably inclined interslice forces and the Janbu method with horizontally inclined interslice forces. The lowest factor of safety was obtained through calculation of a number of potential slip surfaces at a defined rectangular grid of centres, with the option of extending this grid to search for the overall minimum factor of safety.

6.4 BACK-ANALYSIS

All failure surfaces passed through a common point on the crest of the embankment which coincided with the scarp left by the actual failure plane on site, and surfaced at a point some distance beyond the toe where the evidence of failed material came to an end. The analysis indicated that the critical mode of failure may be an undrained failure in the soft clay or may be a drained failure in the soft clay due to high water levels. Both circular and non-circular failure planes yield a factor of safety close to unity in both cases.

7 STABILISING MEASURES

7.1 FACTOR OF SAFETY

Due to the high economic and human risk associated with a slope failure of this kind, a minimum factor of safety of 1.4 for redesign was adopted (Geotechnical Manual for Slopes, 1984) for the groundwater conditions recorded on site. In addition, a factor of safety of at least 1.2 was targeted for the highest probable assessed groundwater levels.

7.2 LOADING CONDITIONS

A surcharge due to train live loading was included as part of the preliminary stabilising measure design. Although not discussed here, an earthquake loading was also incorporated.

7.3 STABILISING OPTIONS

The following stabilising options were considered and analysed:

- *Small toe berm*. This involves excavating the fill and all soft material (colluvium & alluvium) that appear to be the cause of the problem and reconstructing the embankment with a small toe berm with slopes no greater than 1:1.5 (V:H). If soft material is found to extend below the water table, redesign would be required during the construction process, but would be based on the analysis carried out.
- *Large toe berm*. This involves excavating and replacing the fill and soft material (colluvium & alluvium) above the natural ground level and reconstructing the embankment with a larger toe berm with slopes no greater than 1:1.5 (V:H).
- *Install stone columns*. The use of stone columns would require cutting down the existing embankment to a level platform and stockpiling the excavated material.The use of stone columns would then be installed at close centres on a grid pattern over the full width and to beyond the toe of the embankment. A drainage blanket, with free draining granular material would then be placed over the stone columns to allow water to drain through the stone columns. This has the advantage that it would alleviate artesian water pressures if they develop in the underlying rock. The embankment would then be reconstructed over the drainage blanket with side slopes no greater than 1:1.5 (V:H).
- *Construct a bridge over the slip area*. This solution requires the piers and abutments to be piled to bedrock and sufficient soil removed between them to arrest the movement. Failure to do this could result in unacceptably high lateral pressures being developed on the sides of the bridge foundations and failure of the bridge itself.

The stabilising option adopted by the Philippines National Railway comprised constructing a bridge over the slip area. This option was selected because it was the most cost effective and could be completed within the project time scale.

8 INTERESTING ASPECTS OF THE PROJECT

Some personal aspects of this project and lessons to be learnt from working in a developing country such as the Philippines, which the author believes to be of special note are listed below.

Overseas work means different cultures, traditions, religions and languages. As an expatriate engineer you soon learn to be tolerant, respectful and understand the importance of these differences and their significance in the workplace. Working in an overseas country can be challenging, forces you out of your 'comfort zone' and builds self confidence. You quickly learn to take more responsibility, adapt to an unfamiliar work environment and use your own initiative to get the job done, often in a remote area where resources are limited.

Sophisticated equipment or methods of working are not always available in a developing country. As such, learning to retrieve as much information as possible from pragmatic drilling methods adopted by local contractors may be required. Working in an overseas country provides excellent experience in a geological environment which is often greatly different from home. Hence, new problems and new solutions. Australian practice is often not appropriate for solving geotechnical problems. Finance and technology are often the key governing constraints.

This project provided the author with a real insight into a variety of challenges of both a personal and technical nature. Overseas experience can therefore be invaluable to ones personal and professional development.

9 CONCLUSION

The following conclusions can be drawn from the experience on this embankment failure:

- A number of alternative preliminary stabilising measures were designed for the failing embankment.
- A comprehensive ground investigation must be carried out before the remedial works are designed, so that the designer has a full understanding of the failure mechanism and can develop an appropriate solution.
- Geotechnics formed an essential part of the remedial works.
- On a large infrastructure project such as this, it is essential that both consultant and contractor work closely together at the commencement of the project in order to produce an efficient and effective design. Unfortunately this was not the case on this project which resulted in expensive consequences.

10 ACKNOWLEDGEMENTS

The author wishes to thank TMG International for allowing this paper to be published and Ove Arup & Partners for giving the author the opportunity to work on such an interesting and rewarding project.

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NORTHSIDE STORAGE TUNNEL – LANE COVE RIVER CROSSING FROM INVESTIGATION TO COMPLETION

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Coffey Geosciences Pty Ltd

The Northside Storage Tunnel project is a system designed to collect and store sewage that presently enters Sydney Harbour from four major overflow points on the Northern Suburbs Ocean Outfall Sewer (NSOOS). The overflow of sewage during wet weather from the NSOOS is the main source of pollution for Sydney Harbour.

The Northside Storage Tunnel is a rock tunnel system comprising a 15.8 kilometre tunnel from Lane Cove River to North Head Sewage Treatment Plant and a 3.7 kilometre northern branch tunnel to Scotts Creek, Castle Cove. The crossing beneath Lane Cove River posed particular challenges in both the investigation and construction stages. This paper presents the investigation procedure, the geology encountered, the geological model developed, and the required design and subsequent construction.

1 INTRODUCTION

The Northern Suburbs Ocean Outfall Sewer (NSOOS) system is the main sewer collection system for the north shore of Sydney. At present, in times of heavy rainfall this system is inundated by more sewage and water than it was designed to carry. The majority of the excess enters the harbour at four of some twenty five overflow points. The Northside Storage Tunnel (NST) is designed to capture the flow from the four largest overflows. These overflows are located at Lane Cove River, Tunks Park, Northbridge, Scotts Creek in Castle Cove, and Quakers Hat Bay, Mosman. The captured sewer water drains to the east and is stored for later treatment and discharge from the North Head Sewerage Treatment Plant.

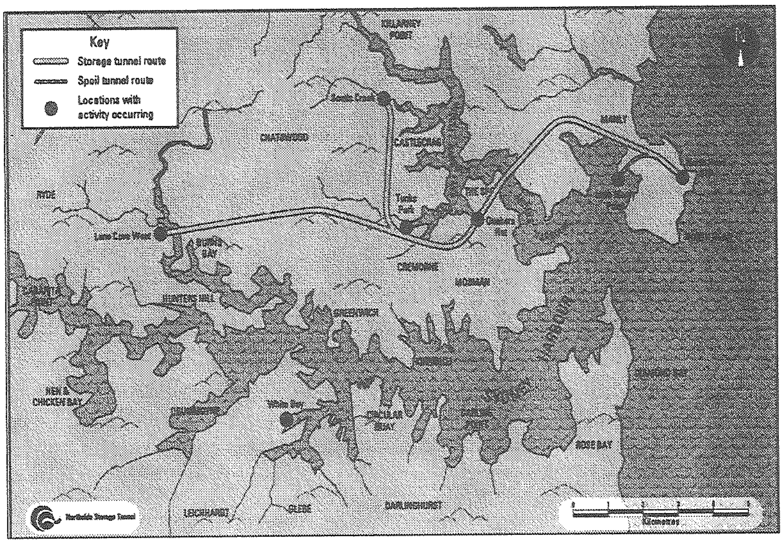

Figure 1 Northside Storage Tunnel Alignment (Sydney Water Web Site, 2000)

The NST is being constructed on an accelerated time frame to be ready before the Sydney 2000 Olympic Games. To facilitate this, the tunnel was designed with one construction methodology, whereby construction was undertaken entirely in hard rock. This required the vertical alignment of the tunnel being taken below the deep paleochannels at Lane Cove River, Middle Harbour and Manly and terminating at North Head with an invert level of RL –97.5m (AHD). The tunnel horizontal alignment essentially follows the existing NSOOS. (Parker et al 1998)

2 LANE COVE RIVER CROSSING

The Lane Cove River Crossing of the NST has involved tunnelling with a 3.8m diameter Tunnel Boring Machine (TBM). The tunnel was bored in a westward direction from the Tunks Park construction site. The tunnel passes beneath the river with an invert level of RL -43m and a horizontal alignment some 30m downstream of the existing NSOOS siphon tunnel (Coffey Report S10901/14-AG).

The Lane Cove River Drive of the NST was not always planned to cross beneath the river. At various stages during the initial phase of the project the tunnel was terminated at either the western or eastern shores of the Lane Cove River. The decision making process was influenced by the fact that the existing overflow is on the eastern shore and there was the possibility of future expansion of the NST to the western suburbs.

In February 1999, a decision on whether to proceed with the crossing of the Lane Cove River had to be made. More information was required to make that decision and as a result an investigation was undertaken.

3 INVESTIGATION OF THE LANE COVE RIVER CROSSING

The investigation of the crossing of the Lane Cove River focused on the paleochannel structure and depth beneath the river sediments, and whether the bedrock had been subjected to stress relief and valley floor bulging. During the initial investigation phase (mid 1998), a borehole (NS20) was drilled on the eastern shore of the river in the vicinity of the existing NSOOS siphon, tunnel and overflow. The emphasis of the later investigation (early 1999), consisting of NS42 and NS43, was towards the centre and western side of the river.

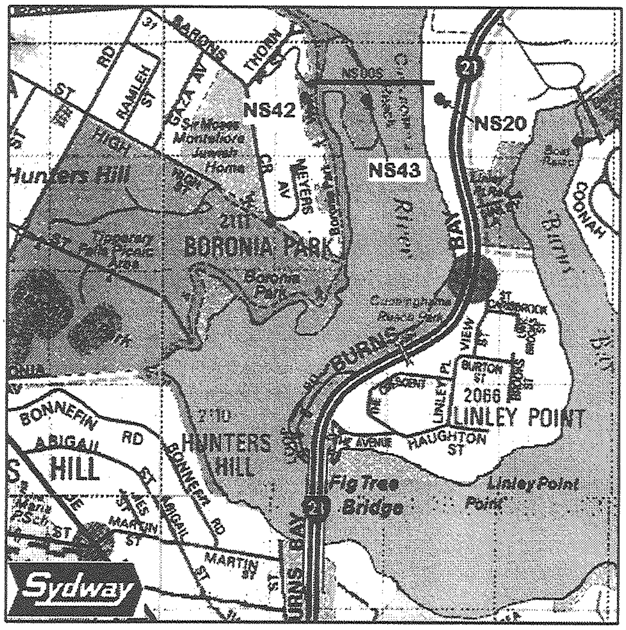

Figure 2 Location of the geotechnical investigations

Drilling work conducted for the NSOOS tunnel in 1928 had found the paleochannel to be located under a mangrove island on the western side of the Lane Cove River. In that investigation, 18 boreholes were drilled from the surface to the top of rock with three boreholes penetrating the rock. A cross section based on the 1928 investigation is shown in Figure 3 below.

In order to assess the accuracy of the 1928 borehole data and define the location and level of the bedrock paleochannel downstream for the proposed NST alignment, a series of three parallel seismic refraction lines were performed across the river. This data was used to locate an inclined borehole (NS43) drilled westwards towards the centre of the paleochannel, from the only accessible place on the mangrove island. Another borehole (NS42) was drilled vertically on the western shore at the site of the drop shaft and TBM arrival chamber.

Borehole NS42 was drilled using a truckmounted drilling rig. HQ-3 coring was commenced at the surface (RL 17.83m AHD) and terminated at 70.14m depth (RL –52.3m AHD).

Borehole NS43 was drilled using a Gemco HC10 drilling rig mounted on a Yanmar 8 wheel ATV, supplied and crewed by McDermott Drilling. The drilling rig was delivered to the site by barge. It was driven onto the mangrove mud flat, where it was jacked up onto a stable platform of wooden blocks above the high tide level.

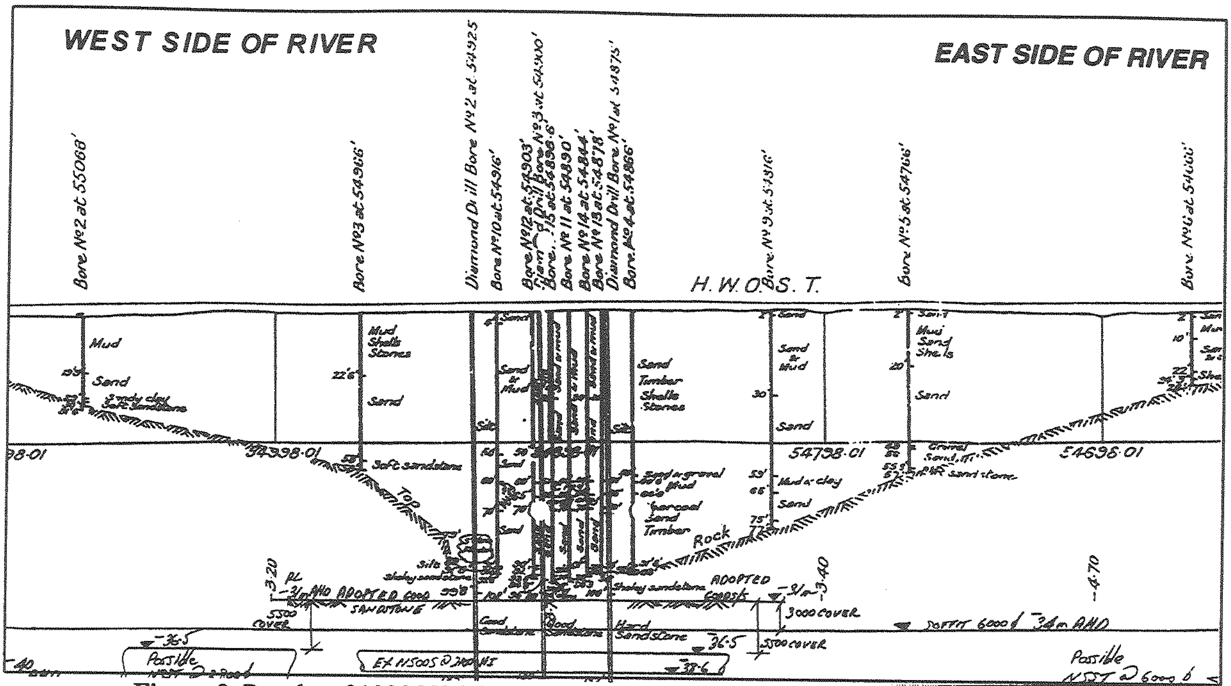

Figure 3 Results of 1928 NSOOS geotechnical investigation (Dept of Public Works Archives)

The NQ-3 sized borehole was angled at 60 degrees above the horizontal. The sandstone was intersected at an inclined depth of 26.56m (RL -22.5 m AHD). Water pressure testing (packer testing) was performed at six metre intervals. The borehole was terminated at an inclined depth of 65.33m (RL –56.1m AHD).

The major geological features intersected by borehole NS43 included:

A 1.67m no core zone. This was part of a three metre thick, extremely weathered, laminate band. This occurred at RL –27.5 to –30.5m AHD.

Low angle defects such as clay seams and parts occurred with defect spacings of 50 to 500mm from RL –23.3 to –36m AHD.

Intense low angle shearing and fracturing, with subvertical jointing and high rock mass permeability was evident between –33.4 to –36m AHD.

The geophysical investigation was extended to include surface to borehole tomographic imaging out from borehole NS43. This seismic work indicated that there was a sub horizontal zone of rock from RL – 30 to –36m AHD that had a lower seismic velocity than surrounding rock. A lower seismic velocity is indicative of fractured rock where major stress relief has occurred. The lower seismic velocity zone was correlated with the low angled shear features and high permeability zone in NS43 to represent a zone of stress relief beneath the paleochannel. The plot of the surface to borehole tomography is shown in Figure 4 below.

Subsequently a recommendation was made that the tunnel vertical alignment at Lane Cove River be lowered one tunnel diameter, so that the tunnel profile was below this feature. The lowered tunnel alignment provided a rock cover of about 11m.

Geological correlation between the three Lane Cove River boreholes (NS20, NS42 and NS43) was difficult due to the the 170m spacing between holes. However, a 3m thick siltstone unit was correlated between the three boreholes suggesting no significant vertical displacements in the geology beneath the river.

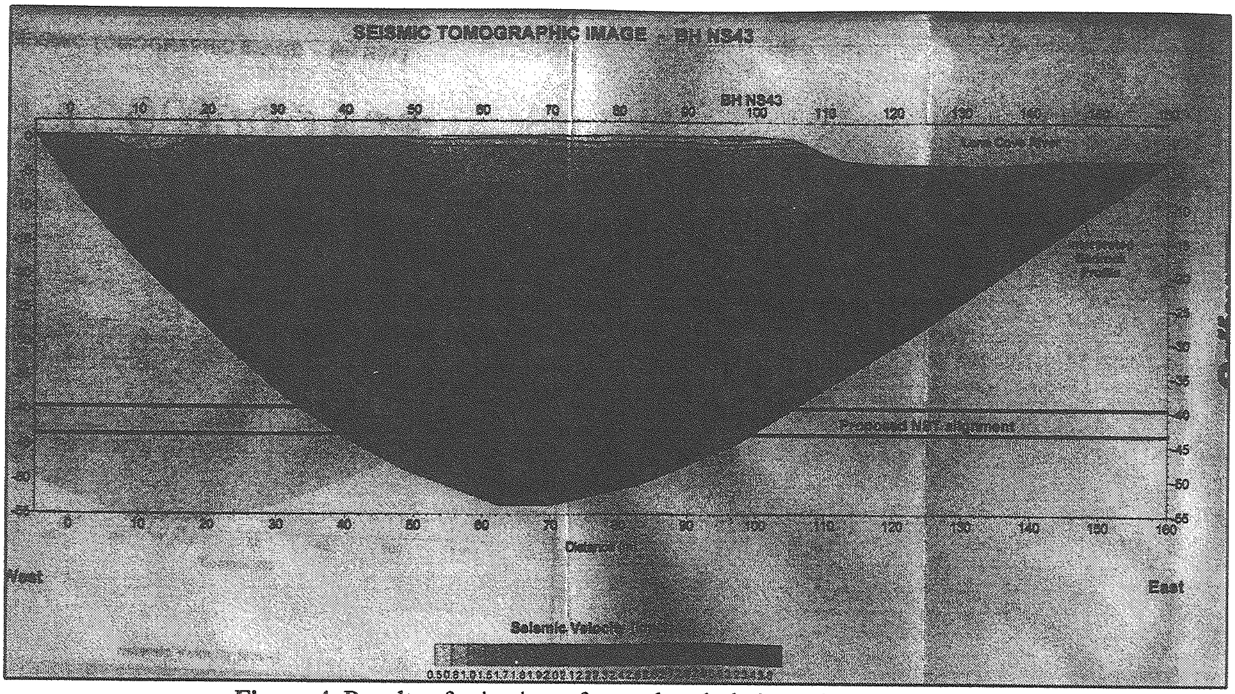

Figure 4 Results of seismic surface to borehole investigation for NS43

4 CONSTRUCTING THE TUNNEL

The Tunks Park to Lane Cove Drive of the NST was excavated using a 3.8 m diameter Wirth TBM. The majority of the tunnel was excavated through massive and cross bedded sandstone. Roof support was designed as rows of four rockbolts of between 1.8 and 2.4m length. The tunnel reinforcement was designed without a central rockbolt in the crown because the TBM's central beam did not allow the drilling of a vertical hole for rockbolt installation. Due to the occurrence of stress induced spalling in the roof and/or where shale interbeds were evident, steel mesh was installed prior to the four rockbolts.

The tunnel geology between boreholes NS20 and NS43 consisted of sandstone with interbeds of laminite and shale lenses that were found to be laterally quite variable. Subsequently, correlating the structure between the boreholes was quite difficult. The following figure (Figure 5) illustrates the geology encountered by the tunnel and possible correlations of features found in the boreholes.

At chainage 6445 m from Tunks Park and beneath the Lane Cove River water inflow caused problems with the removal of muck from the tunnel face. This was revealed to be emanating from a low angle shear zone (thrust fault). Rock cover at this point was approximately 40m and the tunnel was 120m from the deepest part of the paleochannel. The thrust fault was encountered beneath the channel of the Lane Cove River and not beneath the paleochannel, which had been the target of the geotechnical and geophysical investigation. This shear zone, up to one metre thick, manifested itself as a seam of clay and crushed rock. The shear zone rose up the sidewalls of the tunnel over a distance of approximately 20m, from the knee to the tunnel crown. At this point a large amount of spalling occurred in the shoulders and the crown. Significant water inflow still occurred. The unexpected tunnelling conditions resulted in the adoption of a probe and grout procedure being instigated to alleviate the water inflow problems and improve conditions for the TBM. Tunnel progress was impeded for two weeks. The tunnel support provided in this section included steel sets, steel straps, dowels and rockbolts. Timber blocking and timber cribbing were also required as the roof of the tunnel opened up to a span of five metres due to rock fallout.

Tunnelling conditions improved as the shear zone rose above the crown of the tunnel. However, the feature was assessed to trend parallel to the crown of the tunnel for approximately 30 metres. Significant water inflow occurred as rockbolt and dowel holes penetrated the feature.

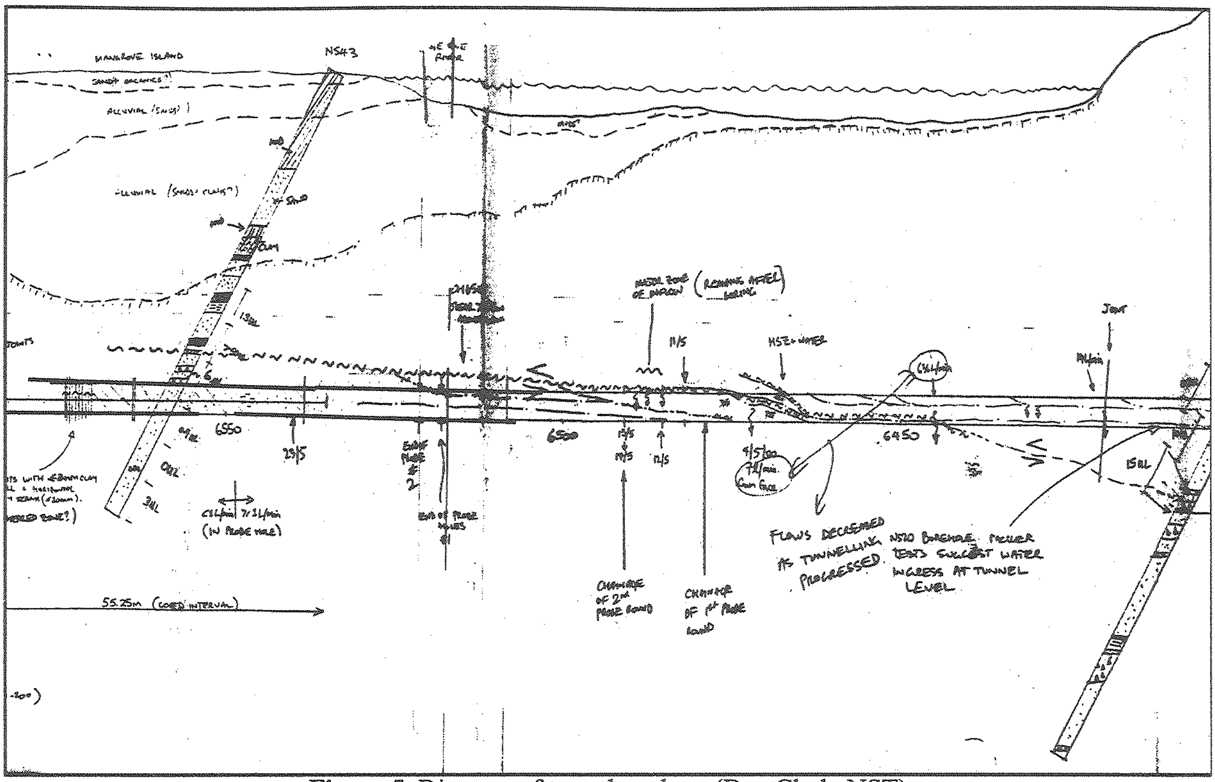

Figure 5 Diagram of tunnel geology (Dan Clark, NST)

5 CONCLUSION

The geotechnical investigation at Lane Cove River involved targeted drilling and seismic tomographic imaging that identified a stress relief feature below the paleochannel which resulted in a recommendation to lower the tunnel horizon. During tunnelling a low angled thrust fault feature was intersected some 120m from the paleochannel floor and this feature caused some tunnelling and support difficulties. While both boreholes NS20 and NS43 intersected shear features, which can be correlated to the tunnel feature, the shear zone was outside the area of the targeted investigations. The geophysics profile did illustrate the extent of the shear zone closer to the paleochannel but did not provide the complete picture of the feature intersecting the tunnel horizon further to the east.

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DEFORMATION BEHAVIOUR OF ROCK SLOPES ON PRE-EXISTING SHEAR SURFACES

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SUMMARY

The deformation behaviour of a rock slope prior to collapse is inherently related to the failure mechanism, strength of the defects controlling the failure mechanism, and in some cases is related to the rock mass strength. This paper presents results of analysis of a selection of rock slopes whose deformation behaviour has been influenced by defects that have experienced significant shearing. The rupture surfaces of these slope failures have experienced large deformation due to either regional folding, stress relief or previous instability. Examination is made of the relationship between normal effective stress acting on the rupture surface, rock mass dilation, over-riding of defect asperities and shearing or crushing of asperities. These factors are considered in the context of slope deformation behaviour and discussion is presented on their influence.

1 BACKGROUND AND NOMENCLATURE

The deformation behaviour of a rock mass prior to collapse is related to the failure mechanism and the strength of defects within the rock mass. Skempton and Hutchinson (1969) presented the terms "first-time slides" and "slides on pre-existing slip surfaces" to distinguish between slides in unsheared ground and slides on surfaces that have experienced significant shearing. Hutchinson (1988) suggested that slides on pre-existing slip surfaces can be further sub-divided into two distinct categories:

1 Failures on surfaces which have been pre-sheared due to geological processes other than landsliding; and
2 Failures on surfaces that have been pre-sheared by previous landsliding episodes (reactivated sliding).

Geological processes responsible for causing shear along defects include rebound/stress relief, regional faulting and folding under tectonic stresses and glaciotectonic influences. This paper discusses the nature of landslides on pre-existing shear surfaces and the influence this shearing can have on the pre-collapse behaviour of a rock slope.

The present industry practice of dealing with moving slopes generally involves establishment of a monitoring system and design of stabilisation measures if appropriate. The critical questions that need to be answered include:

- Will a moving slope collapse?
- If so, will the failure be sudden and brittle?
- What signs are expected prior to collapse? and
- How far and fast will the slope move before collapse?

This paper attempts to address some of these areas of uncertainty by examining a select group of well investigated and well monitored slopes.

Numerous terms have been adopted for describing the deformation behaviour of a rock mass including rebound, regressive/progressive movement, elastic deformation and creep. No two rock slopes will behave in the same manner in terms of deformation, due to the variability in rock mass characteristics. However, the deformation behaviour of a rock slope can be shown to exhibit certain typical stages, such as decreasing displacement rate, constant displacement rate or accelerating displacement rate.

The cases presented in this study are all natural slopes and no assessment of their elastic response to load variation has been made or measured. Following the initial elastic response, plastic deformation associated with stress relief may occur. In most cases this is observed as shearing along defects and may occur as steady movement over protracted periods or as episodic (stick-slip) movement. The rock mass response to stress relief and other external changes (such as increasing groundwater level) may be short-lived and rates of displacement may reduce with time or the response may be ongoing leading to accelerating rates of displacement and eventual collapse. The term creep has been adopted in this study to describe the time-dependent "slow, more or less continuous deformation or flow of natural and excavated slopes" (Emery, 1978). Creep is generally recognised to have three main divisions, as indicated in Figure 1. These terms will be used to describe the case studies presented in the following sections of this paper.

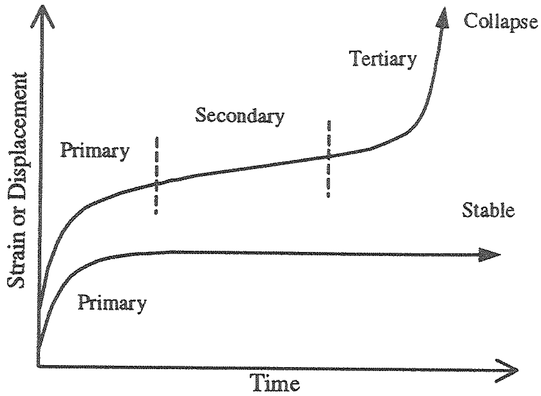

Figure 1 Diagrammatic representation of creep curves for moving slopes

2 DESCRIPTION OF CASE STUDY DATA

2.1 CLASSIFICATION

Ten cases of rock slides on previously sheared surfaces were examined in this study and their characteristics are summarised in Table 1. They were categorised according to whether they were first time slides on pre-sheared surfaces or reactivated slides. They were also sorted in terms of geological environment, failure mechanism and displacement-time behaviour, with comment provided in Table 1.

In all cases, the failure mechanism is broadly described as translational sliding, using Hutchinsons (1988) classification system. On a detailed level, cases represent wedge sliding or block sliding on planar or curved failure surfaces. Diagrammatic illustration of slide geometry is presented in Table 2. Information on the cases was obtained from published literature and unpublished reports. In all cases displacement monitoring commenced after the slope started moving. Comparison of data is not straightforward due to variations in length of monitoring period and intervals between readings. Eight of the slopes presented in the database were monitored using surface survey prisms.

The interpreted mechanism shown in Table 2 for Bumper Gully is based on surface geology and geomorphological mapping. There is some suggestion that the failure mechanism at Bumper Gully may be a compound slide involving some internal shearing. Further assessment of the Bumper Gully case will likely confirm the failure mechanism.

Comparison of total measured displacements is difficult due to variations in size of the slope failures. It is expected that, all other things being equal, a larger rock mass will exhibit greater displacement than a small rock mass. It is suggested that failure limits in terms of strain may be more relevant than total displacements. Therefore, for the purpose of comparison, total displacements for each case have been normalised against the down-slope length of the failed mass, to give an indication of strain of the rock mass. It is considered that the down-slope length is the most appropriate parameter for normalising, as vector deformation of these sliding failures is generally in a direction parallel to the slope face.

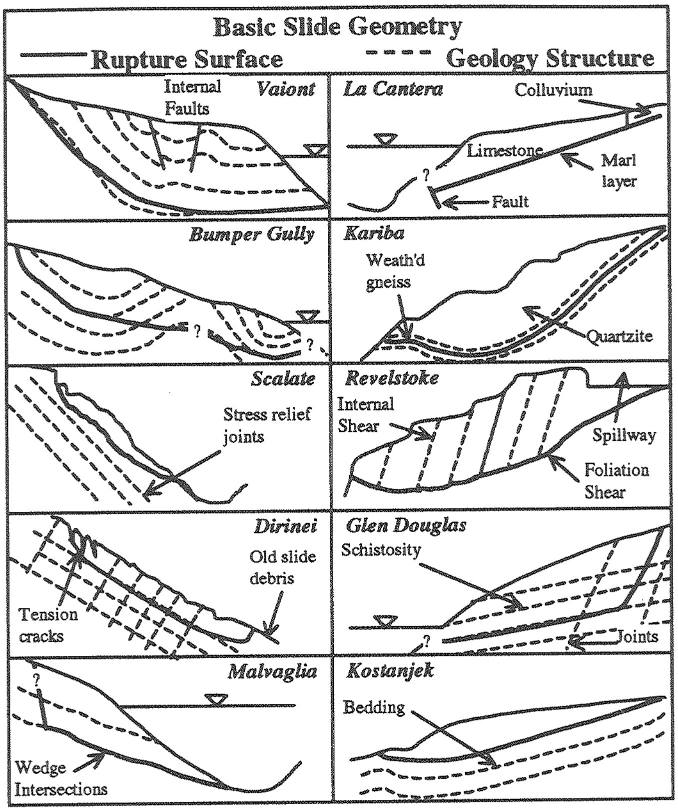

Table 2: Diagrammatic Illustration of Slide Geometry.

2.2 CASE STUDY COMMENTARY

Two of the ten cases examined (Vaiont and Scalate) progressed through to collapse. The remaining eight cases exhibited movement of varying extent and may be considered to have failed in terms of serviceability criteria but they did not show sudden catastrophic collapse.

The mechanism of previous shearing on the rupture surface of each of the slides was examined, and is indicated in Table 1. Causes of previous shearing include regional tectonic folding of strata, stress relief due to unloading and earlier periods of sliding. Five cases had previously sheared basal rupture surfaces associated with both regional folding and earlier sliding. Two cases had previously sheared rupture surfaces derived from both stress relief and earlier sliding activity. Two cases showed no signs of folding or stress relief but were reactivated landslides and one case was likely a first-time slide on a surface previously sheared by regional folding.

There were essentially two geological categories within the ten cases examined. Four cases involved sliding in sedimentary environments (limestone, marl, sandstone and siltstone), with rupture surfaces defined by bedding. The other six cases were from metamorphic terrains (schist, gneiss and quartzite) with rupture surfaces defined by jointing or schistosity. All failures were predominantly defect controlled.

3 ANALYSIS OF DEFORMATION BEHAVIOUR

3.1 HOW DO SLIDES ON PRE-EXISTING SHEARS BEHAVE?

3.1.1 GENERAL

All ten slides respond in different ways to load changes, such as groundwater rise or stabilisation works. Slides with a larger normal effective stress (on the rupture surface) generally show a more regular displacement-time response under periods of constant loading. Slides at low normal stress levels tend to show more erratic behaviour at constant loading. Many of the slides exhibited sensitivity to rainfall events, with larger slides generally showing a more delayed reaction time.

Geomorphological features on a number of the slides suggest significant movement. This movement combined with irregular rupture surfaces has resulted in disaggregation of slide masses. Vaiont and Kariba show disrupted slide masses overlying a rupture surface of complex geometry. Bumper Gully shows a highly to moderately disturbed slide mass, over what is likely to be an undulose and curved slide surface. La Cantera slide is described as highly fractured yet the rupture surface is understood to be relatively planar. The large total movement (50m+) of this slide may explain the high degree of fracturing.

Movement rates for nine of the slides were typically between 0.05 and 0.5mm/day. Vaiont had monitored rates generally above 1mm/day, with occasional peaks at about 20-40mm/day and a final displacement rate immediately prior to collapse assessed to be about 200mm/day. Distinct primary, secondary and sometimes tertiary creep stages are visible in the monitoring data for most cases, with Vaiont showing particularly distinct phases (refer to Figure 2). Many of the slides were stabilised so complete tertiary creep sequences are often not available and may in some cases not have developed.

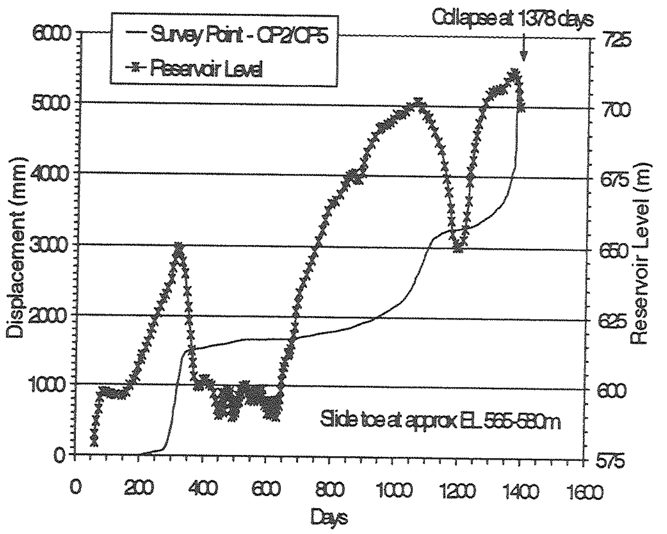

Figure 2: Displacement data for Vaiont slide showing relationship with reservoir level (modified from Hendron and Patton, 1985).

3.1.2 STICK-SLIP DISPLACEMENT BEHAVIOUR

Stick-slip motion is the term used to describe sequential episodes of rapid displacement followed by periods of low displacement rate. It is associated with the sudden over-riding of asperities on the rupture surface. In many slopes this type of behaviour may be observed in association with peaks in rainfall or snowmelt. In examination of these cases, focus was on stick-slip displacement behaviour during periods of relatively consistent stress levels. Consideration also needs to be given to intensity of monitoring. If readings are made at long intervals then a smoother displacement-time curve will be produced and stick-slip motion may be unrecorded. The displacement-time data for Revelstoke slide, presented in Figure 3, suggests some stick-slip type behaviour at very low normal stress levels. This slide was described as reactivated (with some regional folding) but the extent of previous shearing is unknown. Clay gouge and breccia are known to occur along at least part of the rupture surface and the normal stress level is very low. It is possible that the slide surface is not at residual strength and this combined with the presence of breccia may cause some brittle response to shearing (and hence a possibility for stick-slip motion).

Most other slides examined in this study show more uniform displacement-time behaviour at higher normal stress levels. This is attributed to the fact that destruction of asperities has taken place to a greater extent on these other slides. The data also suggests that slides that have undergone large shear displacements exhibit a reduced stick-slip tendency.

Cases such as Kostanjek, Vaiont and La Cantera involve sliding on clay coated rupture surfaces that have experienced significant shearing during previous sliding episodes and/or regional folding. The normal stress levels on these rupture surfaces were high and significant crushing and shearing of asperities is expected to have occurred. These three slides do not exhibit any apparent stick-slip tendencies and tend to show more gradual changes in displacement-time behaviour.

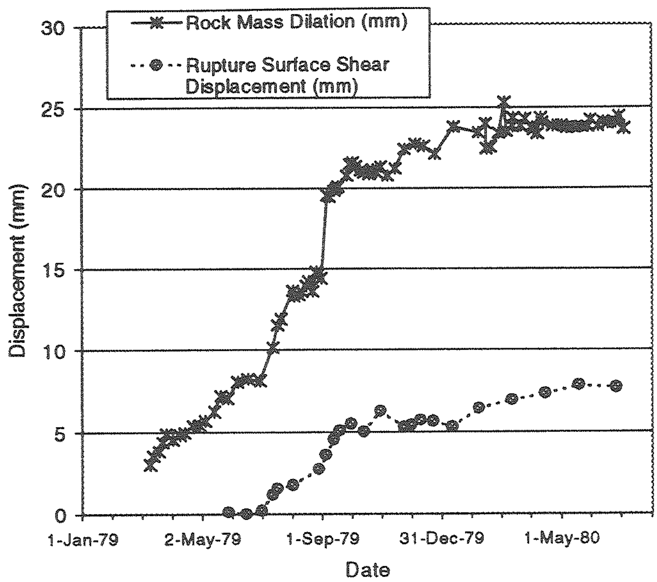

Figure 3: Displacement data for Revelstoke slide showing irregular (stick-slip) behaviour (Martin & Kaiser, 1984).

3.1.3 RADIAL DISPLACEMENT BEHAVIOUR

Radial displacement behaviour was observed in a number of the cases examined. In all cases, it appeared that extent of radial change in displacement behaviour was related to the extent of fracturing of the slide mass. In the case of Bumper Gully, the slide mass was recognised to be very disturbed, with average RQD's of the order of 15-50%. Highest rates and greatest magnitudes of movement were observed towards the centre and front of the slide. Rates and magnitudes of movement appear to show a general decrease with increasing radial distance towards the rear and sides of the slide. Similar patterns were observed in Dirinei and Malvaglia slides, which were described as highly fractured.

3.2 REASONS BEHIND DEFORMATION BEHAVIOUR

3.2.1 OVERVIEW

Translational slides may be considered analogous to laboratory direct shear tests on defects. The critical factors that determine the behaviour of a slide mass on a pre-sheared surface include:

1 the effective normal stress acting on the rupture surface;
2 rupture surface properties and geometry;
3 rock mass properties of the overlying slide mass; and
4 extent of previous shearing.

3.2.2 NORMAL STRESS

Analysis of test data for rock defects highlights the relationships between defect roughness, normal stress and shear strength. At all but very low normal stress levels, the shear strength of a defect decreases with progressive shear strain, until residual friction angle is attained. At low normal stresses dilation of the defect and over-riding of asperities is dominant while at high stress levels shearing and/or crushing of asperities is dominant. At high normal stress levels a more rapid reduction in shearing resistance is expected (ie: the strain required to reach residual strength is expected to be lower). This is schematically illustrated in Figure 4.

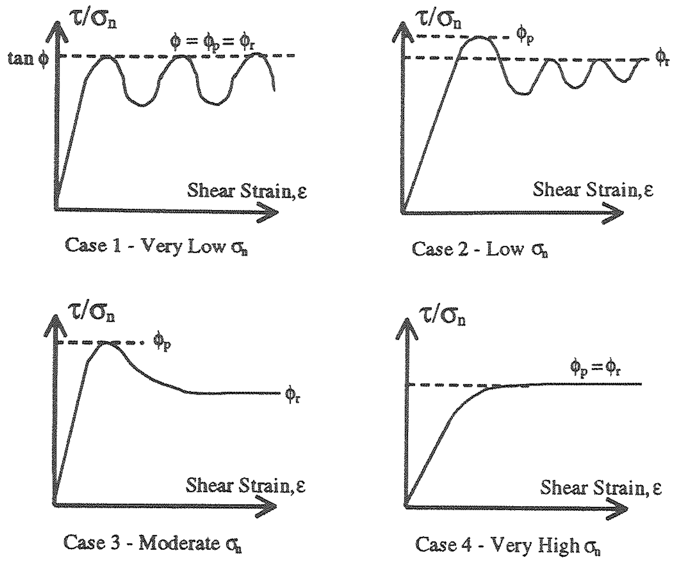

Figure 4: Diagrammatic stress-strain curves for defects at various levels of normal stress (Xu & de Freitas, 1990).

Shearing and/or crushing of asperities is likely to have taken place to some extent in all of the ten cases examined. However, some rupture surfaces are still described as rough and irregular, such as Dirinei and Scalate at low normal stress levels (following Case 2 type behaviour, Figure 4). Other slides have rupture surfaces coated with clay gouge or breccia, such as Vaiont, La Cantera and possibly Bumper Gully at relatively high normal stress levels (Case 3 or 4 type behaviour). Table 3 contains details of average normal stress levels, rupture surface infill and displacement characteristics for each of the ten slides examined in this study.

The case with the highest normal stress level (Vaiont) has long been recognised to have involved sliding on a rupture surface that was at residual strength. The extent of earlier shearing (from previous sliding and regional folding) and the low UCS of the clayey limestone layers contributed to the destruction of asperities and development of residual strength. It is therefore expected that no further strength loss on the rupture surface would have occurred and further displacement behaviour would have been essentially ductile (Case 4 type behaviour, Figure 4). The suddenness of the Vaiont failure may be attributable to brittle failure within the fractured slide mass rather than along the rupture surface. The geometry of the rupture surface was such that internal deformation was required for the slide to proceed. Hendron and Patton (1985) in fact showed that results of stability analysis were sensitive to values of internal friction angle. They also suggested that there was strong three-dimensional control on the slide, which may have also contributed to the brittle collapse.

3.2.3 RUPTURE SURFACE PROPERTIES AND GEOMETRY

A basic friction angle was determined for each rupture surface by examination of direct shear test data (on smooth defects) in published literature (Einstein and Dowding, 1989). The basic friction angle is primarily a function of the rock type but is also sensitive to normal stress levels. Where laboratory direct shear test results on infill material was available these results were adopted for basic friction angle where appropriate.

Many slides in this database have rupture surfaces with large-scale irregularities that affect the overall stability of the slope. An assessment has been made of the relevant field scale asperity from descriptions of the rupture surface geometry or by analysis of the changes in vector displacement direction. This asperity roughness is considered as a dilation angle (i) which adds to the overall frictional resistance, and is presented in Table 1. In the case of large slides with complex rupture surfaces, the asperities seen at laboratory scale testing are of little relevance. McMahon (1985) has suggested that the relevant asperities are those that have a wavelength measured over 2% of the length of the rupture surface. In many cases this value served as a useful guide for assessment of dilation angle. Some behaviour differences were noticed from the data when comparing slides on undulose or irregular surfaces with those on relatively planar surfaces. It was assessed that degree of break-up of the slide mass is significantly influenced by the irregularity of the rupture surface. Vaiont, Dirinei, Malvaglia, Kariba, Glen Douglas and Bumper Gully slides all show significant rock mass disaggregation. These slides all occur on rupture surfaces with higher dilation angle values.

Slide	Avge σ'_n (MPa)	Infill	General Displacement-Time Behaviour
Vaiont	3.4	Clay	Smooth primary-tertiary sequences
Malvaglia	1.1	Rock	Long-term linear
Kostanjek	0.9	Clay	Not clear - likely long-term linear
Kariba	0.8	Rock	Linear but variable with load change
Bumper Gully	0.8	Gouge/ Breccia	Smooth primary creep after initial quarrying and lake filling
La Cantera	0.8	Clay	Linear with jump due to stabilisation works
Glen Douglas	0.6	Rock	Long-term linear
Revelstoke	0.4	Clay	Stick-slip irregular
Dirinei	0.3	Rock	Slightly irregular
Scalate	0.2	Rock	Stick-slip irregular

Table 3: Normal effective stress, rupture surface infill and general displacement behaviour.

3.2.4 ROCK MASS PROPERTIES

The extent of disaggregation of a slide mass is seen from the data to be not only a function of rupture surface geometry, but is also influenced by failure mechanism, amount of previous sliding and the rock mass strength. The extent of shearing along the rupture surface as a percentage of rock mass dilation was measured at Revelstoke, and is illustrated in Figure 3. It was observed that basal shear accounted for about 40% of the total observed deformation with rock mass dilation accounting for the remainder. It was also observed in this particular case that rock mass dilation preceded shearing along the rupture surface. It is suggested that dilation may have been required in order to over-ride asperities. The internal friction angle (and hence the rock mass strength) of the slide mass may be a significant factor in the stability of the slope, particularly for slides on complex rupture surfaces (as illustrated by Vaiont).

The normal stress level at which crushing of asperities commences (and over-riding ceases) depends on the strength of the rock. It is expected (although not readily observable from the data) that shearing and crushing along the rupture surface will commence at lower stress levels for sliding on clay marl than for fresh limestone. Similarly, slides in weathered gneiss may be expected to show yielding and development of residual strength at lower normal stress levels than slides in fresh gneiss.

3.2.5 EXTENT OF PREVIOUS SHEARING

The extent of previous shearing has been shown to be influential in the deformation behaviour of these slides. Based on the amount of shear displacement many of the rupture surfaces examined are at or close to residual strength. Slides that have undergone the greatest total displacement have a tendency for more regular deformation and the degree of stick-slip type behaviour is reduced on these slides. This is again related to progressive destruction of asperities on the rupture surface and development of residual strength.

La Cantera slide illustrates that disaggregation of the slide mass can also occur on relatively planar rupture surfaces. Disaggregation in this case is likely due to the large displacement this slide has experienced.

The type of previous shearing (ie: stress relief, sliding or regional tectonic folding) does not appear to influence the deformation behaviour in any significant manner. It is assessed that the extent of shearing and the normal stress levels at which that shearing occurred are of more influence.

4 CONCLUSIONS

The displacement behaviour of a slide on a pre-sheared surface has been shown to be predominantly controlled by:

- the effective normal stress acting on the rupture surface;
- rupture surface properties and geometry;
- extent of previous shearing; and
- rock mass properties of the overlying slide mass.

The first three factors listed above influence the extent of strength reduction from peak towards residual strength and hence the degree of brittleness remaining on the rupture surface. The fourth factor influences the ability of the slide mass to dilate and over-ride asperities on the rupture surface. The development of residual strength on a rupture surface with progressive shear displacement suggests that the likelihood of sudden brittle failure is reduced. The ten cases examined illustrate that stick-slip type motion is reduced with increased displacement. However, Vaiont illustrates that other factors (such as internal deformation) need to be considered before assessment of brittleness can be made.

In contrast to first-time slides on rupture surfaces that have not experienced shear displacement, slides on pre-sheared surfaces are expected to have a reduced stick-slip tendency. Slides on pre-sheared surfaces often show a high degree of disaggregation and following from this they often show a tendency for radial displacement behaviour.

5 ACKNOWLEDGEMENTS

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Case Name (Origin)[1]	Geology	Avge σ'_n[2] (MPa)	Rupture Surface Description	Approx UCS Category[3]	Approx ϕ_b (deg)	Approx i[4] (deg)	Total Displ (mm)	Displacement/ Slope Length (%)	Comments on Displacement Behaviour
Vaiont (R, F)	Lime-stone	3.4	Dip = 24 deg (avge). Defined by bedding partings in monoclinal fold. Rupture surface in clay units within marly limestone. Direct shears: ϕ_r=8-10deg. Small monoclinal folds perp to direction of shearing giving i = 9-10deg.	R2	10	10	5340+ (measured over 42 months)	0.39+ (measured)	Reactivation due to reservoir filling. Planar/curved slide with some rotational movement influenced by internal deformation of slide mass (analysis sensitive to ϕ chosen for internal deformation). Displ closely associated with reservoir level/rain. Three sequences of primary to tertiary creep behaviour. Expect some strain hardening in fractured slide mass
Bumper Gully (R, F)	Sandstone /siltstone	0.8	Dip = 24 deg. Slide base geometry affected by synclines/anticlines – follows bedding partings in most of length with joints controlling remainder of length.	R2	22	6	3800+ (measured over 480 months)	0.78+ (measured)	Reactivation due to reservoir filling. Complex translational and rotational slide. Numerous small surficial slides defined by local folds on surface of larger slide. Highly-moderately disturbed slide mass (RQD=15-50%). Variation in displ. vector behaviour suggests i=6-10deg.
Scalate (SR, R)	Schistose gneiss (weath'd)	0.2	Wedge sliding on joint sets (incl. stress relief joints) – rough & irregular – with intersection plunge=40deg. Expect some staining of joint surfaces.	R3	25	10	290+ (measured over 9 months)	0.36+ (measured)	Stick-slip behaviour seen in early displacement monitoring. Seasonal rainfall has only minor impact on slide behaviour. Slide exhibited primary, secondary and tertiary creep phases before collapse. Vector displ. plunge = 40deg.
Dirinei (SR, R)	Schistose gneiss (weath'd)	0.3	Planar sliding on 33deg dip stress relief joints – rough & irregular - with minor crushing on slide surface.	R3	25	8	1900+ (over 34 months) 5000+ (inferred from geomorph)	5.0+ (inferred)	Reactivation associated with fluvial oversteepening and rain. Sensitive to high intensity rain events. Slide shows overall primary creep trend. Radial displ behaviour observed. Vector trend parallel to rupture surface.
Malvaglia (R)	Schistose gneiss	1.1	Wedge sliding on joint sets and schistosity. No infill. Dip of 21+ deg on rupture surface.	R3	25	6	130 (measured over 65 months)	0.09+ (measured)	Reactivation due to reservoir filling. Radial decrease in displacement rates suggesting dilatant (fractured) rock mass, therefore possibly large historical displacement. Sensitive to seasonal rainfall. Approx. linear displ-time behaviour.
La Cantera (R)	Lime-stone/ dolomite/c lay marl	0.8	Planar slide in 5m thick clay marl layer (following bedding) dipping at 18 deg. Illite/kaolinite coating on rupture surface with ϕ_r=17-18deg. Distinct shear surface in inclinometer readings.	R2	18	0	80+ (over 10 months) 50000+ (inferred from geomorphology)	11.0+ (inferred)	Geomorphological features suggest previous sliding of the order of 50 metres. Rupture surface at residual. Slide mass is highly fractured. Expect low dilation angle on rupture surface due to shearing in clay marl layers. Slide shows near-linear displ-time behaviour with no signs of stick-slip.
Kariba (R, F)	Gneiss/ quartzite	0.8	Rotational slide along rupture surface defined by syncline along weath'd gneiss/quartzite contact. Rupture surface dips at 35-40deg out of slope at head and 20 deg into slope at toe.	R2	23	8	690+ (measured over 432 months)	0.27+ (measured)	Slide reactivated by dam works and reservoir filling. Slide mass highly fractured and sensitive to rain/spray from spillways. Previous sliding known to be significant with numerous tension features at head of slide. Internal deformation of slide mass required for movement of slide.
Revelstoke (R, F)	Quartzite/ gneiss	0.4	Translational sliding along 23deg dip foliation shear. Direct shear ϕ_r=18deg on 20-50mm clay gouge. Undulose surface due to regional folding. Clay gouge & breccia on rupture surface.	R2	18	6	34+ (measured over 18 months)	0.04+ (measured)	Slide mass has numerous steeply dipping internal shears and some fracturing. Dilation of slide mass (measured using extensometers) observed to occur before shearing along rupture surface. Displacement sensitive to rainfall. Rupture surface dilation determined by regional folding.
Glen Douglas (R, F)	Schist/ phyllite (weath'd)	0.6	Planar sliding on schistosity (up to 30 deg dip) with back release on joint set. Slide located within regional fold. Crenulation on schistosity gives large difference between peak and residual ϕ.	R2	21	8	620+ (measured over 228 months)	0.17+ (measured)	Slide mass quite broken suggesting large prehistoric movement. Crenulation on schistosity + regional folding add to frictional resistance. Dilatant rock mass, therefore likely to be more influenced by larger geometrical features on rupture surface. Approx linear long-term displ-time behaviour.
Kostanjek (F)	Dolomitic breccia/ limestone/ marl	0.9	Planar sliding on three levels. Main surface in silty schists and marls. Upper surfaces within clayey marl. Dip=10deg. ϕ_r=9deg on main rupture surface	R1	9	2	6500+ (measured over 372 months)	0.47+ (measured)	Slide activated by quarrying, blasting and high groundwater pressures. Peak shear strength measured as 20-28 deg therefore large drop to residual strength. Likely 1st time slide but previous shearing due to regional folding.

Table 1: Data on slides occurring on previously sheared rupture surfaces.

(1) R = reactivated landslide, F = folding has affected slide surface, SR = stress relief has affected slide surface.
(2) Average normal effective stress acting on rupture surface.
(3) Unconfined compressive strength category as per ISRM.
(4) Assessed relevant field scale dilation angle.

DISCUSSION "LANDSLIDE RISK MANAGEMENT CONCEPTS AND GUIDELINES"

(AUSTRALIAN GEOMECHANICS, VOLUME 35, NO 1, MARCH 2000, PP 49 TO 52)

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1 INTRODUCTION

Coffey Geosciences Pty Ltd has many years of experience of landslide risk assessment and management and continue to do a great deal of work in this area. The paper entitled "Landslide risk management concepts and guidelines" (Landslide Paper) published in the last edition of Australian Geomechanics will have implications for us, other practitioners, clients, owners and regulators and those affected by landslide risk. The Landslide Paper defines landslides as "the movement of a mass of rock, debris or earth down a slope". This broad definition, which includes falls, topples, slides, flows and spreads from both natural and artificial slopes, means that many geotechnical professionals get involved in slope risk management at some time.

We are currently in the process of preparing notes for internal distribution on landslide risk management and the Landslide Paper and, in these notes, we intend including examples of how they can be applied. During this process it has become clear that some of our experienced practitioners have concerns about some aspects of the Landslide Paper and how it might be interpreted in practice.

The purpose of this letter is to contribute to a constructive debate by highlighting and discussing the strengths of the Landslide Paper, raising and discussing areas of concern and summarising what we think are important issues. We have also included four example case histories which show how short simple reports can be consistent with risk management principles and the concepts and guidelines in the Landslide Paper.

2 SOME OF THE STRENGTHS OF THE LANDSLIDE PAPER

The following comments highlight some of the strengths of the Landslide Paper.

2.1 INTRODUCES RISK MANAGEMENT PRINCIPLES

We see the introduction of risk management principles as the major strength of the Landslide Paper. Applied to slopes the principles can be interpreted as answering the following questions:

- What are the issues and who cares? (SCOPE DEFINITION).
- What might happen? (HAZARD IDENTIFICATION).
- How likely is it? (LIKELIHOOD).
- What damage or injury might result? (CONSEQUENCE).
- How important is it? (RISK EVALUATION).
- What can be done about it? (RISK TREATMENT).

In our experience applying risk management principles encourages people to plan, work and report more effectively because they understand the wider context of the job. In particular, this often involves giving more attention to consequences and treatment options than has sometimes happened in the past. All the examples attached are consistent with risk management principles. The risk management steps are highlighted against the relevant paragraphs in Example 1.

2.2 RAISES THE ISSUE OF LOSS OF LIFE

The Landslide Paper makes it clear that risk of loss of life should be considered. Recognising potential hazards, which can cause loss of life, is the first step. The record of deaths in Australia shows that the following hazards/situations can lead to fatality:

- Sheltering near very steep cliffs, overhangs (or even caves) during rain.
- Dislodging boulders while climbing.
- Fast debris flows from steep natural slopes.
- Fast debris flows from loose fills.
- Slower debris flows or slides which trap people in buildings or cars and bury them.
- Sudden failure of temporary cut slopes (eg, slide of trench or basement excavation).

Slow moving earth or debris slides seldom lead to death because people have time to get out of the way.

Example 2 is a situation where a significant risk of loss of life was recognised and immediate action was taken to reduce the risk.

2.3 PROVIDES USEFUL REFERENCE MATERIAL

The inclusion of the landslide risk management terms in Appendix A and other landslide terms in Appendix B will encourage a more uniform approach. Other appendices provide useful references and background material.

3 CONCERNS ABOUT THE LANDSLIDE PAPER

The following comments highlight some concerns on the Landslide Paper raised by experienced practitioners. Although some of the concerns are related to emphasis, perception, interpretation and presentation we feel there is value in raising and discussing the issues as they may also be of concern to other practitioners.

3.1 LACK OF EMPHASIS ON GEOLOGY

In our opinion there is a lack of emphasis on the geological and geomorphological skills needed to understand landslide hazards. In the majority of cases, the most difficult and time consuming issue in landslide risk management is recognising, understanding and explaining the hazards. In our experience the best work on understanding landslide risk is usually carried out by engineering geologists with knowledge and experience of landslides and an understanding of risk management principles. Such geologists usually have a good understanding of uncertainty and are less likely to quantify unrealistically than people with little geological knowledge. All too often, in our experience, people start arguing about numbers (eg. factor of safety or probability of failure) when a better contribution to the landslide risk management process would be to put more effort into understanding and explaining the particular geological processes at the site in question.

3.2 OVER-EMPHASIS ON QUANTIFICATION

Where realistic, some form of quantification based on understanding the hazard helps decision making. The attached examples, although simple are partly quantitative. In our opinion the level of risk quantification implied in the Landslide Paper is inappropriate for the majority of jobs. In conjunction with the lack of emphasis on geology discussed above there is a danger that the Landslide Paper may give the impression that a quantified risk assessment, however unrealistic, is better than the judgement of an experienced practitioner who has the knowledge and wisdom to understand the limitations of quantification in that particular situation. The quality of a landslide risk assessment is related to the extent that the hazards are recognised, understood and explained which is not necessarily related to the extent to which they are quantified. The level of quantification shown in the examples will be adequate in many cases.

The Landslide Paper recommends that risk for loss of life should be quantified. In our experience that is not always necessary. Sometimes when risk of loss of life is identified as an issue in an initial study it is possible to take immediate steps to reduce the risk without the need for further assessment. In some situations where uncertainty remains, the client/owner/regulator or those affected may be happy to make a decision on the basis of the initial qualitative or semi-quantitative assessment. In other situations, where quantifying the likelihood of loss of life involves a great deal of uncertainty and subjective judgement, recognising and explaining the uncertainty and making cautious judgements may be more helpful to the decision making process than unrealistic attempts at quantification.
As explained in the Landslide Paper there can be many direct and indirect consequences of a landslide. Fully understanding and quantifying the consequences of a landslide will seldom, if ever be achievable. Often, we will not have the knowledge or expertise to estimate some of the direct costs, let alone the consequential costs. We are usually engaged by the client/owner/regulator because of our knowledge of the hazard. Our major role is to understand and

explain the consequences. The client/owner/regulator may wish to attempt to quantify the cost of some of the direct consequences but in our experience, in many cases they will be willing to make value judgements on risk acceptability and treatment options on the basis of the assessor's description of possible consequences.

3.3 THE LANDSLIDE PAPER IS TOO PRESCRIPTIVE

There is a recommendation in the Landslide Paper to "follow the guidelines" and encouragement to use particular terms and a complex risk matrix. The first sentence of Section 3.5.3 of the Landslide Paper (page 62) states that "risk for loss of life should be quantified..." and there are other similar statements elsewhere. As explained above, there are many circumstances when it may be reasonable to make decisions about risk without quantifying the risk of loss of life. In our opinion each case should be judged on it merits. Unrealistic attempts to quantify the risk of loss of life with inadequate knowledge may be quite misleading.

In our opinion the Landslide Paper is too prescriptive and we do not agree with all the recommendations. We believe that the best way to advance the practice of landslide risk management in Australia is to encourage a constructive debate on the Landslide Paper and the publication of case histories.

3.4 THE APPLICABILITY OF THE EXAMPLE TERMS AND RISK MATRIX IN APPENDIX G

Appendix G provides examples of some qualitative terms and shows how they may be linked to a risk matrix. The text of the Landslide Paper recommends the terms in Appendix G and the frequent cross-references to this appendix lend it authority.

We have reservations about Appendix G because it is an example that represents a particular situation. It will not be applicable to many jobs because:

- It includes specific description of consequences which may not be relevant to the job.
- It includes very specific risk level implications that are unlikely to be generally applicable. The amount of investigation required, and cost of treatment is not necessarily related to the level of risk. For example, as explained on page 64 of the Landslide Paper, if the high risk is associated with a single large boulder it may be easy to remove the boulder and reduce the risk without further investigation.
- The use of dual terms in the qualitative risk analysis matrix effectively creates eight risk levels (without including low risk which does not appear on the matrix). This complexity is unlikely to be required.

Users of Appendix G also need to recognise that it is not possible to make a qualitative judgement as to likelihood, and to use this judgement to assign a probability. The first step in the process is to make a quantitative assessment of the probability of landsliding, and then assign a qualitative likelihood description based on this information.

More fundamentally, we caution about the over use of risk matrices. As shown in the attached examples risk matrices are not essential to the risk assessment and management process. Trying to fit a relatively simple job to a pre-defined matrix can result in unnecessarily complex and clumsy reporting. At worst it could encourage an unthinking cookbook approach instead of understanding the particular site and the clients concerns and clear and simple reporting.

Risk matrices are of value where many risks have to be compared. In such cases, it can be useful to develop risk analysis matrices in conjunction with others (clients/owners/regulators/other practitioners) to help rank risks, set priorities and develop a uniform approach to decision making. If we do contribute to the development of a risk matrix we have to remember that consequence ranking and risk ranking are value judgements. We have to try to avoid the tendency (identified in a recent British government report on risk) for "experts throughout the decision making process to substitute their own value judgements for those of the stakeholders".

We recognise that the above discussion points out the limitations of Appendix G without offering an alternative. We encourage the profession to publish examples or partial examples of case histories where risk matrices have been used to assist decision making. Examples which have a track record of being accepted by clients/owners/regulators/those affected by the risk and, if quantified, explain how indicative probability was assessed will be particularly useful references.

4 SUMMARY

The quality of a landslide risk assessment has always been and remains primarily dependent on the extent that the hazards are recognised and understood. Where dealing with natural slopes this requires good geological and geomorphological skills and knowledge of landslide types and landslide behaviour, experience and judgement. Developing a sound geotechnical model and understanding failure mechanisms are of prime importance. This means that the quality of the assessment is usually related to the quality and experience of the people carrying out the work and the time and effort spent trying to understand the hazard.

Realistic quantification based on knowledge, insight and understanding of the particular site, the region and the hazards in question with open acknowledgement of the uncertainty and judgement involved helps the decision making progress. Unrealistic quantification based on poor understanding of the hazard will be meaningless and misleading, will not contribute to good decision-making and should be avoided.

The Landslip Paper appears to be primarily addressed to skilled geotechnical engineers and engineering geologists who have the experience and wisdom to apply the principles outlined. It is not in our view in a form that is readily assimilated by other potential audience categories, such as local government, developers, owners and the like. Once this debate has proceeded to a resolution it may be appropriate to prepare a separate document to clearly outline the recommended processes to clients.

The strengths of the Landslide Paper discussed above, in particular the application of risk management principles should help improve slope risk management. We look forward to hearing other opinions on the Landslide Paper and landslide risk management in Australia.

example reports

The following examples show how short, simple reports can be consistent with risk management principles and the Landslide Paper. It is not necessary to attach appendices to these reports although it may have been useful to include sketch maps and sections. Although the examples are simple they may be regarded as partly quantitative as they provide some indication of the likelihood of the hazard. Fuller attempts at quantification (sometimes using spreadsheets to manipulate assessed probabilities) although useful in some circumstances will not be required for the majority of jobs.

EXAMPLE 1 – ROCK FALLS
(COMMENT: This example illustrates each step involved in risk management.)

Mr Smith parks his car in an old quarry. Recently some rocks fell from the quarry face and landed near the car. Mr Smith asked us to advise him on the rock fall risk to his car.	(SCOPE DEFINITION)
The quarry face is 10 m high and slopes at 70°. The face consists of slightly weathered high strength sandstone. The bedding dips at 80° into the face and there are short joints orthogonal to bedding. No persistent defects were observed that could contribute to overall slope failure but toppling of individual blocks up to 0.5 m across is possible and has occurred in the past.	(HAZARD IDENTIFICATION)
The quarry face is about 30 years old. Rock falls on the quarry floor and discussion with local residents indicate that there are on average 3 to 10 rock falls a year (between 0.1 m and 0.5 m across). About 1 in 3 of the rock falls reach the car park (ie, 1 to 3 per year).	(LIKELIHOOD– QUANTIFIED)
If a rock fall reaches the car park while the car is parked there may be some damage to the front of car (eg, a broken headlight or a dent to the bonnet). The car is there about 50% of the time.	(CONSEQUENCES)
Mr Smith did not want to take the risk of having his car damaged. We	(RISK EVALUATION)

advised him that a fence would stop the rock falls reaching the car or he could park elsewhere. Barring down some obvious loose rocks would reduce (but not eliminate) the likelihood of rock falls. (RISK TREATMENT reducing the consequence, avoiding the risk, reducing the likelihood)

Mr Smith chose to park elsewhere. (CLIENT DECISION)

EXAMPLE 2 – temporary slopes
(COMMENT: This is a short letter but it includes all the steps discussed in the previous example.)

Jones Construction Pty Ltd has cut a vertical slope 3 m high in stiff clay at a building site in the city. There is a footpath and buried services at the top of the slope. The council asked us to advise them on the risk to the footpath and services.

We advised the council that (based on similar experience elsewhere) sudden failure of the slope could occur at anytime which could cause the footpath to collapse and disrupt the services. We also pointed out that the slope collapse could injure or kill anybody working below the slope.

The council immediately passed our advice on to Jones Construction Pty Ltd. Both parties agreed that the risk was unacceptable and temporary support for the slope was installed.

EXAMPLE 3 – ROAD CUTTINGS ON MAJOR HIGHWAY (PARTIAL EXAMPLE)
(COMMENT: For a single cut slope use of a risk matrix is probably unnecessary. However if there are many cuts the matrix will help rank risks and establish priorities for slope treatment.)

Table 1 is an example of a simple qualitative risk matrix. In this table there are four divisions of relative likelihood, three divisions of relative consequence and six levels of relative risk. As the main purpose of the matrix is to rank risk in order to help set priorities for risk treatment we have used numbers to describe the risk level. Injury and loss of life issues were also considered (not shown here).

LIKELIHOOD	**CONSEQUENCES**		
	Severe	**Moderate**	**Minor**
High	1	2	3
Medium	2	3	4
Low	3	4	5
Very Low	4	5	6
Numbers (1 to 6) are the risk levels			

TABLE 1 Qualitative risk assessment matrix

An example of a job involving landslide risk to a major highway using the Table 1 risk matrix is summarised on Table 2. In this example the client may decide to take no action for Cutting 14.

	WHAT MIGHT HAPPEN	**HOW LIKELY**	**WHAT DAMAGE**	**RISK RANKING**
A	**Small debris flow** Rock fragments and soil may be washed off slope during intense rain. Individual flows generally less than 1 m^3.	**High** Likely to occur several times in first winter after construction and during locally intense rain in subsequent years.	**Minor** Debris may block gutter. Soil, small rock fragments (generally less than 100 mm across) may reach carriageway	3
B	**Small rock fall** Toppling and wedge failures up to 3 m^3 on left hand side.	**Medium** Assuming good quality construction may occur on average every 3 to 10 years (based on performance of	**Minor** Most debris likely to be retained in the gutter. Some rock fragments may reach carriageway.	4

		existing cuts).		
C	**Large rock fall** Wedge failure up to 10 m^3 left hand side where persistent defects.	**Low** May occur due to stress release or mechanical weathering (eg, root jacking). Annual probability judged to be less than 0.1 (1 in 100).	**Moderate** Some debris likely to reach highway and may block one lane. Damage to vehicle could result in vehicles running into debris or debris hits vehicle.	4
D	**Overall collapse** Overall failure of whole slope on combination of persistent defects.	**Very low** No defects or combination of defects recognised that could contribute to such a failure. Annual probability judged to be less than 0.0001 (1 in 10000)	**Severe** Debris could block entire highway. Vehicles could be damaged and traffic disrupted for several days.	4

Table 2 Risk assessment for cutting 14

EXAMPLE 4 – HAZARD ZONING

(COMMENTS: The example uses some relative hazard terms, which were used in an existing hazard zoning scheme and development control system. The basis of the ranking terms was explained in the hazard zoning scheme and the council; other practitioners and residents had accepted their use.)

Mrs Brown wants to extend her house. The house is located on a steep slope (escarpment) underlain by weathered basalt of Tertiary age. Previous hazard zoning in the region has been used as a basis for development controls. The basalt escarpment has been zoned as a high hazard area. Existing landslides (mainly debris slides and slow debris flows in colluvium) affect about 10% of the basalt escarpment in the region and it was judged that new landslides could occur on some of the unfailed slopes. The local council usually only allows further development in the area if a site specific assessment results in a downgrading of the hazard rating.

There are 20 lots on the basalt escarpment where Mrs Brown lives. We observed recent or active landslides on six of these lots including one of the lots next to Mrs Brown's house. Local records and aerial photographs indicate that some of the landslides have moved several times in the previous 50 years. An investigation report by another consultant indicated that soil strength material occurred to a depth of at least 6 m at one of the landslides. The borelogs did not distinguish between colluvium, residual soil and extremely weathered basalt. The landslides occur on slopes of 15° and 22°.

The overall slope on Mrs Brown's property is about 25°. Site observations (under the house and in the cutting for the garage) revealed that slightly weathered basalt underlies residual soil at shallow depth. Behind the house (where the extension is planned) there were low outcrops of basalt. Long grass, shrubs and the effects of landscaping largely hid the outcrops. The outcrops were undisturbed. No open joints or infilled seams were observed.

On the basis of the site specific assessment we advised the council that it was very unlikely that a landslide would affect Mrs Brown's property. In accordance with criteria agreed with the council this resulted in a downgrading of the hazard rating to low. The council allowed Mrs Brown to extend her house.

(DISCUSSION: *The example shows that a purely statistical approach based on the occurrence of landslides on the basalt escarpment would have been misleading. The landslide hazard on Mrs Brown's property is very much less than the average hazard elsewhere on the escarpment. While it was not possible to quantify the probability of a landslide on Mrs Brown's property it is clearly very low.*

Ackowledgments

A number of senior geotechnical engineers and engineering geologists within Coffey have provided valuable input to this paper. Their contribution is gratefully acknowledged.

LANDSLIDE RISK MANAGEMENT CONCEPTS AND GUIDELINES AUSTRALIAN GEOMECHANICS VOLUME 35, No. 1 RESPONSE TO DISCUSSION BY G. POWELL

Bruce Walker,
Chair, AGS Sub-Committee on Landslide Risk Management

1 INTRODUCTION

The discussion was forwarded to me, as Chairman of the AGS Sub-Committee by the editor of Australian Geomechanics. The members of the sub-committee were forwarded a copy of the discussion paper and asked to contribute to this response if they wished. This response has been prepared from comments received up to 25th August 2000.

2 RESPONSE TO GENERAL ISSUES

The discussion by G. Powell is largely supportive of the Guidelines, and the use of risk management concepts. We thank him for his contribution and elaborating on some issues which were debated within the Sub-Committee. There are some differences of emphasis, but that is normal in risk management matters, which are new to many, and it is a developing art.

We believe that it is worth elaborating on the process which led to the publication of the Guidelines. The Guidelines were developed over a period of 8 years by the Sub-Committee which consisted of four engineering geologists, and four geotechnical engineers, all of whom had extensive slopes experience. There were several drafts of the Guidelines, and they developed as the procedures in risk management in landsliding and other areas matured over that time.

In 1997, three members of the Sub-Committee attended a workshop on landslide risk assessment, at which 25 of the worlds leading practitioners and academics were present. This provided valuable input to the process of developing the Guidelines.

Prior to publication, the final draft of the Guidelines was sent to the AGS National Committee, who asked for review from practitioners and academics in Australia. Responses from those reviews were incorporated into the final Guidelines as considered appropriate by the Sub-Committee.

It is intended that worked examples using the procedures outlined in the Guidelines be presented in workshops to be held in the major capital cities, and published in Australian Geomechanics. These should assist those who find the concepts new, and possibly a little daunting.

3 RESPONSE TO SOME SPECIFIC ISSUES

We do not intend to respond item by item to the issues raised by G. Powell, since many are matters of relatively minor detail, and in some cases, only reiterate what is in fact already in the Guidelines.

The following issues warrant specific response (the numbering refers to that in G. Powell discussion):

3.1 LACK OF EMPHASIS ON GEOLOGY

We do not agree that the Guidelines underplay the importance of good geological, geomorphological and geotechnical inputs. They are critical to any assessment of slope stability whether in a traditional or risk based framework. This aspect is the thrust of Section 3.2.1 of the Guidelines. The Guidelines do not set out to repeat what is already readily available in the literature on the geology and geotechnical engineering of slopes.

The assertion that engineering geologists are the source of all good slope assessments is contrary to the reality. There are many geotechnical engineers who are at least equally able, and who have an excellent feel for slope processes and geology. We agree that experience and competence are important – to both traditional or risk based assessments, and for geotechnical engineers or engineering geologists.

3.2 OVER-EMPHASIS ON QUANTIFICATION

We do not retreat from the position that quantification should be attempted, where loss of life is an issue. If it is not, decision makers have no viable risk tolerability criteria against which to assess the risk. However, as we point out in the Guidelines, there are situations where qualitative methods will suffice.

3.3 THE LANDSLIDE PAPER IS TOO PRESCRIPTIVE

The quotation by G. Powell, from our Section 3.5.3 is incomplete and fails to point out paragraph 2 "In some situations where risk to loss of life is identified as an issue in semi quantitative analysis, it may be possible to take immediate risk reduction measures without further assessment". Having said that, we do not retract from the position that the potential for loss of life should be considered, and that is best done in a quantified manner, so the risks can be assessed by those responsible. Unless this is systematically introduced into the profession, there is a likelihood that high risk loss of life situations will be overlooked, as they have been in the past, to the detriment of the public, and to the potential exposure of the professionals involved to legal proceedings.

3.4 THE APPLICABILITY OF EXAMPLE TERMS AND RISK MATRIX IN APPENDIX G

(a) Appendix G is an <u>example</u> of a risk matrix approach. It is headed such. The Guidelines clearly state other matrices may be used, provided they are defined by those using them. However, there would clearly be advantages in all adopting the Appendix G approach so far as practicable, so assessments are comparable. This is particularly the case where one authority, such as a local council, receive reports from a number of practitioners. Uniformity of terminology and definitions makes comparison and understanding significantly easier. Appendix G was developed after much discussion, and several iterations. Simpler (e.g. 3 x 3) matrices were preferred by some, but a viable simplified scheme could not be developed.

The use of dual descriptors (e.g. VL-L) is intended to allow some latitude (which for a specific case may be isolated to a single term), and in reality, the outcome may span a wider range, e.g. from VL to M, when the uncertainty in the inputs are allowed for. This may simply highlight that more detailed investigations are needed to refine the answer, or in some cases, the wide range may not be an issue.

(b) G. Powell is incorrect in asserting that it is not possible to make a qualitative judgement as to likelihood, and to use this judgement to assign a probability. Risk analysis, particularly where expert panels are used to assess conditional probabilities within event trees, commonly use such approaches.

(c) It is agreed that it is not necessary to use risk matrices – they were provided as an aid, not as a prescriptive requirement that they be used. For a quantitative analysis a matrix is unnecessary.

(d) It is agreed that stakeholders must be involved in the risk assessment process, as shown on Figure 1 of the Guidelines. However many stakeholders seek guidance from the risk analysist on what are tolerable risks, particularly for loss of life. It is for this reason that we have included some information on this issue.

4 SUMMARY

(a) Our Guidelines are directed primarily at skilled practicing geotechnical professionals. We are expecting that the IE Aust/AGS Landslide Taskforce will prepare guidelines for Slope Management and which will be presented in a format more readily used by owners and regulators.

(b) Example reports –

Example 1. This is a useful example, which could have been readily quantified for loss of life and damage, and hence enabled evaluation relative to acceptance criteria rather than simply accepting "No risk".

Example 2. The approach taken produced an apparently good outcome, to what appears to have been a fairly obvious case of relatively high risk. If the parties had not agreed to temporary support on the grounds the risk level had not been demonstrated, a quantitative approach would have assisted.

Example 3. In the first table, the risk levels are arbitrary and not quantified, and there is no "acceptable" criteria i.e. Is remedial work needed on risk levels 1 to 6 inclusive, or only 1 to 4. Our experience is that such simple index schemes have greater limitations when loss of life is involved.

In the second table, loss of life is ignored even though it is clearly an issue. It is specifically because of this sort of approach that we recommend life loss be considered in all assessments. The highway authority may be quite satisfied with the potential, for example, of a low likelihood of large damage, but the loss of life risks to society could be well beyond normally accepted limits. Since the likelihood had been quantified, it would be relatively straightforward to complete the quantitative analysis.

Example 4. The argument that a risk based quantification would have been misleading assumes the person doing the assessment would have ignored what is apparently vital information. There is no reason why that should be the case. Again there is no mention of loss of life, which may have been the critical issue.

The Editor,

RE: LANDSLIDE RISK MANAGEMENT CONCEPT AND GUIDELINESREPORT BY AGS SUB-COMMITTEE, AGS JOURNAL VOL 35 NO 1 MARCH 2000

I am writing to you with my comments on the above paper. The comments arise as the result of work I have been carrying out for the National Parks and Wildlife Service on Risk Assessments for the various lodges and infrastructure development in the Kosciuszko National Park following the Thredbo Landslide. Can you please pass them on to the committee responsible for issue of the report for their consideration.
My comments follow:

In Section 1.0 when talking about the deficiencies of the 1985 approach it appears to me that a significant shortfall of this earlier approach was that it did not consider the consequence of any identified hazard. That is, whilst the purpose of that paper was to define the risk of instability at a site, the word "risk" was used as a synonym for likelihood because it did not take into account the consequence of that landslide. Therefore the 1985 approach was a method for determining the likelihood of a landslide.

In Section 3.1, Scope Definition, I believe that any risk assessment which omits injury to persons or loss of life as a consequence should also carry a warning that it is an incomplete assessment. I say that because, if we are aware of a situation with the potential to pose a serious risk to a person's welfare i.e. tolerable, high or very high risk as defined by the guidelines document, then I believe that the duty of care in our legal system will require us to inform of that risk. I don't believe our legal system will allow us to avoid that duty by limiting our scope in the risk assessment. The duty of care is likely to still apply if the client specifically requests that injury / loss of life be omitted from the assessment. I believe it to be worthwhile to have a legal opinion on this aspect.

In Section 3.5.3 the recommendation is made that risk for loss of life should be quantified. However, the paper also recognises that in general any such quantification will require significant judgement to assign probabilities to the various elements of the probability product chain because of the difficulty of obtaining numerical data in most situations. As stated in the paper this approach can only be called (at best) a semi-quantitative approach.

I believe that any attempt to quantify what is in effect a qualitative judgement will have the effect of conveying a degree of accuracy to the assessment which it does not have. Further, having to assign quantitative estimates of likelihood to the assessment process will add a further degree of difficulty for no further gain in accuracy.

As an alternative and as a means of avoiding the above situations, I suggest that a qualitative scale of consequence can be used which addresses the varying prospects for injury / loss of life. That is, in Appendix G, the table which gives the qualitative measure of consequence can be modified to include the varying prospects for injury / loss of life.

I developed one such table for use in our work for the NPWS in which I assigned a likelihood for loss of life to each consequence level (see attached). The same likelihood descriptions for loss of life have been used as for the qualitative measure of likelihood. In the modified table I have assumed that the likelihood for injury is similar to that for loss of life. I have found that this approach can work successfully and that it is easily understood by non-geotechnical personnel. This same table has also been adopted by Wollongong Council for their internal use.

A possible criticism of this approach is that the description of consequence contains a probability term. However, I don't believe this to be a serious drawback because the table can in this instance be considered as combining vulnerability and consequence.

The Qualitative Risk Matrix in Appendix G.

I must admit to being confused by the dual notations on some of the risk levels in this table e.g. a D1 risk classified as an M-H risk level. Does this mean a risk level intermediate between M and H or does it mean that it can be either? In the latter case I presume that it is the assessor who decides which one to assign?

You would be aware that the risk analysis matrix given in AS/NZS 4360 : 1999 (the Standard) has only four risk levels. The Standard also gives a legend to describe these levels. I have tried to compare the risk levels of the matrix in the Standard with those of the matrix in the Guidelines document. However, I find that the wording which is provided in the Standard to describe the levels is very confusing. As a result I have found it difficult to compare the risk levels in the two matrices. I have interpreted the risk levels L, M and H in the Standard to compare respectively with levels VL, L and M of the Guidelines document. However, this requires one to interpret the legend for Level H of the Standard to mean that "senior management attention" allows for the possibility of no action by them to reduce the risk This may not be what is intended because why would you bring a situation to senior management's attention if they are not going to act on it?

If the above interpretation is correct then the E risk level of the Standard encompasses both the H and VH levels of the Guidelines document. I can understand why the Standard would group these two risk levels into a single unacceptable risk level. However I would still favour the use of the five risk categories as defined in the Guidelines paper. Notwithstanding, can the Committee offer comment on the earlier comparisons as I believe a correlation to the Standard is important. Was the Guidelines risk matrix discussed with any member of the Standards Committee?

Further, if the above interpretation is correct then the H level of the Standard would be equivalent of what we would generally accept as a tolerable risk.

Review of the risk matrix in the Standard shows that a hazard with an almost certain likelihood and minor consequence (i.e. an A2 hazard in the Standard) is tolerable. A B3 hazard is also tolerable in this matrix. In contrast the Guidelines document defines both these hazards as having an unacceptable risk.

At the low likelihood end of the matrix, the Standard considers a D5 hazard as unacceptable whilst the Guidelines document defines it as M-H, and an E4 as tolerable whilst the Guidelines document defines it as L-M.

That is, from the above it appears to me that the Standard has "skewed" the acceptability criterion in the direction of consequence - if the consequence is not great then it is prepared to consider the situation acceptable notwithstanding that other numerically similar risk levels (along the same matrix diagonal) but with a more serious consequence are defined as having an E level. This same "skewing" is apparent with the E4 hazard which the Standard considers to be tolerable whereas other hazards along this same matrix diagonal are considered acceptable.

The Guidelines document tends to follow a consistent pattern for risk rating along the diagonals except that it does allow some reduction in the risk level at the low likelihood end of the matrix.

I would be interested in the Committee's comments on the reasons why the risk matrix was structured in the manner shown in the report, particularly at the high likelihood end. In the document we have prepared for the NPWS I have structured our matrix by skewing it in the manner shown in the Standard at this end. I believe this to be more in keeping with what the general public might accept. For example from a personal point of view I would be prepared to accept the risk from an A2 hazard on the basis that any damage would likely be only of nuisance value. The alternative of this risk being classified as an unacceptable risk could result in large costs to clients.

In closing I'd like to take this opportunity to congratulate Bruce and the Committee on the publication of the paper. I believe it to be an excellent document and one which will aid enormously in the preparation and understanding of slope stability risk assessments.

Yours faithfully,

GHD-LONGMAC PTY LTD
Laurie de Ambrosis
Manager

Alternative Table for the Defination of Vulnerability and Consequence.

Qualitative Measures of Vulnerability and Consequences

Level	Descriptor	Description
1	CATASTROPHIC	Almost certain fatality, or structure completely destroyed or large scale damage requiring major engineering works for stabilisation.
2	MAJOR	Likely fatality or extensive damage to most of structure, or extending beyond site boundaries requiring significant stabilisation works.
3	MEDIUM	Possible fatality or moderate damage to some structure, or significant part of site requiring large stabilisation works.
4	MINOR	Unlikely fatality and limited damage to part of structure, or part of site requiring some reinstatement stabilisation works.
5	INSIGNIFICANT	Rare fatality and little damage.

Note: The "Description" may be edited to suit a particular case.

Note: The likelihood of human fatality has been added as an assessment criterion when compared to the Guidelines.

Dear Editor,

Re: Land Risk Management Concepts and Guidelines & Hillside Building Guidelines

I would like to congratulate the society for the publication of the latest guidelines which are a continuation of the "risk" and probabilistic approach to the assessment of slope stability pioneered by Dr Barry McMahon in 1971 [Ref: 1]. I also note that Dr McMahons' work was developed by myself to the urban environment in 1975 [Ref: 2] and later by a NSW committee of the society in 1985 [Ref: 3]. It is pleasing to see the way in which the "risk assessment" approach to the evaluation of slope stability has now been accepted nationally.

As such, my consulting practice has naturally adopted the latest guidelines and is incorporating the same in the various geotechnical reports issued by the firm.

However, in the course of implementing this policy it has become apparent that the guidelines need further development; this is because the latest publication really only provides guidance for the assessment of a "particular development" in a "specific" situation, whereas many geotechnical reports are written in an environment where the style, or details of a proposed development are not known. It is also difficult to apply the guidelines to the slopes behind cliff-top and coastal bluff areas where several conflicting mechanisms are occurring. Perhaps therefore the society could consider devoting one of its technical meetings / seminars to an open discussion of the guidelines to identify those areas of the guidelines that need further refinement.

Further, whilst the latest publication refers to the importance of "hazard zoning", there is little guidance [other than in the most general of terms] as to how such hazard zoning should be carried out. In this regard, I refer you to my 1975 paper in which a basis for hazard zoning as applied to town planning was suggested; I also note that this suggestion was further developed in 1983 [Ref: 4]. Also, as land stability issues are often associated with coastal processes, perhaps the more developed hazard identification / classification system for town planning could also embrace the work of coastal engineers [e.g. Lex Nielsen - Ref: 5].

I now refer to the suggestion by the society that the task of developing appropriate hillside building guidelines should be the subject of a "funded program" [i.e. carried out by a paid consultant, or group] and in this regard make the following observations:

1. In the society publications to date, no reference has been made to the work of Ingles in the 1970's on hillside building guidelines [Refs: 6 & 7], nor to his latest paper [Ref: 9] which addresses desirable hillside building practice & landslide risk zoning in Tasmania.
2. Whilst the Thredbo inquest paid particular attention to the Hillside Building Guidelines published by myself and Peter Burgess in 1976 [Ref: 8] the latest publication does not refer to these guidelines although the 1985 Society guidelines [Ref: 3] substantially embraced them.
3. Whilst the society as a whole has not provided extensive technical information on hillside building practice since 1985, a number of consulting firms [including my own] have regularly appended notes on hillside construction to their various geotechnical reports since the late 1970's, with some firms issuing very extensive notes.

In view of the above, I suggest that the society convene at an early date a committee to formulate a "draft set" hillside building guidelines based on what is currently available within the technical community. These draft guidelines could then be published in "draft form" [similar to the draft codes produced by Standards Australia] and form the basis for a series of "seminars / workshops" by the various state groups. These seminars would then naturally lead to a formal set of published guidelines.

In this way the finally published guidelines would have wide "technical consensus" with the significant expenses associated with the workshops, etc., being the subject of the "funded development".

Sincerely,

A F Shirley.
SHIRLEY CONSULTING ENGINEERS PTY LTD

References:

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2. Shirley A F [1975] "The Theoretical and Practical Aspects of Land Stability Classification" – Proceedings of 2nd Aust. & NZ Conference on Geomechanics - pp. 303-307.
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8. Burgess P J & Shirley A F [1976] 'Some Guidelines to Preferred Practice in Hillside Construction' Australian Geomechanics Journal - 1976.
9. Ingles O G & Pedersen A M [1999] "Recognition and Responsible Development of Unstable Land"; Proceedings of the 1999 Australian Disaster Conference 'Disaster Prevention for the 21st Century'. pp. 27-32.

cc A Leventhal, B Walker
B McMahon, O G Ingles.

The Editor,

Re: LANDSLIDE RISK MANAGEMENT CONCEPTS AND GUIDELINES
Letter from ANDREW SHIRLEY, Shirley Consulting Engineers Pty Ltd

I write as Chairman of the AGS Sub-Committee on Landslide Risk Management in response to the above letter.

We thank the Andrew Shirley for his contribution. We note that he has found some difficulty in applying the Guidelines to some situations.

In response, we consider it is sufficient to say that the methodology given in the Guidelines enables any situation to be addressed. For situations of "conflicting mechanisms", then each mechanism (that is, each hazard) should be addressed separately using the methodology. The interaction between the mechanisms (hazards) should also be considered. The risk assessment would only be limited by the understanding of the processes thought to be applicable.

Since the methodology given in the Guidelines requires consideration of the consequences of the hazard, it is necessary to have a "notional development" to enable the consequences to be evaluated. At a preliminary stage this may be simply a designated site area. Clearly if the hazard is considered unlikely to affect that area, then the form and details of the development are immaterial. If the hazard does affect that area, then the consequences could be considered in relation to different forms and extent of development, or alternatively the minimum treatment options identified. Clearly, there should be the interaction and reassessment implied by the Figure 1 Flowchart as the design develops.

The general steps for Hazard Zoning are given in Section 6 and should be sufficient for experienced practitioners. Other references may assist and we thank Andrew Shirley for those given. It is important to recognise that past zoning studies may have been titled as "hazard" or "risk" zoning studies, but may be neither under the current definitions.

Andrew Shirley's comments on hillside construction are noted. Appendix J of the Guidelines has been based on the Guidelines for Hillside Construction given in the 1985 paper but they have been revised to be consistent with the Risk Management Guidelines. It is hoped that the IEAust Landslide Taskforce will proceed to formulation of more extensive guidelines covering all aspects of landslides in relation to engineering works, including more detailed discussion of hillside construction. It is expected that the Risk Management Guidelines will form an integral part of such future guidelines.

B F WALKER
Chairman
AGS Sub-Committee on Landslide Risk Management

LETTERS TO THE EDITOR

The Editor,

Subject: Australian Geomechanics Society - Landslide Risk Management Concepts and Guidelines

I refer to our discussion in relation to this matter and make the following comments on the guidelines:

1 The definition of landslide in appendix B is very difficult to understand. It is very unclear and I also do not understand why ground subsidence and collapse are excluded;

2 While the document expressly deals with risk of loss of life or injury, appendix G is limited to property damage. No example seems to be provided of an assessment for loss of life or injury;

3 Most importantly, there appears to be some conflict between the concepts which underlie the document and the legal theory of liability in negligence. In short, the document focuses on tolerable and intolerable risks, which concept does not seem to take into account the capacity (in monetary and other terms) to remove the risk but, rather, appears to be based on the likelihood of the event and its consequences in terms of property damage or loss of life/injury. In terms of the common law of negligence, the practicality and cost associated with remedial options is critical and it is a relevant issue in determining whether or not a duty of care exists at all. In short, if reasonably practicable steps are available to remove even a tolerable risk, then the duty of care will be discharged.

The document, however, seems to work on the basis that if the risk is tolerable, then no steps, in particular, need to be taken.

4 It is not clear to me how the document distinguishes between loss of life and capacity for injury in terms of determining "vulnerability". In fact, although the document refers to injury, it seems to me that the actual risk assessment is based on loss of life, rather than injury. Particularly, the basis appears to be the annual probability of the loss of life of an individual (not the risk of injury at all). If this is the case, then the document should not purport to deal with risk of injury.

5 Related to my comments above, the consequence analysis in section 3.4 refers to a number of matters other than property damage, injury or loss of life. I think it is important to recognise that while those additional factors may be relevant to a decision to take some action to remove risk, they could never justify a decision not to take some action to remove risk.

That is to say, political effects, loss of business confidence and effect on reputation, as well as the potential for litigation, could not be relevant matters in determining whether or not removing or mitigating the risk is an appropriate action. Just as it is not appropriate for a car manufacturer to fail to withdraw cars from the market for a defect, the car manufacturer could not defend proceedings in negligence by reference to failing to take steps to protect their reputation. In deed, this would be an aggravating circumstance.

Kind regards

Jayne Jagot
Partner - National Environment and Planning Group Mallesons Stephen Jaques Sydney
Direct line (61 2) 9296 2196

The Editor,

E.H. Davis Memorial Lecture
Embankment Dams - Robin Fell

This year's E.H. Davis Memorial Lecture by Professor Robin Fell contains much worthy information and valuable insights into the design and performance of embankment dams. However, two sections of the lecture would, I suggest, benefit from further discussion.

Piping Failures.

As pointed out in the lecture, the vast majority of "piping" failures within embankment dams occur on first filling or on raising partially impounded reservoirs. Failure in these cases is rapid and located close to the (transient) level of the rising water level in the reservoir. Hydraulic gradients of i = 1/50 or even 1/100 obtain. Now, piping, by definition, refers to critical hydraulic exit gradients. In 1935, E.W. Lane published his weighted creep ratios using field evidence, demonstrating that an effective hydraulic gradient of about 1/8 was the cut off value for piping beneath dams, even in the most susceptible soils. Thus, what we have *within* the embankment dam is not a normal piping situation. It is, in fact, the result of porous horizons, usually located at the base of the layers of emplaced fill. Seepage from the reservoir travels along such horizons as a "hydraulic tunnelling" front. If it reaches the downstream face, a concentrated leak develops with subsequent erosion of the embankment. The process has been discussed elsewhere (1).

The above distinction might appear academic, but not so. If one does not define an adverse mechanism with clarity, then follow-on decisions may well be inappropriate. However, as Professor Fell rightly points out, the best safeguards against this sort of "piping" failure are adequate filter design and a suitably high placement water content. To this I would add good supervision during construction and an awareness of the nature of the problem.

Risk Analysis.

Risk analysis presented in the lecture appears to apply mainly to estimating the safety of existing dams. However, the problem with risk analysis in dam engineering is that the number of failures by any one mechanism - apart for the type of failure given above, and possibly the mechanism of sliding on systematic weak seams in the foundation - is typically too small to be of any statistical significance.

Indeed, in geotechnical dam design, I would suggest that the risk analysis has no place. This is not to suggest that one need not have a check list of potential failure modes, as listed in the risk analysis. However, these risks all need to be checked out on a deterministic, not a probabilistic basis. A dam owner presented with a list of the probabilities of failure should quite justifiably send the designers back to the drawing board, or to the site, to remove these probabilities, particularly since probabilities too often appear to be used to camouflage an inadequate definition of the relevant material parameters which would be required for an adequate deterministic approach.

An example of the above situation arose on a large rockfill dam project in Sri Lanka, in the early '80's. Geomorphologists had identified (erroneously as it turned out) a massive potential landslide in the reservoir. A sort of risk analysis debate resulted from this, with regard to the height of wave which might be generated and the height of wave which could be tolerated overtopping the dam. Then Ranji Casinder arrived on site to head the design team and stated quite simply that no wave overtopping the dam could be tolerated. People lived by and/or did their washing in the river, downstream. This philosophy led to a deterministic (rather than probabilistic) reassessment of the potential slide, with the result that no realistic mechanism for sliding was found to be present. The problem of probable waves overtopping the dam disappeared.

In design, one simply cannot treat loss of life on a probability basis. It would be useful to have other comments on this.

(1) James, P.M. (1996). "A note on the origin of wet seams in embankment dams".*Proceeds 7th Aus. N.Z. Geomech. Conf.*, Adelaide, 1- 5 July, 320-324.

Yours sincerely,

Peter James,
Consulting Geotechnical Engineer

LETTERS TO THE EDITOR

The Editor,

AUTHOR IN REPLY
The author would like to thank P. James for his contribution, and offers the following response.

PIPING FAILURES.

The definition of piping given by P. James is too restrictive and not used by the author. As pointed out in Section 3.2.1 of the paper, piping develops after the initiation and continuation of erosion, and may develop from a concentrated leak, backward erosion and suffusion. The example of a "porous horizon" is only one initiator of piping. Others are given in the paper. It is important to keep in mind all potential causes of initiation, and not to concentrate only on one. Piping can initiate in well constructed dams e.g. from cracks, or hydraulic fracture caused by differential settlements due for example to irregularities in the foundation, or "hang up" of a narrow core on stiff, well compacted filters and rockfill.

RISK ANALYSIS.

P. James arguments regarding the role of risk analysis are possibly reasonable for the design and construction of large, new dams, where one naturally tries to reduce the risks to as low as reasonably practicable.

They are not correct for existing dams, which is where most risk assessment is being applied. Many (or most) existing dams do not meet current standards, and risk assessment is a valuable tool in prioritising the remedial works on these dams within an owner's portfolio. This is done accounting for the likelihood of failure (for all modes of failure) and the consequences of failure.

the value of the risk assessment approach is emphasised by the fact that most large dam owning organisations in Australia, and organisations like USBR in the USA, are using it.

It is correct, as inferred by P. James, that a risk assessment can only be reasonable if it is based on sound engineering.

Sincerely,

Rob Fell

THE GREAT DEBATE

AGS Sydney Chapter – 14 June 2000

"This House believes Soil Testing is a Waste of Time"

It has to be said that there was a certain amount of prejudice in the large audience that assembled to hear Paul Hewitt, Garry Mostyn and Dick Davidson try to defend the motion against a strong attack by Tony Phillips, Philip Pells and John Carter. The Affirmative team knew they were in trouble from the start and stooped to some low and dirty tricks to gain a skerrick of credibility.

Garry Mostyn opened and, in accordance with normal debating protocol, restated the case. This he said, to the surprise of many of those present, was that *"This House believes that most laboratory testing is a waste of time"*. This brought some groans from the negative team, not to mention the audience. He followed this by introducing a series of major projects from the past including the Pyramids, the Colosseum, the Cathedral of Santa Maria del Fiore in Florence and the Eiffel Tower, claiming they had all been built without soils testing and they were all still standing, so testing was clearly unnecessary. Getting into his stride he ridiculed the Liquid Limit test as one of many tests that could hardly be less scientific. As a parting shot he presented the results of a survey he had carried out of the general public. Somewhat surprisingly he found that such worthies as George Speight, Kapil Dev, Cathy Freeman and President Bill Clinton all thought soil testing a waste of time. *"Who"*, he asked, *"would prefer to believe the speakers in the negative team, all with second degrees, when popular opinion so clearly showed them to be wrong?"* He sat down believing his arguments constituted a lay down misère for the affirmative team. He was right, but it was not a winning hand.

Philip Pells led for the negative team and pointed out that "*....it was Humpty Dumpty who said 'when I use a word it means just what I choose it to mean – neither more nor less' and we all know what happened to Humpty Dumpty*". The subject of the debate, he said, was about soil testing in whatever form. He then proceeded to mount a case based on his strongly held belief that engineering must involve rigorous application of the scientific method, which comprises "*observation, hypothesis, testing, theory and control*" To support his case he quoted from the writings of Henry Petroski and invoked the philosophy of Francis Bacon and Rene Descartes. He then concentrated on where testing fitted into the process, making the point that without testing the engineer had to play God in order to come up with a design. Testing, he assured the audience, can and did throw up unexpected results, without which the engineer would be almost bound to get the wrong answer. He gave two examples to support his case. One, a test from the Thredbo landslide where "static liquefaction" occurred in an undrained triaxial test leading to collapse of the sample. This explained why the failure had been so rapid. The other example was of a landslide on the banks of the Siak River in Sumatra, where a sample of the soil, when kneaded in the hand, exhibited all the characteristics of Norway's quick clays. In neither case was the behaviour of the soil expected and in neither case would it have been discovered without testing.

Dick Davidson spoke next for the affirmative team and carried its argument forward by looking at what real engineers did with real test data. He was able to show that in recent ANZ Geomechanics Conferences, papers concerned with soil testing all came from academics. This, he contended, showed that in the "real world" testing was considered to be a waste of time. He referred to a number of major projects that he had been involved with to support the affirmative case. There was the One Magnificent Mile Building in Chicago, a sky scraper, for which an enormous amount of laboratory testing was carried out and the results subjected to great scrutiny. However, at the end of the day, the foundation design parameters were based solely on local experience relating the moisture content of the soil to allowable bearing capacity. Another was the New Waddell dam in Arizona where, despite a large amount of good quality testing of a dissolving tuff, design decisions were taken by Ralph Peck to build a buttress even though testing demonstrated it was not needed. A final case was the evolution of the evaluation of liquefaction potential of sands where early research focused on complex dynamic testing of every conceivable variable, but late in his career Professor Harry Seed changed his mind and advocated that the only reliable method was based on nothing more than the standard penetration test results. Dick argued that these cases showed that even when testing of the highest quality was undertaken, the results tended to be ignored in favour of conservatism and the personal experience of the key decision makers. He went on to argue that a large number of firms that undoubtedly had geotechnical skills did not run their own laboratories. Why was this, he asked. Was it possible that these firms valued testing and yet took so little interest in it that they were prepared to contract it out? Would they be prepared to contract out the analysis component of the work? He rested his case.

John Carter brought an academic, but no less important, focus to the negative side's case. He broadened the argument to point out that, without testing, the complexities of soil behaviour would still be a total mystery and we would not have the strong basis of understanding that we do today. He singled out effective stress as one concept that would not have been derived, without a large amount of patient soil testing. This early breakthrough opened the doors to a long journey of understanding that depended on proper application of the scientific method espoused by Philip Pells. We have a long way to go on the journey, but it is leading to the development of constitutive models that now play an essential part in modern engineering. Critical State Soil Mechanics was clearly at the forefront, but its development depended on the careful analysis of test data on a large number of reconstituted soil samples. In his analysis every parameter is based on test results. For application in industry, the practising engineer needs test data as a reality check on assumptions made in analysis and in the resulting output. John referred to the North Rankin off-shore oil platform, where the design of foundations was based on the results of cone penetration tests in carbonate sands. That the piles did not carry the anticipated loads was a clear indication that more elaborate laboratory testing was needed to establish the differences between the fundamental behaviour of carbonate and non-carbonate sands. He concluded by making the important point that the correct amount of testing was not what an uninformed layman, often a client or project manager, thought was appropriate, but what the experienced engineer knows is needed to have confidence in the outcome. The amount of testing should not be determined by budget, but by the over-riding technical requirements of the job.

The two cases thus concluded, it was up to Paul Hewitt and Tony Phillips to sum up for their respective sides. Paul reiterated the affirmative side's view that soil testing is not a substitute for a knowledge of the soil profile, geology, groundwater conditions, experience, monitoring and field observations. A point with which the negative side did not disagree. He noted that tests such as Philip Pells kneading soil in his fingers in Sumatra was not a laboratory test, but more an excellent example of the importance of experience and precedent – since the nearest site investigation data was 300m away. Paul also argued that the initial laboratory testing for North Rankin was not addressed at establishing the mineralogy of the sands – hence was a waste of time. In this case there was no precedent for understanding the behaviour of piles in carbonate soils.

Tony Phillips felt that the affirmative side had trivialised the debate, by changing the topic and introducing furphies. In fact they had played straight into the negative side's hands. He pointed out that it was not surprising that Garry could only find very few examples of ancient structures that had been built without testing because the others had all fallen down. Indeed it was clear from the photographs that the Colosseum was still in the process of falling down. The examples of reliance being placed on moisture content measurement, or SPT's, did not show that soil testing was a waste of time, so much as that all test data is valuable and needs to be assessed in the light of experience. He made the point that without adequate test data the engineer is vulnerable to claims of negligence.

The motion was defeated in the eyes of the judges, Harry Poulos, Jack Hodgson and Kurt Douglas, and in the eyes of the audience which indicated by a show of hands a large majority in favour of the negative side's case.

Some important issues were raised. Simple commercial pressures are preventing practitioners from doing the amount of testing on projects that they really believe is necessary. The profession has only itself to blame for this since, in the race to win work, it has chopped the testing budget on many jobs to the point where it is almost non-existent. This has to be seen as a retrograde and dangerous course of action for the industry as a whole and a poor example of risk management. After all, how can an engineer who has guessed parameters and got them wrong, defend his actions in the face of a failure? Why should the client have to pay for that failure to do the job properly the first time? Engineering judgement and experience has a large part to play in the design process, but that judgement needs to based on facts that can only be obtained from testing.

Geodiary

ANCOLD CONFERENCE ON DAMS

OCTOBER 21-27, 2000

Themes:

Dams and the environment
Rehabilitation of dams
Dam safety management
Innovations in dam design and construction
New developments in hydro power
Flood mitigation and other benefits

Contact: Ian Paskins
Tel: 61 7 3225 1829 Fax: 61 7 3221 4441
Email: ian.paskins@dnr.qld.gov.au

AUSTRALIAN UNDERGROUND CONSTRUCTION AND TUNNELLING ASSOCIATION (AUCTA) WORKSHOP

NOVEMBER 18, 2000
University of Melbourne

Theme: Planning for Tunnelling in the Urban Environment
Topics: Underground space planning in Sydney
Urban expansion – planning issues from an engineering perspective
Scandinavian underground space practice
Melbourne's Eastern Freeway extension
Underground quarrying
Paper extracts should be 200 words and describe the key content of the paper

Contact: Alan Robertson
Tel: 07 3367 3388 Fax: 07 3367 1422 Email: tip@uq.net.au

INTERNATIONAL SYMPOSIUM ON SCOUR OF FOUNDATIONS (IS-SCOUR 2000)

NOVEMBER 18*, 2000
Melbourne, Australia

Organised by ISSMGE Technical Committee TC33 in conjunction with Geo Eng 2000.

Contact: AMM Safiullah, Chair Orgainising Committee, IS-SCOUR 2000
Professor and Dean, Faculty of Civil Engineering
Bangladesh University of Engineering and Technology
Dhaka-1000, Bangladesh
Tel: 990-2-866250 Email: safi@ce.buet.edu

GEO ENG 2000

NOVEMBER 19-24, 2000
Melbourne, Australia

Contact: Mr Max Ervin, Chairman Geo Eng 2000 Organising Committee, C/o Golder Associates Pty Ltd, 25 Burwood Road, Hawthorn, Victoria 31-5, Australia.

Fax: +61 3 9818 7990

GEODIARY

YEAR 2000 GEOTECHNICS

NOVEMBER 27-30, 2000
Bangkok, Thailand

Topics: Labrotory and field testing techniques including the centrifuge
Constitutive modelling of soils
Analytical methods in geotechnics
Soil structure interaction – piled foundations, buried pipe lines, tunnels
Cut slopes and excavations
Ground improvement techniques
Environmental geotechnics and natural hazards
Risk and reliability analysi

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School of Civil Engineering, Asian Institute of Technology,
PO Box 04, Klong Luang, Pathumthani 12120, Thailand
Tel: 66-2-524-5519, 66-2-524-5508 Fax: 66-2-516-2126, 66-2-524-5523
Email: bala@ait.ac.th

FOURTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS

MARCH 26-31, 2001
San Diego, USA

November 30, 1999 – Last Date for Receipt of Abstracts
August 1, 2000 – Last Date for Full Manuscripts

SHORT COURSE ON SOIL MECHANICS
A 1½ day short course on Soil Dynamics will be held on March 25-26 in San Diego. The topics will cover dynamic soil properties, elementary seismology, ground motion, amplification, liquefecation analysis of soils, stability analysis of rigid retaining walls, bridge abutments etc

ADDITIONAL INFORMATION
Information such as registration fee, proceedings of the conference, accomodations, social events, spouses' program, exhibition and post conference tours will be sent to persons who have pre-registered, and will be posted at:
http://www.umr.edu/~conted/conf8767.html

Contact: Shamsher Prakash, Conference Chairman
308 Civil Engineering, University of MO-Rolla
Rolla, MO-65409-0030 USA
Tel: 1-575-341-4489 Fax: 1-573-341-6553/4729
Email: prakash@novell.civil.umr.edu

INTERNATIONAL CONFERENCE ON TUNNELLING AND TRENCHLESS TECHNOLOGY

APRIL, 2001
Kuala Lumpur, Malaysia

Themes: Tunnelling includes process, operation, ventilation and maintenance
Micro-tunnelling, pipe jacking, directional drilling and rehabilitation
Detection, inspection services robotic development, sewer, services and structural aspects
Safety, quality and legal aspects

Contact: Conference Secretariat, Tunnelling and Trenchless Technology
The Institution of Engineers, Malaysia, Bangunan Ingenieur, Lots 60 & 62,
Jalan 52/4, PO Box 223 (Jalan Sultan), 42760 Petaling Jaya
Selangor Darul Ehsan, MALAYSIA
Tel: (603) 768 4001, (603) 768 4002 Fax: (603) 7577678
Email: iemactivities@usa.net www.jaring.my/iem/act/tt/html

REGIONAL CONFERENCE ON GEOTECHNICAL ASPECTS OF UNDERGROUND CONSTRUCTION IN SOFT GROUND (SHANGHAI 2001)

APRIL 16-18, 2001
Shanghai, China

Paper deadline: November 15, 2000

Contact: Prof. Guobin Liu
Secretary of Shanghai 2001, Geotechnical Engineering Department, Tongji University, 1239 Siping Rd., Shanghai, 200092, China
Tel: +86-21-65985013 Fax: +86-21-65985288
Email: shanghai2001@seakon.com http://www.seakon.com

IN SITU 2000

MAY, 2001
Bali, Indonesia

Themes: Site characterisation by in-situ tests
In situ measurement of shear strength, deformation and volume change characteristics
in situ measurement of initial stresses and stress history
in situ measurement of permeability and direction of groundwater contamination
Development of calibration chambers and theoretical interpretation of insitu tests
In situ tests for foundation design and geotechnical analysis
in situ tests for seismic analysis

Contact: Dr. Paulus P. Rahardjo
Soil Mechanics Laboratory, Department of Civil Engineering – Parhyangan Catholic University, Ciumbuleuit 94, Bandung 40141, Indonesia
Fax: 62-22-2038485 or 230698 Email: unpargec@home.unpar.ac.id
http://ntp.unpar.ac.id/gec

ENGINEERING GEOLOGICAL PROBLEMS OF URBAN AREAS

JULY 30 – 2 AUGUST, 2001
Ekaterinburg, Russia

Themes: Engineering geology and rational use of urban areas
Engineering geological and engineering environmental site investigations o urban areas
Natural hazards and stability of urban areas
Technogenous changes in urban environment
The use of urban underground space
Protectio nof historical, architecural and cultural sites
Geoinformation systems of urban geoenvironment
Deadline for summaries and preliminary forms: March 30, 2000

Contact: Secretariat of EngGeolCity-2001,UralTISIZ
79, Bazhov str.,Ekaterinburg, Russia, 620075
Tel: (3432)559-772 Fax: (3432)550-043
Email: uraltis@etel.ru Website: www.skyman.ru/~uraltisiz

NZ GEOTECHNICAL SOCIETY SYMPOSIUM 2001: "ENGINEERING IN HAZARDOUS TERRAIN"

AUGUST, 2001
Christchurch, New Zealand

Themes: River and coastal erosion, deposition, scouring etc
Slope failures, including landslides, debris flows, avalanches, etc
Active faulting, earthquake shaking, liqufecation
Volcanic activity
Man-made hazards
Contact: Kevin McManus, University of Canterbury,
Private Bag 4800, Christchurch, New Zealand
Tel: (03) 351 6808 Email: k.mcmanus@civil.canterbury.ac.nz

GEODIARY

AGGREGATE 2001 – ENVIRONMENT AND ECONOMY

AUGUST 6-10, 2001
Helsinki, Finland

Themes:
- Geology of aggregate production
- Classification of aggregat and available production techniques
- Prospecting and testing the raw materials for aggregate production
- Mineralogical studies and long term durability of aggregate
- Environmental influences of aggregate quarrying and processing
- Importance of aggregate industry for national economies

Contact: Aggregate 2001
Tampere University of Technology, Engineering Geology
PO Box 600, FIN 33101 Tampere, Finland
Email: AGGREGATE@RGY.FI (KUULAVAI@CC.TUT.FI)
Fax: +358 – 3 – 365 2884

FRONTIERS OF ROCK MECHANICS & SUSTAINABLE DEVELOPMENT IN THE 21ST CENTURY (ISRM 2001 – 2ND ARMS)

SEPTEMBER 11-14, 2001
Beijing, China

Themes:
- Rock mechanics testing and characterisation
- Physical, numerical modeling & monitoring systems in rock engineering
- Rock mechanics & rock engineering related to resources, environment and sustainable development
- Artificial intelligence, information systems and non-linear dynamics in rock mechanics and rock engineering
- New design & construction methods and case histories of major projects

Contact: Prof. Fu Bingjun, Secretary General
ISRM 2001 - 2nd Arms
Institute of Geology and Geophysics, Chinese Academy of Sciences
PO Box 9825, Qijiahuozi, Beijing (100029) CHINA
Tel: +86 10 62008067 Fax: +86 10 62040574
Email: egml@igcas.igcas.ac.cn http://isrm2001.homepage.com

GHENT ENVIRONMENTAL GEOTECHNICS SPECIALTY CONFERENCE: UNDERWATER GEOENVIRONMENTAL ISSUES

OCTOBER 29 – 31, 2001
Ghent, Belgium

Themes:
- Environmental dredging
- Underwater ground improvement
- In site and off site remediation of dredged material
- Chemical and biochemical phenomenae related to soft soil mechanics
- Laboratory geoenvironmental testing related to soft soil
- Land reclamation problems
- Waste ponds/tailing dams design

Contact: Organizing Committee of GEG
tel: +329 2645717 – (2645723) Fax: +329 2645849
http://allserv.rug.ac.be/~wvanimpe/

2ND AUSTRALIA-NEW ZEALAND CONFERENCE ON ENVIRONMENTAL GEOTECHNICS

NOVEMBER 28-30, 2001
Newcastle, Australia

Submission of Abstracts 29 Dec 2000

Themes:
- Site investigation and transport modelling
- Engineering solutions
- Implementation and economic planing and regulatory frameworks

Contact: ICMS Pty Ltd, 3rd Floor, 379 Kent St, Sydney, NSW 2000, Australia
Tel: 02 92903366 Fax: 92902444 Email: geoenv@icms.com.au

IS KYUSHU 2001 – EARTH REINFORCEMENT

NOVEMBER 14-16, 2001
Fukoko, Japan

Themes: Embankments, Wall structures, Foundations and Slopes and excavations

Submission of abstracts (300 words)by September 30, 2000

Contact: Prof. Hidetoshi Ochiai
Department of Civil Engineering, Kyushu University, 6-10-1, Hakozaki, Higashi-ku, Fukoka 812-8581, Japan
Tel & Fax: +81-92-642-3285 Email: iskyushu@civil.kyushu-u.ac.jp
http://www.civil.kyushu-u.ac.jp/geotech/iskyushu/

XV INTERNATIONAL CONGRESS ISSMFE

2001
Istanbul, Turkey

Themes: Testing and property characterisation of geomaterials
Foundations and retaining structures
Tunnelling and underground space development
Ground improvement and reinforcement
Environmental issues of geotechnical engineering
Design, construction and maintenance of transportation infrastructure

Attention is drawn to a new award for ISSMGE members under 36 years of age on 31 December 2001 who have made an outstanding contribution to the development of geotechnical engineering as judged by a paper from the previous regional conference (Hobart) or to the ICSMGE, with an accompaning CV

3RD INTERNATIONAL CONFERENCE ON SOFT SOIL ENGINEERING

DECEMBER 6-8, 2001
Hong Kong

Themes: Soft material behaviour, constitutive modelling and predictions
Field monitoring, numerical and physical modelling
Ground improvement techniques and settlement control
Hazard mitigation, environmental and risk management

Contact: Miss May Ho/Miss Ming Li, Conference and Seminar Section, Hong Kong Institute of Engineers, 9/F Island Beverly, 1 Great George St, Causeway Bay, Hong Kong
Tel: 852-2895-4446 Fax: 852-2577-7791 Email: conf@hkie.org.hk

THE 14TH SOUTHEAST ASIAN GEOTECHNICAL CONFERENCE: GEOTECHNICAL ENGINEERING MEETING SOCIETY'S NEEDS

DECEMBER 10-14, 2001
Hong Kong

Themes: Managing risk
Environmental Geotechnics
The value of geotechnics in design and construction
Theoretical advances and scientific innnovations
Professional practice
Specialty session on slope engineering

Contact: Miss May Ho, Conference and Seminar Section
Hong Kong Institution of Engineers, 9/F Island Centre, 1 Great George St
Causeway Bay, Hong Kong
Tel: +852 2895 4446 Fax: +852 2577 7791 Email: conf@hkie.org.hk

ADVANCING ROCK MECHANICS FRONTIERS TO MEET THE CHALLENGES OF THE 21ST CENTURY

SEPTEMBER 24 – 27, 2002
New Delhi, India

Topics: Engineerin geology, site investigations and field testing
Stress-strain and strength characteristics and rheological behaviour of jointed rocks
Numerical & physical modelling and solutions in rock mass
Underground works in weak rocks including tunneling
Design and construction of large caverns
Techniques for improving quality of rock mass
Surface excavations in rocks
Foundations of large dams and other large structures on rocks
Deep drilling technology and innovations
Environmental issues of underground excavations and storage of contaminates

Contact: Mr ARG Rao, *Treasurer*
Indian Group of ISRM c/o Central Board of Irrigation and Power
Malcha Marg, Chanakyapuri
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EDITORIAL POLICY

Australian Geomechanics is published quarterly, in March, June, September and December, by the Institution of Engineers Australia, it is edited and produced by the Australian Geomechanics Society. It provides a journal and news magazine for matters of interest to the Australian geotechnical community. The statements made or opinions expressed do not necessarily reflect the views of the Institution or the AGS.

The Editorial Panel of Australian Geomechanics seeks contributions for future editions. The following comments are offered to assist would-be contributors.

Contributions can include: refereed technical papers; technical papers or notes; or news items and reports.

Technical papers can be refereed to ensure that they are of a standard similar to those published in international geotechnical journals. Authors should aim for a maximum overall length of no more than 3500 words, although shorter papers or technical notes are particularly welcome. Authors should indicate if they want their submision to be refereed, the status of the paper will be indicated on publication.

Refereed technical papers should be original and:

- Papers on geotechnical engineering, engineering geology and environmental geomechanics. Papers should be topical, practically oriented and preferably of national interest. Case studies describing innovative geotechnical work are particularly encouraged.
- Papers on geotechnical or geoscience research undertaken in Australia or of relevance to Australian geomechanics. These should clearly indicate their practical relevance and limitations.
- Authoritative reviews of aspects of geotechnical practice or aspects of geotechnical education.

Technical papers or notes can be:

- Items as above but submitted for rapid publishing. These will not be refereed but will be reviewed. They will be accepted at the discretion of the editorial panel. The intention is to provide a source for rapid dissemination of technical material to the geotechnical community.
- Discussions on papers published in previous editions.

News items and reports can be:

- Items describing significant projects, instructive failures, conferences, courses or other matters of general interest to the Australian geotechnical community.
- Geotechnical book reviews.
- Letters to the Editor.

It is preferable for contributors to submit formatted text, tables and figures in electronic format using WordPerfect or Microsoft Word on IBM compatible hardware. Contributions are preferred by email or on 3.5" floppy disk in either of these formats. **All contributions should be followed by a hard copy (i.e. laser printed or ink on paper).**

Submitted material should be presented in the following format :

- Single column format on A4 paper.
- Left and right margins of 20 mm.
- A top margin of 30 mm and a bottom margin of 25 mm.
- 10 point character size of Times font with single (normal) line spacing.
- A single blank line between paragraphs, and after headings. Two blank lines before internal headings.
- No indent at the beginning of paragraphs.
- Main headings numbered 1, 2, 3.... etc. in 12 point Times, bold, upper-case and centred in the column.
- Sub-headings numbered 2.1, 2.2, 2.3 ... etc. in 10 point Times, bold, upper-case and left justified.
- Items in bulleted or numbered lists should not be separated by a line, but should be indented by 10 mm.
- Formulae typed and numbered (1), (2), (3) ... etc. and centred in the column.
- Captions for figures and tables should be placed beneath the item and numbered Figure 1, Table 1 etc., and referred to in the text as figure 1, table 1, etc. Figures and tables should be centered in the column.

EDITORIAL POLICY

- Do NOT use page numbers, these will be added later.
- In text citation according to the Harvard system of author (year) or (author, year) as appropriate. Multiple references should be separated by semicolons (author1, year1; author2, year2)
- References should be listed at the end of the paper in alphabetical order using the Harvard system: Author (year) title, publication, volume, pages, publisher with a 10 mm hanging indent and no blank line between each.
- Underlining should be avoided and symbols shown in italics.

FIGURES AND TABLES

Where possible figures and tables should be placed at the correct position in the text. Figures should be scanned. Failing this place them together at the back of the text. These should be sharp black on white and of the correct size for incorporation into the finished document. The width of these must be less than or equal to the width of the text column (165 mm). Do not use colour unless you have discussed it with the editors.

Photographs should preferably be good contrast black and white gloss prints and of the correct size for incorporation directly into the copy. Please ensure that all such items are clearly marked to indicate position in paper.

Authors will remain responsible for the integrity of their material and for permission to publish.

Material will be accepted at any time and published in the next available issue.

EDITORIAL CONTACTS

Contributions and other correspondence should be forwarded to:

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E-Mail	agsnews@civeng.unsw.edu.au

All members of the AGS National Committee have defined roles in providing papers and reports. Contributors are encouraged to liaise with their state representatives. Contact details for these are listed the first issue each year.

ADVERTISING RATES

Every three months, Australian Geomechanics reaches nearly 1000 professional geotechnical engineers and engineering geologists spread throughout Australia. Most of these are associated with significant field work and computing. As such Australian Geomechanics provides a very targeted delivery for advertising.

Advertising rates are:

	One issue	Two issues	Four issues
Colour			
Inside front or back cover or back cover	$800	$1400	$2000
Black & white			
Full page	$400	$700	$1000
Half page	$200	$350	$500
Quarter page	$150	$250	$350

The prices quoted are for advertisements supplied with either camera ready art for single colour ads or 4 colour separated negatives for colour.

Inserts into an individual mailout of Australian Geomechanics can be accepted at the rate of $50 per A4 leaf, with a minimum charge of $500.