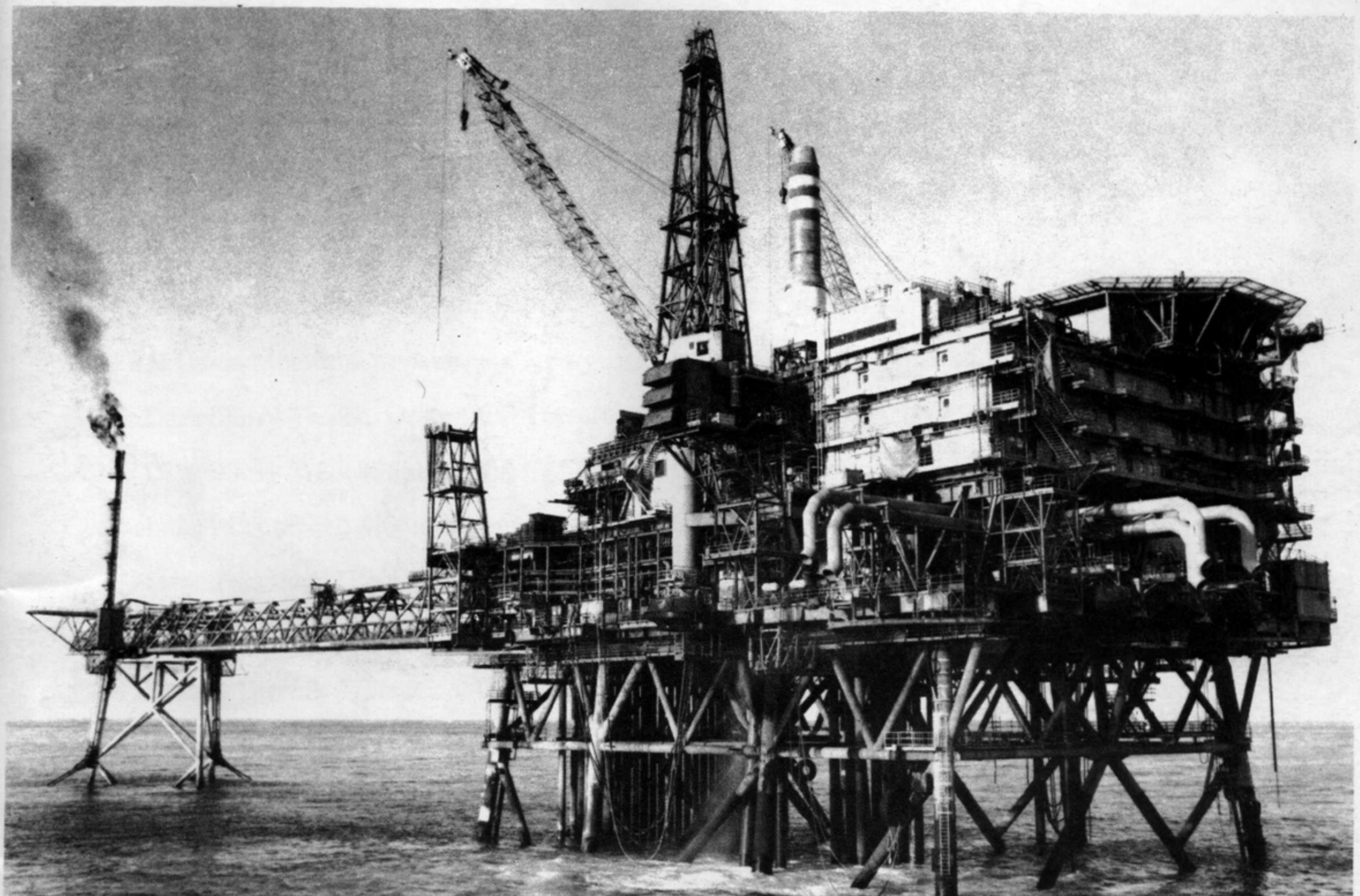
Australian Geomechanics



News Journal of the Australian Geomechanics Society

ISSN 0818 - 9110

No. 19 DEC. 1990

FRONT COVER

The cover photograph shows the North Rankin "A" platform. This is the first of the offshore natural gas production platforms to be developed by Woodside Offshore Petroleum Pty Ltd and its joint venture partners as part of the North West Shelf Gas Project off the Western Australian coast. The foundations for the platform were the subject of a major review investigation and upgrading programme which was described in the 1989 EH Davis Memorial Lecture delivered by Dr Mohamed Khorshid and which is featured in this issue of Australian Geomechanics.

-Photograph appears by courtesy of Woodside Offshore Petroleum Pty Ltd.

AUSTRALIAN GEOMECHANICS

Issue No. 19 **CONTENTS** **December 1990**

AUSTRALIAN GEOMECHANICS

News Journal of the Australian Geomechanics Society

ISSN 0818 - 9110 **No. 19 DECEMBER 1990**

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EDITOR'S NOTES

The Editorial Panel of Australian Geomechanics seeks contributions for future editions. The following comments are offered to assist would-be contributors.

Technical contributions can include any of the following:-

-Papers, not necessarily of standard or content required for acceptance in, say, the Transactions of the I.E.Aust. State groups might consider submitting selected addresses.

-Technical notes.

-Comments on papers published in Australian Geomechanics.

-Brief notes on "wrinkles" encountered in the practice of geotechnical engineering which a contributor may be prepared to share with readers.

-Descriptions of geotechnical projects of special interest.

-Failures or "partial successes". Share your experiences with others.

-Contributions for the various regular columns and features.

Australian Geomechanics is now being produced in electronic format using Pagemaker 3 on IBM compatible hardware. Contributions are therefore preferred in computer format.

The ideal is, of course, as a Pagemaker document, however a variety of word-processing formats are acceptable. Wordperfect to version 5.1, Wordstar and Multimate can all be accepted. Text submitted as an ASCII file is also acceptable as are documents generated on Macintosh hardware. Please specify with your submission the format used to generate the file and include a hard copy. If the submission includes figures or photographs, these may incorporated directly into Pagemaker documents submitted. If submission is in word-processing format, please submit camera ready copies of all figures and photographs.

Contributors may still present camera ready material which should either A4 size if prepared on a laser printer or A3 for lesser quality of print. The following guidelines will assist with maintaining some uniformity of style and production (applies in the main to production on other than laser printers):-

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Photographs: These should preferably be good contrast black and white gloss prints and of the correct size for incorporation.

Position: 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.

Contributors are reminded that the deadlines for submission of material are 1 May for the June edition and 1 Nov for the December edition. Contributions should be forwarded to the Editorial Panel, Australian Geomechanics, C/- Dames and Moore, 26 Lyall St, SOUTH PERTH WA 6151.

CONTRIBUTIONS FOR THE DECEMBER 1990 EDITION WILL BE ACCEPTED UP TO 16th JANUARY 1991.

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EDITORIAL

This is the first edition of Australian Geomechanics to be edited by the Western Australian Group. We have of course approached the task with much journalistic zeal, aiming to at least emulate the high standards set by our predecessors in the editorial chair.

In this edition you will note a few changes of format. In the future you may see more, hopefully for the better, as we develop the a fore mentioned journalistic zeal.

One of the principal objectives of the new Editorial Panel is to narrow the gap that seems to exist between geotechnical people in the mining and civil engineering areas. Whilst recognising that such a gap is purely artificial, it is nevertheless there. This was underlined when the AGS WA Chairman only knew, *even by sight*, approximately half of the audience at a recent meeting addressed by Prof Barry Whittaker, whose special areas of interest include mining subsidence and slope stability.

The WA Group proposes to try to strengthen the bonds between the two groups and, to this end, we are scheduling several talks of mining interest. We are also investigating the possibility of setting up an AGS group in Kalgoorlie, where there is currently a great deal of interest in geotechnical matters, probably because of the recent open pit slope failures and the subsequent interest now being directed onto mining geotechnics by the Department of Mines!

It is in the interests of all of us to promote liaison between the "miners" and the "civil engineers" to overcome the perception that "the AGS is just for civil engineers" (heard during recent discussions with a group of mining people.) Will potential mining topic authors please take it on notice that their contributions will be gratefully received (and may even get "special consideration"!).

In 1991, we will embark on something of a first as we produce an edition containing a special feature dealing with "Waste Management". This topic appears to be an ideal vehicle to provide for this greater level of interchange between *the miners* and *the civil engineers.*

To support an expanded edition , we will be actively canvassing advertisers. This in itself will result in a somewhat changed format. Your comments in due course would be welcome.

These changes may be considered as representing something of the changes within Geotechnical Engineering in Australia as we see people like Professor Mark Randolph bringing a fresh approach to many aspects of the profession. This issue of Australian Geomechanics brings together some of these new developments as we present a major article on the UWA centrifuge and a guest editorial prepared by Mark Randolph.

Also in this issue we feature the 1989 E.H. Davis Memorial Lecture. This is particularly fitting as the award is a memorial to one of the pioneers of geomechanics in Australia whilst this lecture presents a new generation pioneering a new era. The subject of the lecture, "Geotechnical Aspects of the North West Shelf Project", represents the catalyst which proved to be so much a part of the development of the new facilities and faces which are now a part of our profession particularly in Western Australia.

It would be difficult to overestimate the part played by Dr Khorshid and Woodside Offshore Petroleum along with its joint venturers in developing local engineering and geotechnical skills through the foundations project for the North Rankin A platform.

The research and investigation associated with the project has significantly advanced our understanding of calcareous soils and elevated Australia to an eminent position in this area. To this, the success of the Calcareous Soils Conference in 1988 is ample testament.

Notwithstanding this position, we must remain forward thinking and move to meet the new challenges of this, the last decade of the twentieth century.

One of these major challenges must surely be the management of waste. In this generation we are presented with a two pronged challenge as we set about to manage the "mistakes" of the past as well as managing the mass of environmentally threatening products which are generated by our industrialised lifestyle. The whole style and quality of life which will be enjoyed by our children and other generations to follow may well depend upon how successfully we meet this challenge today.

Another challenge confronting all professions including geotechnical practitioners is the vexing question of legal liability. The "era" of waste management can only elevate the importance of this challenge as the consuming public seeks to place higher and higher levels of responsibility upon the shoulders of all professionals.

In this regard there are some promising signs that we may avoid the legal morass that seems to beset the professions in the United States. The Institution of Engineers, Australia has recently conducted a series of seminars around the country to bring engineers up to date with the present position and canvass the views of those directly involved.This exercise will be followed by the publication of a discussion paper later in the year. Hopefully this may present further light at the end of the tunnel.

Charles Waterton, Colin Bradbury, Trevor Osborne, Martin Press and Denis Smith. Editorial Panel 1990

GUEST EDITORIAL

POSTGRADUATE EDUCATION OF GEOTECHNICAL ENGINEERS

Professor M. F. Randolph, Department of Civil & Environmental Engineering
The University of Western Australia

I was very pleased to be invited to contribute this Guest Editorial. It comes at an exciting and challenging time, both for me personally, and for engineers and teachers in the geotechnical profession throughout Australia.

There have been a number of initiatives in recent months in connection with the education of engineers, particularly at postgraduate level. A key element of the planned Co-operative Research Centres, which are discussed further below, will be postgraduate research training. This is intended to redress the current paucity of scientists and technologists educated to Masters or PhD level. A parallel move has come from the Institution of Engineers, with the proposed requirements for Continuing Education of graduate engineers. This initiative, although it appears to undervalue the longer term role of higher degrees, clearly acknowledges the insufficiency of even a four year Bachelor degree in today's specialist and changing engineering world.

Somewhat at odds with the perceived needs for postgraduate training referred to above, the Institution has also recently introduced a requirement for a minimum quantity of 'management' topics within undergraduate degree courses. Initially, the interpretation of management topics is to be relatively broad, including any elements of a course that address communication and design skills. However, there appears to be a background trend towards explicit management courses, such as those more usually encountered in postgraduate MBA courses. These courses would be feasible in a five year engineering course, such as the double degrees in Engineering and Management or Commerce that are currently available, but would inevitably dilute the technical content of standard four year degrees.

Rather than stipulating a minimum level of management content in undergraduate engineering courses, the Institution of Engineers should judge individual courses on the overall quality of the graduates produced. Courses that produce highly skilled technical engineers who, if they so wish, may proceed to higher level specialist education, should not be penalised in the interests of uniform mediocrity.

Continuing education is essential for engineers, both to acquire formal management skills and to update technical skills. Higher degrees should play a major role in this area In Europe and the North American continent, a Masters or PhD degree is considered essential in order to specialise in geotechnical work. The Australian Geomechanics Society should be lobbying for a similar standard. In the past, potential postgraduate students have been deterred by the lack of support from government, in terms of the level of scholarships, and from industry, in terms of salary levels that reflect the additional training. The situation is now changing. Recent increases in the value of postgraduate scholarships, and the introduction of industry-linked awards, have improved the terms under which higher degrees may be earned. The industry should now seize this opportunity to encourage graduates of 2 - 5 years experience to return to university for further specialist education.

The Federal Government has recently launched a scheme for Commonwealth funded Co-operative Research Centres. The principal goal of these centres, which will concentrate on science and engineering, is to provide further links between industry and research groups. Particular emphasis has been placed on postgraduate training through higher degrees, with direct involvement of postgraduate students in industry.

It is interesting to contrast the role of geotechnical specialists in two key industries in Australia - offshore engineering and mining. This issue of Australian Geomechanics contains Dr Khorshid's E.H.Davis Lecture, which describes the geotechnical experience on the North West Shelf. The early design for the North Rankin 'A' foundations was dominated by European and American consultants. The need for remedial treatment of the foundations initiated a major programme of research on calcareous soils throughout the world, but with a high percentage of that research being carried out in Australia. As a result, world expertise on calcareous soils now rests in Australia and foundation design for the subsequent Goodwyn platform was performed primarily by Australian engineers. The programme of research underwrote many PhD theses and there is now an appropriate pool of specialists within the industry.

By contrast, the mining industry has a woefully low presence of geotechnical engineers. In Western Australia, a large percentage of gold mines are designed with no specialist geotechnical input and suffer accordingly from high rates of failure. There has also been a recent spate of failures of open-pit slopes. The financial consequences of these failures dwarf the remedial measures of North Rankin. There is an urgent need for a major re-assessment of geotechnical design in the mining industry, and for an increased level of postgraduate research training.

To summarise, geotechnical engineers have a major role to play in Australian society, particularly in the development of natural resources. As a body that has close connections with the Institution of Engineers, the Australian Geomechanics Society should lobby for increased training of geotechnical engineers. We should resist any reduction of technical content at the undergraduate level, and push for greater recognition of higher degrees in the realm of continuing education.

E.H. DAVIS MEMORIAL LECTURES

The E.H. Davis Memorial Lecture was established in 1983 by the Australian Geomechanics Society to honour a man who was one of the pioneers of geomechanics in Australia.

Ted Davis came to Australia from England in 1952. He joined the University of Sydney as a Senior Lecturer in Civil Engineering and began to develop lectures and laboratory classes in soil mechanics for undergraduate students. He proceeded to put the subject of soil mechanics on a sound theoretical basis.

He perceived that progress and understanding would only come if consistent, theoretically sound but simple models of soil behaviour were used. He proceeded to apply the theory of elasticity to foundation deformation problems and the theory of plasticity to stability problems. These two topics remained the central focus of his research interests during his career, and both were advanced significantly by his work. He also integrated into his sound analytical framework, the theory of consolidation of clay soils and again made a major contribution in this area.

Ted was always extremely conscious of the link between theory and practice, the relationship between the idealised and real material, and the engineering significance of his work. He was particularly conscious of the need to present theoretical results in a manner which could readily be understood and applied by the practicing engineer. Consequently, he was widely sought as a consultant.

He was instrumental in the formation of the Australian Geomechanics Society in 1970, and served as its Chairman and also represented the Australasian Region as Vice President of the International Society of Soil Mechanics and Foundation Engineering between 1969 and 1973.

Two highlights of Ted's career came in 1980, towards the end of his life:

* he was elected to the Australian Academy of Sciences; and
* he was selected as the inaugural John Jaeger Memorial Medallist.

Following Ted's untimely death in 1981, it was decided that the E.H. Davis Memorial Lecture would be established and that it would be delivered every two years by a person who had made "distinguished recent contributions to the theory and practice of geomechanics in Australia".

-Reprinted from "Civil Engineering Transactions" Vol.,CE30, No3, October 1988. Published by The Institution of Engineers, Australia.

Previous E.H.Davis Memorial Lectures have been delivered by Dr. Barry K. McMahon and Prof.. Harry G. Poulos.

These lectures:

* Geotechnical Design in the Face of Uncertainty; and
* From Theory to Practice in Pile Design:

have previously been published in "Australian Geomechanics" and appear in the issue of "Civil Engineering Transactions", referenced above.

IMPORTANT NOTE FOR POTENTIAL AUTHORS

6TH AUSTRALIA - NEW ZEALAND CONFERENCE ON GEOMECHANICS

Note that although this conference has been announced to have the theme "Geotechnical Risk - Identification, Evaluation and Solutions", it will not be exclusively on this theme, and papers are invited on the full range of Geomechanics topics, as has been the case for previous ANZ conferences.

Because of the late distribution of the Call for Abstracts (the pink brochure mailed to you recently) and the misunderstanding about the range of subject matter to be covered, we have requested the Conference Committee to extend the date for the receipt of abstracts to February 1, 1991.

DR. M. KHORSHID
- 1989 E.H. DAVIS MEMORIAL LECTURER

The 1989 E.H. Davis Memorial Lecture was delivered in Sydney on 8th November 1989 by Dr. Mohamed Khorshid. His lecture presented the highlights of the North Rankin "A" foundations project which has spread over seven years and become this country's most significant and costly geotechnical project.

The project has won wide acclaim and is recognised as being the catalyst for much of the recent development of geotechnical research facilities throughout Australia.

Earlier this year, the project was awarded "The Sir William Hudson Award" by the Institution of Engineers, Australia. This is the Institution's highest engineering excellence award.

The appointment of Dr. Khorshid as the 1989 E.H.Davis Memorial Lecturer is a fitting testimony to the very significant part that he played in the successful realisation of the project.

Dr. Khorshid graduated with a B.Sc. in Civil Engineering from Ain Shams University, Cairo in 1968. In 1972 he obtained a M.Sc. in Civil Engineering from Kings College in London and was subsequently awarded a Ph.D., also from Kings College.

After three years as a research scientist in the CSIRO Division of Applied Geomechanics; two years as Project Manager on a major industrial development in Saudi Arabia and two years with Soil Surveys Pty Ltd, Dr. Khorshid joined Woodside Offshore Petroleum in 1981 as Chief Geotechnical Engineer. He currently holds the position of Manager for Civil Engineering with Woodside Offshore Petroleum.

Dr Khorshid (left) is seen here at the presentation with Mrs K Davis and Dr N Mattes.

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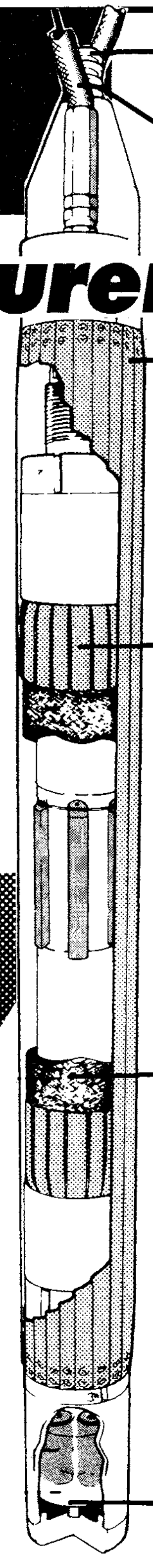

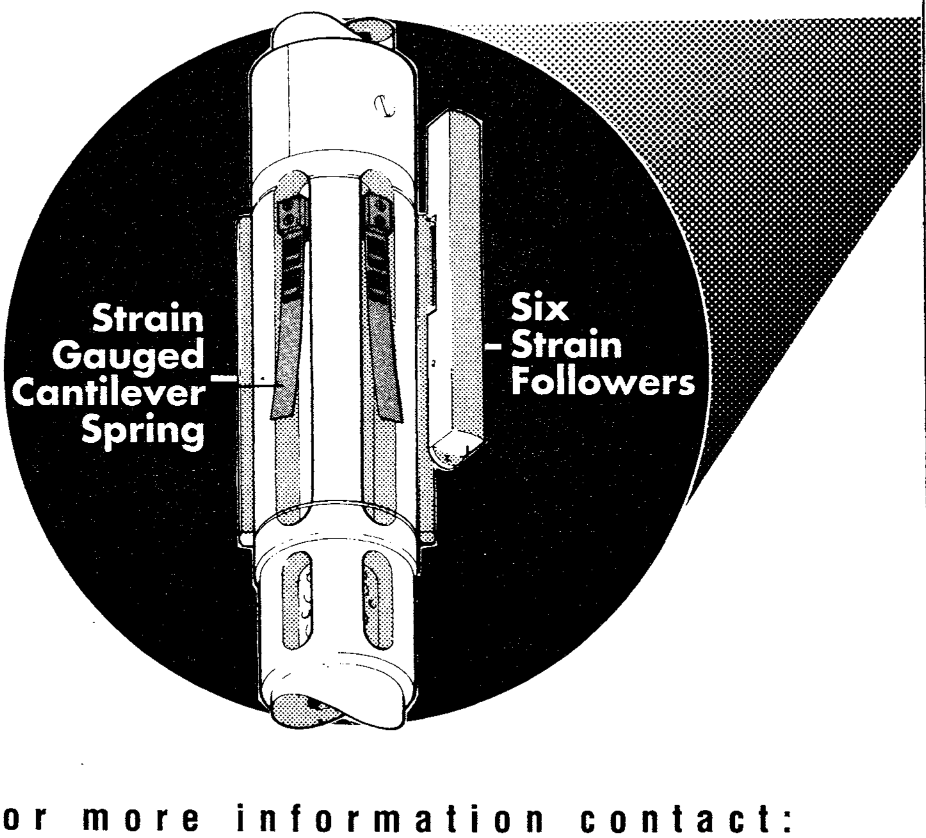

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DEVELOPMENT OF GEOTECHNICAL EXPERIENCE ON THE NORTH WEST SHELF

Dr. Mohamed Sief El Din Khorshid

ABSTRACT

This paper describes the development of geotechnical experience on the North West Shelf, from the early 70 's when natural gas was first discovered, to 1989 at the completion of the Goodwyn A (GWA) foundation design. The development of the original foundation design criteria for the North Rankin A (NRA) platform is outlined along with the realisation that a significant remedial programme was needed with the adoption of a solution based on end bearing. The logic for adoption of drilled and grouted piles for the GWA platform and the development of design criteria for these piles is also discussed.

INTRODUCTION

The NRA platform is a gas and condensate drilling and production facility located 134 kilometres off the north-west coast of Western Australia and stands in 125 m of water. The GWA platform is located 25 kilometres to the south west of NRA and stands in 130 m water depth. Both platforms are designed to withstand, with adequate safety margins, the 100 year return period cyclonic storm. Figure 1 shows isometric views of both the NRA and the GWA platforms.

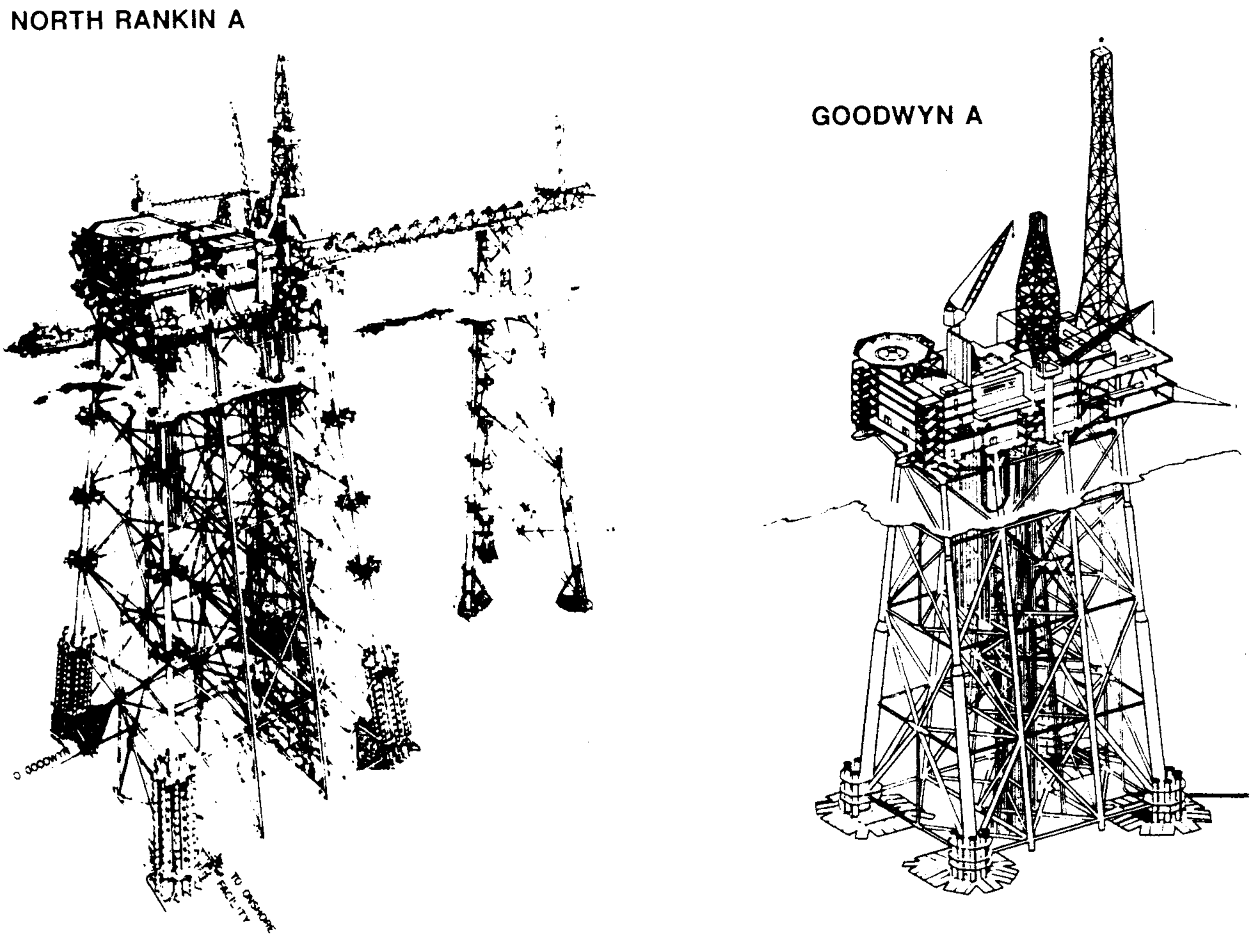

FIGURE 1 - NORTH RANKIN A & GOODWYN A PLATFORMS

This paper was presented as the E.H.Davis Memorial Lecture in October 1989.

Dr Khorshid is the Manager for Civil Engineering for Woodside Offshore Petroleum Pty Ltd.

DEVELOPMENT OF NORTH RANKIN A FOUNDATION DESIGN CRITERIA

Extensive site investigations were undertaken for the NRA foundation design. These commenced in the early seventies following the discovery of gas on the North West Shelf.

Exploratory Site Investigation (1972).

The first site investigation was limited to the classification of borehole cuttings during the drilling of exploration wells and shallow seismic surveys. At this early stage it became known that the seabed was formed of calcareous sediments generally weak in the top 100 metres and stronger below that depth. The geotechnical consultant's report stated that the friction mobilised on a driven shaft would be approximately one tenth of that which could be mobilised by a drilled and grouted pile. Loss of friction on driven piles was attributed to crushing and disturbance of the high void ratio sediments adjacent to the driven pile. The report recommended the use of drilled and grouted piles in this material.

Preliminary Site Investigation (1974)

By 1974 the gas reserves on the North West Shelf were identified and a major offshore development seemed likely. To enable development of realistic cost estimates a preliminary site investigation was carried out. This investigation was undertaken from a semi-submersible drilling vessel and comprised wireline sampling with a driven sampling tool, geophysical logging and insitu tests on both drilled and grouted and driven tubulars. In addition, an open hole stability test was performed to verify the constructability of drilled and grouted piles in the weak calcareous deposits.

This site investigation confirmed the findings of the earlier work and provided a basis for the design of the drilled and grouted piles based on the pull tests.

Though the insitu tests were undertaken from a semi-submersible vessel, the loading system was not heave compensated and the results were affected by cycling due to the vessel motion. In addition, only disturbed samples were recovered by the driven sampling tool. Nonetheless, this site investigation confirmed the previous findings that the top 100 m of sediments were relatively weak and the underlying layer was generally stronger.

This investigation also recommended the adoption of drilled and grouted piles based on the results of the field tests.

Final Site Investigation (1978) and Pile Design

After the North West Shelf Project was set up, a detailed site investigation was carried out. A consultant was selected who was experienced with the latest site investigation techniques and with geotechnical expertise developed for the early North Sea platforms.

This investigation included cone penetrometer tests (CPT), push tube sampling and coring to a depth of 140m below the sea bed. Plate load tests and lateral pile tests were also performed at the mudline to provide design criteria for the mudmat and lateral pile stability calculations.

The use of the CPT provided clear evidence of the stratigraphy and provided a measure of the strength of the underlying calcarenite layers. The realisation that the calcarenite was significantly stronger than thought previously resurrected the driven pile option.

The site investigation was modified to evaluate the friction on a driven pile. This required the development of a special down hole tool, the steel friction test (SFT). The SFT used the down hole jacking system for the CPT tool to insert and then pull out a thin walled tubular pipe 2.5m long and 59mm in diameter. Figure 2 shows a general arrangement for the SFT tool.

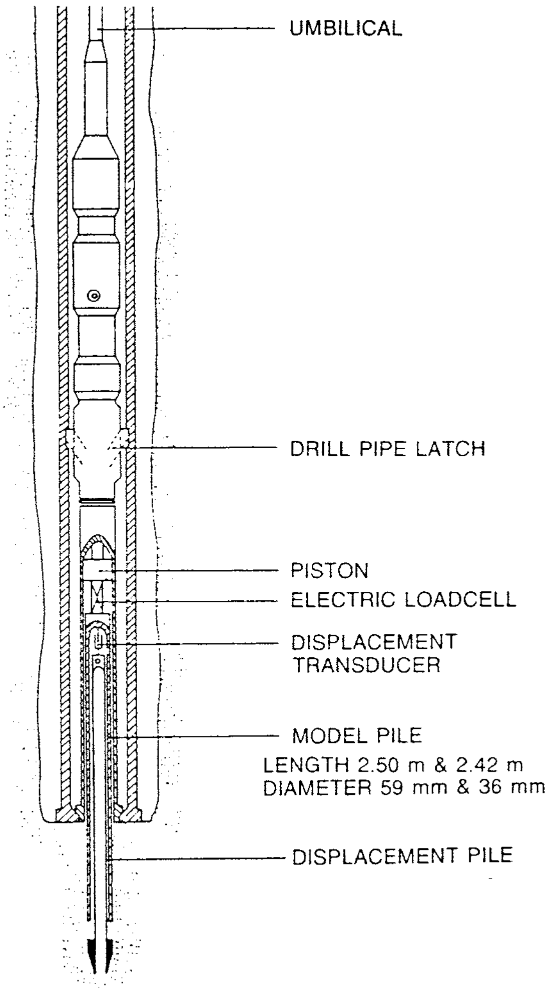

FIGURE – 2 STEEL FRICTION TEST APPARATUS

From the SFT results and the evaluation of strength, the top 100m of sediments were expected to collapse onto the pile wall thus ensuring the mobilisation of shaft friction. Figure 3 shows results of the SFTs. A value of 40kPa was adopted as the design unit friction based on these results.

The recommendation to adopt driven piles was controversial due to the poor track record of driven piles in this type of material and the Esso experience with driven piles on the first generation of Bass Strait platforms. Pull tests on conductors carried out by Esso suggested design friction values of the order of only 10-15kPa. Unfortunately, none of the tests carried out offshore at that stage were instrumented and the design friction values were deduced from average peak and residual static tests.

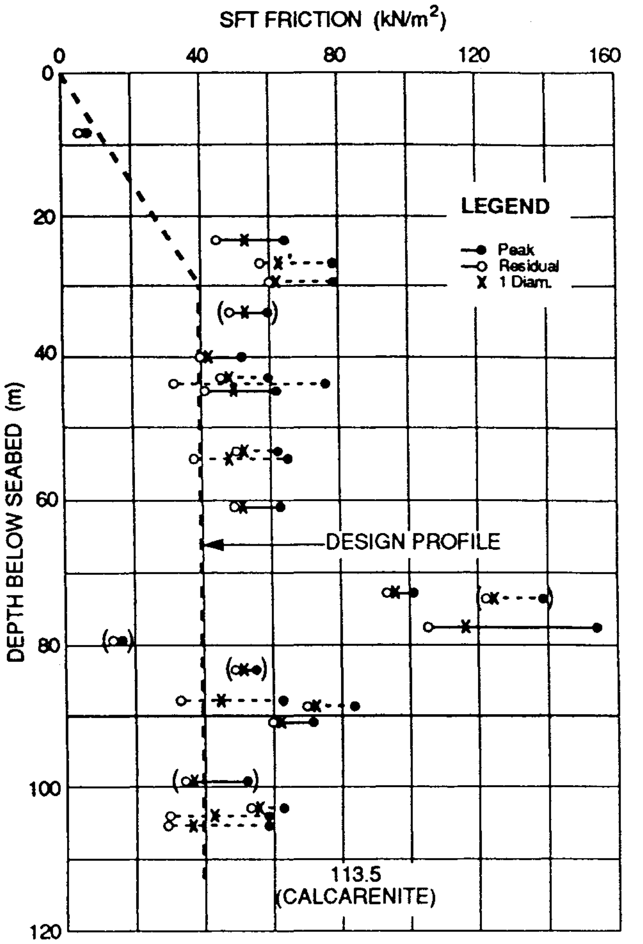

FIGURE 3 - RESULTS OF THE SFT's & THE DESIGN FRICTION PROFILE

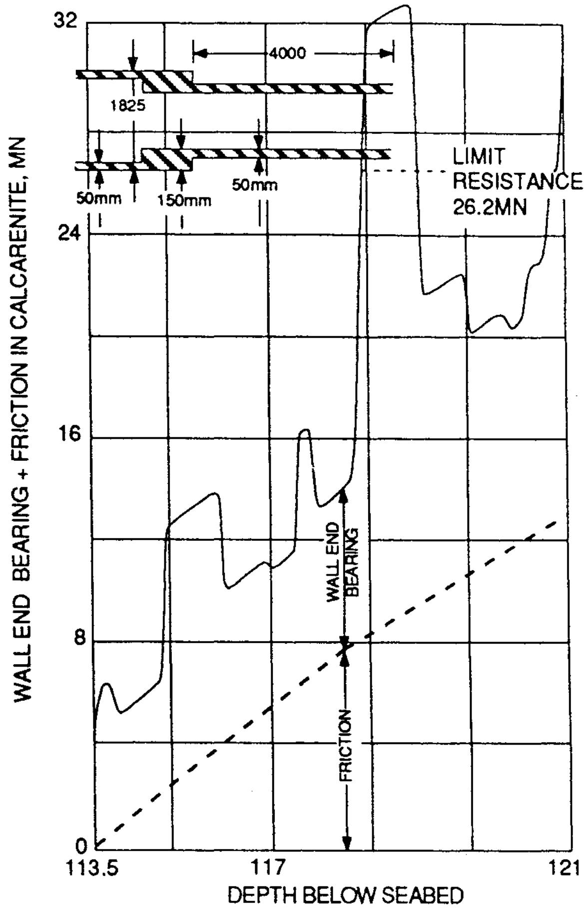

FIGURE 4 - STEP TAPER SHOE & PILE TIP CAPACITY

However, because of possible lower friction values, a contingency plan was adopted. This consisted of installing grout plugs in the driven pile should the driving records indicate friction values less than design. Weld beads were installed in the pile at the anticipated level of the top of the soil plug and at the tip of the pile, the latter location of the grout plug being required if the friction turned out as low as the Bass Straight tests indicated.

At that time difficulties were being experienced with driving piles in the Arabian Gulf calcarenites. This caused concern with the ability to drive the NRA piles through the intermediate 3.5m thick hard layer at approximately 65m below sea bed. To ensure drivability of the 1.84m diameter piles the largest offshore steam hammers (Menck 12,500) were specified. This hammer was later changed to the underwater hydraulic hammer, MHU-700. In addition, the pile was constructed with a step-taper shoe such that the 1.5m diameter portion penetrated the intermediate hard layer at 65m before the shoulder reached the top of this layer. This shoe also increased the displacement ratio of the pile which was expected to increase the mobilised pile friction by increasing the normal stress on the pile shaft.

The ultimate end-bearing resistance of the pile was evaluated on the basis of empirical formulae developed for the CPT in silica sands. However, the maximum capacity of the pile tip was limited to 10 MPa on the projected tip area, as this was the maximum unit end-bearing capacity allowed for dense sands in the offshore environment. Figure 4 shows the driving shoe and the predicted end bearing capacity of a driven pile.

The design pile capacity of 48.2MN was made up of 22MN in friction and 26.2MN in end-bearing. A factor of safety of 2.0 resulted between the maximum loading capacity and the design storm load.

If appropriate the fall-back options of placing grout plugs on top of the inner soil column or at the pile tips would satisfy the code requirement of a safety factor of 1.5 on the design 100 year storm load.

NORTH RANKIN A PILE INSTALLATION

Pile driveability predictions.

The predicted blow count for installation of the NRA piles for a MHU 1700 hammer was 4000-5000 blows per pile. The upper-bound parameters suggested the possibility of pile refusal at the design depth. Figure 5 shows a typical example of predicted versus actual pile driveability.

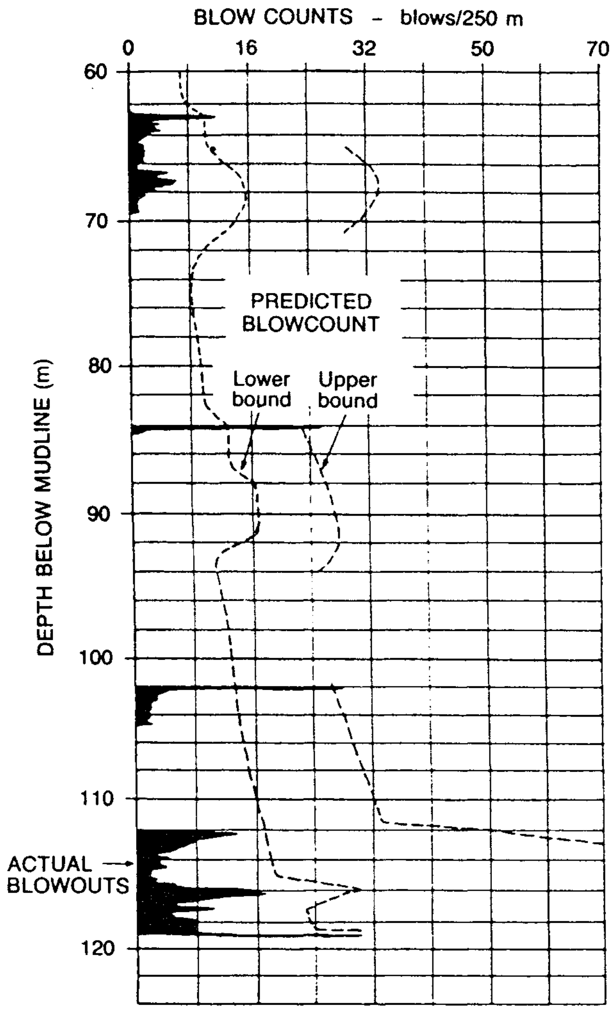

FIGURE - 5 PREDICTED & ACTUAL PILE DRIVEABILITY

Pile instrumentation for capacity derivation.

The pile capacity can be estimated from the pile penetration per blow if the hammer energy is known. However, to increase the confidence in pile capacity, especially for monitoring the friction component, underwater instrumentation was developed. This instrumentation was installed on some piles. Data from this instrumentation, in conjunction with hammer impact velocity measurement, were considered to provide a basis for the accurate evaluation of the pile capacities.

To validate the prediction of the pile static capacity from the dynamic data, both the CAPWAP and the OPTIMISATION programmes were used by independent consultants.

Installation of the Piles.

The NRA piles virtually free fell through calcareous sands and silts in the top 112m, needing to be driven only through the intermediate hard calcarenite layers. On average pile installation needed only 400-500 blows with half the hammer energy assumed in the original analyses.

With the aid of an underwater remote camera, the free-fall velocity of a pile was estimated to have reached 3.2m/s. The fact that the piles stopped at the hard layers and required driving only through the calcarenite below a depth of 112m suggested that the bulk of the capacity was being mobilised in end bearing. Pile re-drive tests also indicated that setup was occurring, with predicted pile capacities in the order of 31-37MN.

Initially, it was considered that approximately half the pile capacity was developed in end bearing with the remaining capacity being generated in friction after the excess pore pressures, developed during pile installation, dissipated. In a last minute effort to achieve an acceptable 'as-driven' pile, the end bearing area at the pile tips was increased by welding internal pipe segments in the pile shoe. A total of 6 piles were installed with these shoes. However, these piles did not appear to have higher as-driven capacity. Moreover, re-drive tests required lower initial blow counts, suggesting a softer foundation response.

To test the effect of the step taper shoe, a pile was installed with the driving shoe having been cut off. However, this pile was found to have the lowest as-installed capacity.

Analysis of the pile instrumentation data.

Analyses of the instrumentation data from the NRA piles, proved rather tedious because of the sensitivity of the results to parameter selection due mainly to the low friction component.

Irrespective of the parameters used, a general trend emerged which suggested that the friction capacity degraded by approximately 20 % with successive hammer blows, from a capacity of approximately 30MN on the first blow to 6MN after 100 blows. Figure 6 shows the friction degradation with successive hammer blows.

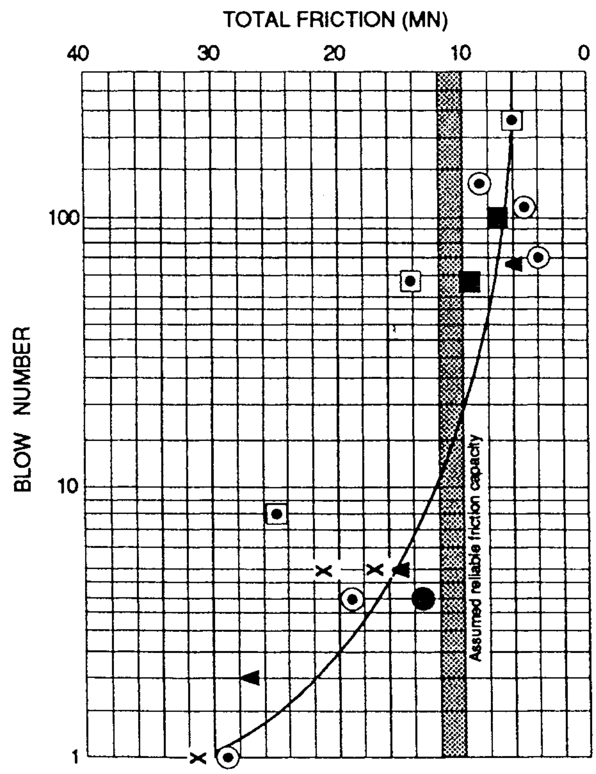

FIGURE 6 - DEGRADATION OF FRICTION SRD

This rapid loss in capacity raised questions about the reliable pile friction value which could be deduced from these data. Opinions ranged from the reliance on the first blow capacity "hence acceptance of the piles as installed", since pile driving induces strains which the pile is not subjected to during its life exposure, to reliance on the residual capacity "hence the need for major remedial actions", since the friction on driven piles in calcareous deposits was unreliable, as demonstrated by the conductor tests.

Following an extensive evaluation period, it was decided to adopt the friction capacity associated with the 10th hammer blow (12MN), which required the placement of the grout plugs at the tips of all the piles.

This friction capacity was considered to be a conservative evaluation of the pile friction. To provide confidence for the future of NRA, and design criteria for the second platform then under consideration (North Rankin B), it was decided that pull tests on conductors should be carried out to verify the design friction value.

CONDUCTOR PULL TESTS

While the equipment to undertake the conductor pull tests on the 760mm well drilling conductors was being fabricated, early production conductors were observed to settle under their own weight when drilling for the 500mm casing below the conductor tip. Since a unit friction of only 5.5kPa was needed to resist the weight of the conductor, and the reliable unit friction, deduced from the pile driving records, exceeded 15kPa, the initial loss of friction was attributed to the well drilling procedure.

The conductors had been drilled with direct circulation, using mud slugs to remove cuttings. It was considered possible that if mud had contaminated the conductor outside surface, the friction could be reduced to less than 5.5kPa, and hence self weight penetration would ensue. Two conductor tests were to be carried out. Each test included an initial static pull test, followed by post peak cyclic loading to determine a reliable residual capacity. The first test was on a conductor set to a depth of 65m thereby generating a similar axial stiffness as the piles. The second test was on a conductor set to a depth of 112m to test the effects of embedment.

CONDUCTOR TESTED	DEPTH BELOW MUDLINE (m)	PEAK SKIN FRICTION (kPa)	RESIDUAL SKIN FRICTION (kPa)	DATE
NW5	63.7	14.5	8.9	Sept. 1983
CE1	112.5	7.5	5.6	Oct. 1983
CE1	113.0	4.6	3.0	Feb. 1984 (111 days set up)
NE2 - TEST 1	111.6	7.6*	-	Sep. 1984
NE2 - TEST 1A	111.6	18.6	-	Sep. 1984
NE2 - TEST 2	116.5	4.1	1.4 - 2.0	Sep. 1984
SE3	112.5	15.6*	-	Oct. 1984

* No Failure

TABLE 1 - CONDUCTOR LOAD TEST DATA

Table 1 gives the results of this test programme and it can be seen that the peak unit friction was only 14.5kPa on the short conductor and reduced to 7.5kPa for the longer conductor. Residual values were 8.9kPa on the short test and 5.6kPa on the longer test. The most important (and worrying) result was that the unit friction after cycling reduced to below the level needed to support the conductors weight.

The validity of using the conductor pull test result to evaluate pile unit friction values was questioned since the conductors stiffness at full length was significantly lower than that of the piles. The low stiffness would cause the conductors to deform laterally during driving more than the piles, thereby reducing the normal stress and the mobilised friction. The tests were further limited by the loss of the underwater strain measurements, thus limiting the evaluation to average unit friction values.

Friction on the driven piles was finally considered to be unreliable following a pull test on a conductor which was allowed 111 days to setup. This conductor mobilised lower friction values than the previous residual values. It then became clear that a project team was required to investigate and implement some form of foundation improvement.

THE NORTH RANKIN "A" FOUNDATION PROJECT

The project brief was to design and construct foundation modifications such that the platform would have the required demonstrated integrity. This was to be done within the constraints of an operating platform and a tight schedule. A test programme was formulated to assess the conventional methods for foundation improvement. The following options were considered.

(a) Installation of insert piles through the primary piles: Additional capacity would be generated by mobilising friction on the drilled and grouted shaft of the insert piles. This was initially considered to be the most likely solution for the following reasons:-

* it caused minimal loss of capacity during construction
* it mobilised the capacity of several layers beneath the pile tips, and
* it provided a stiff response in both tension and compression.

Moreover, drilled and grouted piles are the conventional technique for foundation construction in calcareous sediments.

(b) Installation of bells at the tips of the piles:

Additional capacity would be generated by increasing the end bearing area of the piles.

(c) Reinstatement of the friction on the driven piles:

The friction on the driven piles would be reinstated by injection of grout in the disturbed zone adjacent to the pile circumference through one-way valves which would need to be installed in the pile wall.

(d) Improving the end bearing capacity of the layers at the pile tips by means of cementitious or chemical grouts:

Consolidation of the weaker calcarenite layers by repeated pressure grouting or the injection of low viscosity chemical grouts would lead to an increase in the inter-particle bond thereby improving the founding layer bearing capacity.

(e) Guying the platform and/or the flare support structure:

Stiff guy wires would resist part of the overturning moment on the structure and hence reduce pile loads.

(f) Freezing the foundation:

A frozen formation would have substantially higher capacities and would most likely meet the design requirement as installed. Though the concept had theoretical potential it was considered to be too difficult and was therefore dropped. However, at a later date the concept was resurrected, as detailed in the section of this paper describing the fall-back options.

New approach to the geotechnical work

After the installation it became clear that most of the original foundation behaviour assumptions were invalid and a better appreciation of the mechanical behaviour of the material was necessary prior to the adoption of any remedial solution. Recovery of high quality undisturbed material, and the undertaking of comprehensive laboratory tests coupled with fabric analysis, would provide a reliable basis for understanding the material mechanical behaviour.

It was decided that all design parameters were needed to be validated by a large scale field tests carried out from NRA.

Field Tests on NRA

A special testing module was developed from which the bulk of the test programme was undertaken. Both drilling and loading equipment were mounted on this module which could be skidded along the NRA drilling derrick skid beams. Testing was to be carried out in three conductors. The test programme however, had to ensure that these conductors could be used for future hydrocarbon recovery.

The test module was located some 50m above mean sea level and the tests were all carried out below the pile tip depth of 119m. Thus tests were undertaken some 300m below the equipment level. This necessitated the development of special tools and techniques for the testing programme.

Coring

Coring from NRA has been very successful due to the use of a triple tube PQ core barrel and normal on-land coring techniques. The availability of a stable drilling platform was a significant improvement on the earlier drilling attempts from small vessels. This technique has since been tried from a semi submersible drilling vessel and found to be equally successful.

The recovery of high quality relatively undisturbed material, coupled with extensive physical and index testing, enabled the development of a comprehensive stratigraphic model of the calcareous sediments below the pile tips.

A comprehensive suite of laboratory testing was also carried out on the recovered samples. These tests included triaxial tests (isotropic, anisotropic, cyclic and stress path testing) simple shear and constant normal stiffness direct shear tests, static and cyclic tests, consolidation tests, model pile tests and fabric analysis of samples.

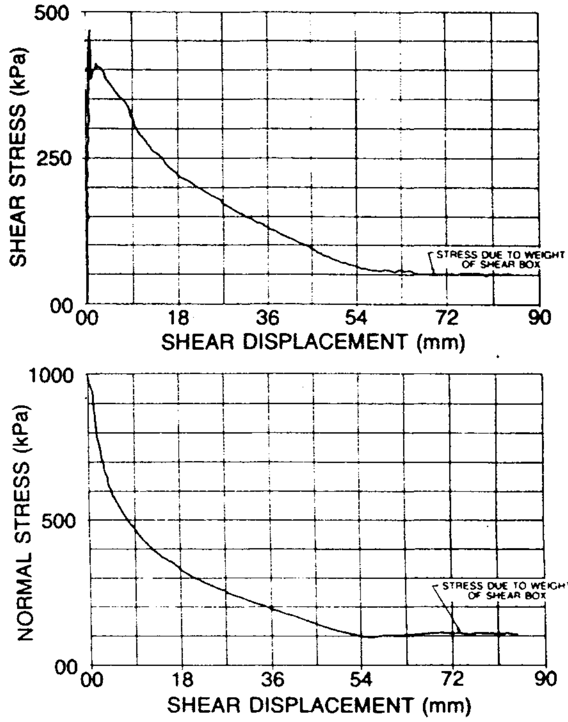

FIGURE – 7 CONSTANT NORMAL STIFFNESS TEST RESULTS

THE FRICTION RESPONSE

The friction response is best highlighted by the results from a constant normal stiffness (CNS) direct shear test (Figure 7) from which it can be seen that the mobilised shear strength increases rapidly to a peak stress and then reduces. This reduction in the mobilised shear stress correlates with the loss of the normal stress due to negative dilation (crushing).

Three tests were carried out through the conductors in which a 400mm diameter, 3.0m long grouted section was loaded to failure in tension. The tests exhibited a similar behaviour to that predicted based on the laboratory tests. However, the tests were of limited use as there was a loss of loading control post peak due to the elastic energy accumulated in the 300m long loading column. The elastic stretch resulted in a test section being displaced some 300mm before equilibrium was reached. It was noted, however, that a near total loss of friction capacity resulted when the test section was lowered through the failed zone.

The brittle response noted in most of the laboratory and field tests, coupled with the dramatic loss of friction capacity in some tests, caused concern about the reliability of any remedial solution based on friction.

END BEARING RESPONSE

The P-Alpha model was found to best represent the material behaviour. In this model, the material is assumed to behave elastically to yield following which the stiffness is significantly reduced, representing the phase where the inter-particle cementing is destroyed. The stiffness increases with strain as the void volume reduces.

Consolidation tests in oedometers, the triaxial apparatus and small scale model tests were found to be well represented by this model.

Predictions of the 500mm diameter plate load tests (PLT) carried out at NRA were generally very good. The elastic-plastic end bearing response and the ability to predict the PLT test behaviour, increased the confidence in the end bearing solution.

POSITIONS OF THE PILE TIPS

It was essential to know exactly the position of the pile tips for both the pile insert option and the bell option. Techniques to establish the position of the pile tips were not available at that time and it was necessary to modify tools used in oil well drilling to cope with the significantly larger 1.84m diameter piles.

Down-hole tools and probabilistic techniques were developed to measure the portion of the pile above the existing grout plug, and hence predict the pile tip and the position projected at various levels below the tip.

Results from this work showed that, in general, the pile tips converged and that there was a high probability of projection clashes at some 20m below the pile tip level. Figure 8 shows an example of the pile tip projections with depth.

EFFECTS OF THE PILE TIP CONVERGENCE ON THE REMEDIAL SOLUTIONS

Effects of pile tip convergence on the insert solution

On the basis of the pile tip projection study only 5 inserts could be installed with sufficient (90%) confidence. The individual and group pile tip responses were calculated by two independent programmes, AXCOL and RATZ. Both programmes indicated that the capacity of the five inserts was marginal, based on the conservative input parameters used. This, coupled with the uncertainty of the brittle response and the dramatic loss of friction observed in one of the field tests and the laboratory tests, were the main reasons for dropping the insert option.

Effects of pile tip convergence on the end bearing (bells) solution

The philosophy adopted initially was to increase the end bearing such that the ultimate loads would fall within the elastic range. An early estimate of the elastic limit was 3MPa, resulting in the need for approximately 3m diameter bells at the tips of all eight piles. However, due to the pile tip convergence only 4 bells of 4.65m diameter could be installed with the required confidence level. This resulted in an ultimate stress of 5.0MPa, slightly above the elastic range. Therefore the evaluation of the end bearing response to this load level was required to account for the effects of excess pore pressure generation. The field tests had been carried out in a drained manner and hence there were no large scale field data available to calibrate the undrained response of the material. The MOLENKAMP numerical model was adopted for the evaluation of the end bearing response. This model is a non-linear elastic double-hardening plastic model. The model was calibrated against the triaxial and oedometer tests and was then able to predict the drained field plate load tests.

The undrained response of a group of four 4.65m diameter bells did not satisfy the design criteria with zero primary pile friction capacity. As this condition was unrealistic, partial consolidation equivalent to the self weight of the platform, was assumed. With this assumption the design criteria were satisfied and this option was finally adopted.

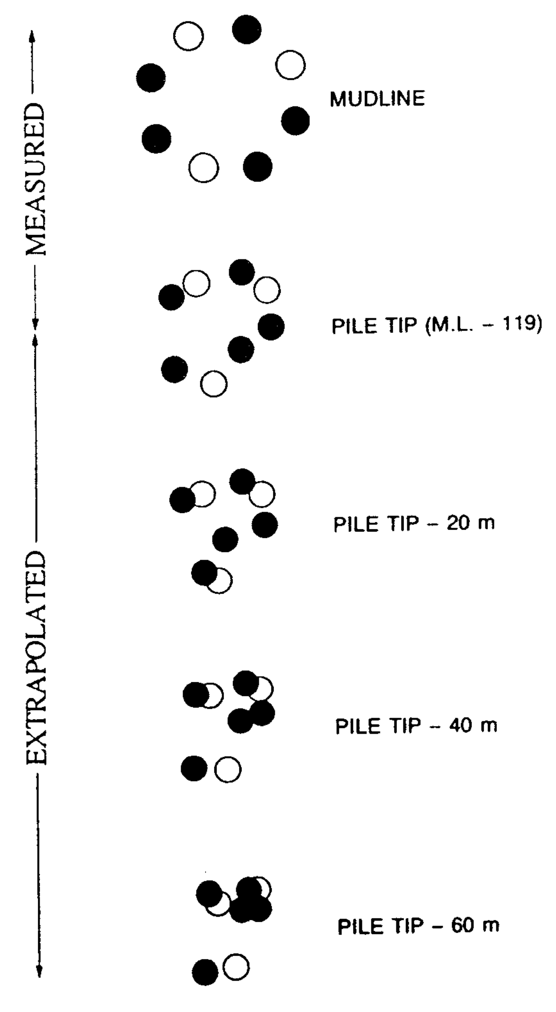

FIGURE - 8
TYPICAL PILE GROUP SURVEY

INSTALLATION OF THE BELLS

The installation of the bells beneath a gas producing platform required the evaluation of the load transfer during the construction process. The design needed to account for the condition in which high stresses are locked in one or more piles due to cyclonic activity during the construction process.

To ensure that the maximum loads on any individual bell were within the acceptable limits, the pile tip stiffness for all the existing piles needed to be increased by one of the following techniques. These were evaluated during the testing programme.

Pressure Grouting

Multiple pressure grouting could be employed to consolidate the formation. This technique was tested in a field trial but was unsuccessful.

Improving the Founding Layer Strength by Cement Impregnation.

Laboratory tests were carried out to evaluate the potential for cement impregnation of the calcarenite formation below the pile tips. However, even ultra-fine cement particles were found to cake on the calcarenite surface and consequently this option was also dropped.

Chemical Grouting

There were no chemical grouts with an acceptable track record except for EPOSAND, a Shell propriety product used for sand stabilisation in hydrocarbon wells. EPOSAND increases the inter-particle cementation with minimal effects on the permeability. This option was tested in the field and was successful. It is described further in a later section of this paper.

FALL-BACK OPTIONS

During the evaluation of the NRA foundation capacity, it was recognised that there was a potential that the primary remedial options would not provide the necessary pile capacity. As a result two options were studied in detail, as fall-back options:

The Freezing Option

An independent team was set up to evaluate the freezing option. Though this option had previously been discarded the project team were able to show that this concept was viable and was capable of mobilising the highest foundation capacity. Extensive laboratory tests were carried out to evaluate the frozen material properties and any effects that repeated freezing/thawing may have on the calcareous materials.

The freezing concept relied on the creation of a massive frozen pier around the lower half of the existing piles. This pier would be capable of mobilising both the friction and the end bearing on the pier.

Numerical analysis estimated that when equilibrium was reached the system could sustain a significant shut down period without the loss of capacity.

The project team, however recommended that the concept should be tested on land before implementation offshore. The time required to undertake a large scale land trial, and the reliance on an active remedial solution, were the primary reasons for this option not being adopted.

The EPOSAND Option

As discussed earlier EPOSAND was evaluated as there were no other proven chemical grouts for offshore use.

Laboratory tests on impregnated calcareous material showed that the strength increased by a minimum factor of three. Moreover, the impregnation transformed the material response from brittle to ductile. However, the process for impregnation required the samples to be flushed with fresh water, then with alcohol, then with xylene and finally with the epoxy resin. A field trial was carried out through the tip of a pile. Core samples were recovered both from the impregnated pile and from adjacent piles. These proved that the reliable impregnation zone extended radially approximately 2.0m from the injection point.

EPOSAND had not been used previously in shallow, low temperature zones (28 degrees centigrade) hence its long term reliability could not be guaranteed.

The response of the formation with an impregnated zone beneath the pile tips was significantly stiffer than that of a bell group, due to the ductile shear response of the impregnated material. In the final solution however, EPOSAND was used as a construction aid only.

THE CHOSEN SOLUTION

Figure 9 shows the modified platform foundation.

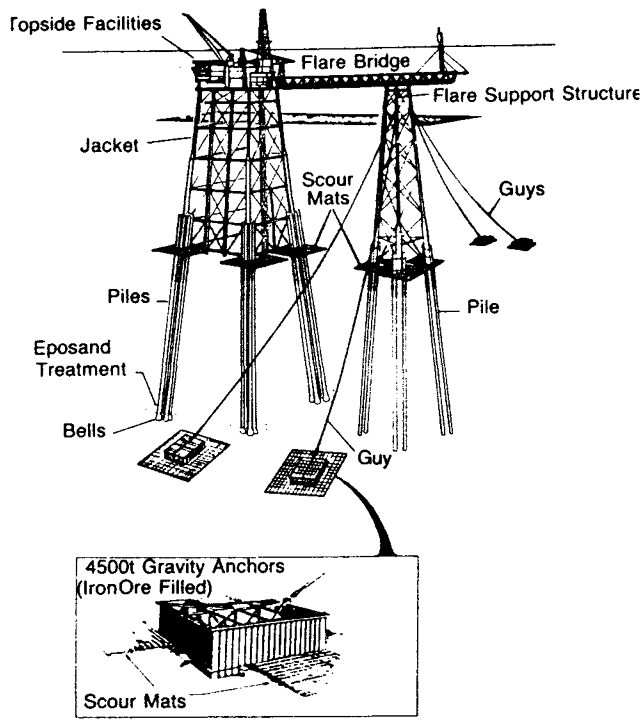

FIGURE 9 - MODIFIED PLATFORM

The remedial solution finally adopted comprised:

For the main platform:

The impregnation with EPOSAND of all the pile tips and the layer through which bells would be constructed, and then the construction of four 4.65m diameter bells in each pile group.

For the flare support structure (FSS):

The FSS was guyed in the East-West direction with stiff guy wires fixed to the sea bed by gravity anchors and the bridge supports were fixed to provide support in the North-South direction. Grout plugs were also placed at the tips of all the FSS piles.

The NRA foundation project was completed in November 1987 and full certification (for design loads associated with 100 year return period storms) was granted in March 1988 when all the temporary works required to support the platform modifications were removed.

In April 1989 cyclone Orson passed just to the west of NRA. The central pressure measured during this storm was 905hPa at NRA. Orson was the most intense storm yet recorded in the Australian region and among the most severe worldwide. The maximum wave associated with cyclone Orson was estimated at 21m at NRA, such waves being slightly in excess of the predicted 100 year return period storm.

Detailed platform surveys after cyclone Orson and the recorded loads in the guy wires indicate that the platform foundations behaved elastically. This suggested that the design assumptions were conservative as these had predicted settlements of the order of 200mm under such load conditions.

FOUNDATION DESIGN FOR THE GOODWYN "A" PLATFORM

Design activities commenced for the Goodwyn A (GWA) platform before the completion of the foundation activities on NRA. The preliminary site investigation at GWA showed the formation to be very similar to that of NRA with approximately the top 90 m comprising of relatively weak calcareous sediments underlain by more competent calcarenite varying in strength from strong to weak.

Both the end-bearing bell option and the insert pile option were considered for GWA with the insert pile being selected as the most probable foundation for the following reasons:

(1) There was no restriction on the size or number of insert piles on a new platform.

(2) Insert piles provided a stiff response in both tension and compression.

(3) Insert piles are the most traditional foundation system for calcareous sediments.

(4) The adoption of the insert piles as the most likely foundation system did not jeopardise the ability to adopt the end bearing option if it was found to be more economical at a later date.

To obtain reliable data for the design of the insert piles a programme of large scale grouted section tests was carried out.

DEVELOPMENT OF TEST PROGRAMME

The test programme targeted the areas of concern raised during the NRA field test programme, namely the effects of pile diameter on the displacement to peak friction and the effects of cycling both one-way and two-way, pre- and post-peak.

With the experience of the NRA grouted section tests it was decided to undertake the tests on land if a suitable test site could be found. Fortunately, during the review of geological work for NRA, it was pointed out that the fabric of some limestones in South Australia was similar to that of the North West Shelf calcarenites.

Following extensive site investigations, a site at Overland Corner on the Murray River was selected for the test programme.

A comprehensive site investigation was undertaken which effectively duplicated the offshore programme. Core samples were recovered and cone penetrometer, self-boring pressuremeter tests and small scale anchor pull tests were carried out. A complete suite of physical strength tests and small scale model grouted pile tests were carried out to ensure that sufficient data was available to enable the correlation of the material properties at Overland Corner with the offshore site properties.

The stratigraphy at this site comprised 13m of overburden sands and clays overlaying 19 m of calcarenite of varying strength followed by 15m of uniform low strength calcarenite. This layer was chosen as the test zone with the reaction pile being founded in the top 25m. This left a sufficient gap to ensure that the test elements were not affected by the reaction pile.

Development of the test programme

To test the effect of diameter on the displacement to peak friction load, three diameters were selected: 400mm, 950mm and 2000mm. Initially three test were proposed for each diameter with a single static test and two cyclic tests. However, due to cost constraints the programme was reduced to three tests with the smallest diameter and only one test in each of the larger diameters. In addition, a long flexible pile test was carried out to model progressive degradation along the pile length.

As the revised programme was the absolute minimum needed to provide design criteria, spare test sections of all the sizes were prepared to enable the repeat of any test, if the test results were in doubt.

Test Organisation

At the start of the programme a steering committee was formed of the Woodside engineers responsible for the test programme, the geotechnical consultant's project manager and representatives from Sydney University, the University of Western Australia and Division of Geomechanics of CSIRO. All steering committee meetings were also attended by the contractor's project manager.

For the duration of the field test programme two teams were formed, one to set up the test and the other to perform the tests.

The cost of the grouted section test programme was approximately $5.5 million, which included the field work, site investigation, laboratory tests and analysis of the test data. Additional costs were incurred by in-house staff and the primary geotechnical consultant.

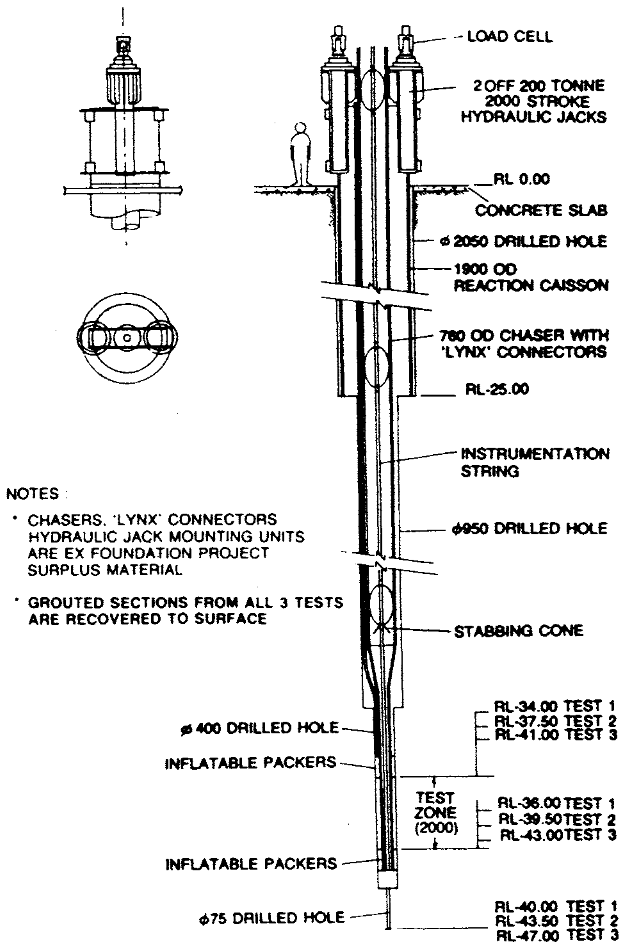

FIGURE 10 -
ϕ400 SHORT GROUTED SECTON TEST

Test Set up

The general arrangement of the 400 mm test is shown in Figure 10. Special care was taken to ensure that the reaction casing was sufficiently stiff to ensure full control of the test post-peak. The test zone was confined with inflatable packers. The design brief required back-up instrumentation for both load and displacement measurement with the primary system being located down hole.

The load was established by calibrated load cells at ground level, and two levels of vibrating wire and resistance strain gauges just above the test section.

The displacement was evaluated by a down-hole 'TEMPOSONIC' displacement transducer and tell-tale rods connected to a manometer at ground level.

The extension and load in the long test section was evaluated by DCDT's and strain gauges placed at one metre intervals.

Pore pressure and grout pressure transducers were also placed in all the test sections, with the exception of the long test where only grout pressure transducers were used.

Control during the test

A programme was developed to control the test which enabled the projection, on-line, of all the primary functions on a VDU. In addition, back up plots of all the prime functions were also obtained on line.

Test results

Figure 11 shows the effect of pile diameter on the peak stress for the three test diameters.

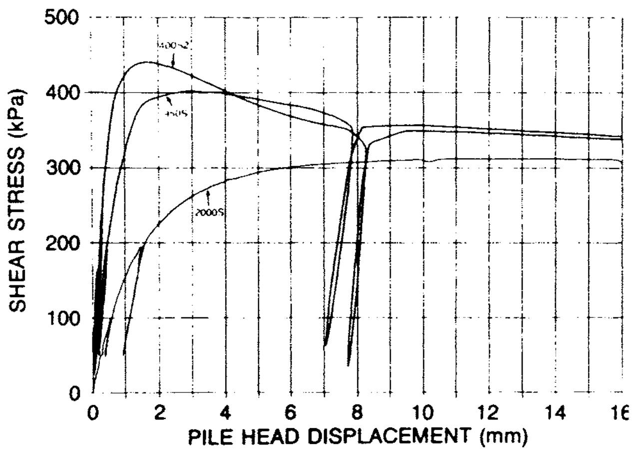

FIGURE - 11
GROUTED SECTION TEST SIZE EFFECTS

The conclusions from the tests were that the displacement to peak was smaller than predicted and that this displacement was dependent on the test element diameter. The level of peak shear stress was also affected by the size of the test. However the monotonic residual shear stress was significantly higher than predicted.

The cyclic tests indicated that pre-peak two-way cycling could be sustained to stress values approaching 75 % of the peak stress and that friction was dramatically lost if load cycling occurs in zones that have already experienced peak shear stress.

As a result of this programme, the RATZ programme was modified to capture the fundamental behaviour obtained from this test programme and load transfer curves generated by the revised formulation compared well with the results of the 400mm long test, as can be seen in figure 12.

The GWA foundations will consist of five primary piles 2.6m OD extending through the top soft sediments to a depth of 116m below the sea bed. Four insert piles will then be constructed through any of the five primary piles. The insert piles are 2.3m OD and extend 64m below the primary piles.

Five primary piles are needed for lateral stability and to provide a contingency for the installation of an additional insert pile if problems are encountered during construction. GWA piles will be nearly vertical with a slight rake to minimise the potential for pile tip convergence.

LESSONS LEARNT

(1) It is important to understand the fundamental characteristics of calcareous sediments and how they affect the mechanical response.

(2) The use of empirical design methodology developed for other areas should be verified before use.

(3) In new areas, all the design assumptions should be challenged.

(4) Responses to all the "what ifs" should be prepared during the design process.

(5) In critical situations, project management pressures should not be allowed to blinker vision.

FUTURE WORK

There is a need to develop a more cost effective foundation system since drilled and grouted piles are very expensive. The driven and grouted pile system appears to offer the highest potential at present.

Current pile designs do not address the long term effects of creep and low stress cycling and consequently there is a need to continue research in these areas.

Prediction of the performance of shallow foundations also require further research to evaluate the effects of cyclic loading and settlement on uncemented weak calcareous silts and muds.

ACKNOWLEDGMENT

The success of the work undertaken for the North Rankin "A" foundation project and the Goodwyn "A" foundation design is undoubtedly largely due to the massive team effort from local and overseas consultants, local universities and the CSIRO Division of Geomechanics. The quality, effort and dedication of all those who took part in these activities is gratefully acknowledged. Special acknowledgment is however due to Woodside's Geotechnical external reviewers Professor H Poulos and Mr R Jewell for the many useful discussions and assistance with the formulation of the work programmes.

REFERENCES

Full details of the North Rankin Foundation Project are published in Volume 2 of the Proceedings of the International Conference on Calcareous Sediments held in Perth, 15-18 March 1988.

This was edited by R. J. Jewell and M. S. Khorshid and is published by A. A. Balkema.

Results from the laboratory and field testing for the Goodwyn A foundation design work are yet to be published.

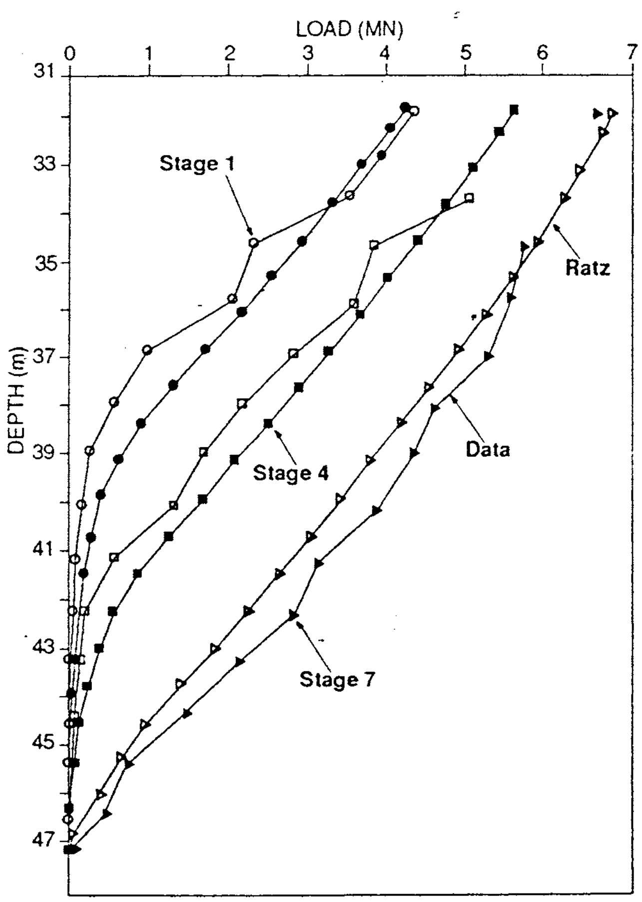

FIGURE 12
400L - LOAD DISTIBUTION PLOT

HARRIS DAM SLURRY TRENCH : DESIGN AND CONSTRUCTION

by C E Bradbury*

SUMMARY

A slurry trench cutoff up to 27 metres in depth through residual granitic soils, has recently been constructed as part of the seepage control measures for the Harris Dam, Western Australia.

The slurry trench had to accommodate foundation settlements of about 500 mm and still remain relatively impermeable.

Extensive laboratory trials were carried out to develop a cement-bentonite slurry mix that satisfied these requirements. Close monitoring during construction confirmed that these design criteria were largely achieved in the finished product.

This paper describes the design and construction of the Harris Dam cutoff. A companion paper (1) describes the investigation and design of the slurry mix used in construction of the trench.

INTRODUCTION

The Water Authority of Western Australia is constructing a dam across the Harris River in the Darling Ranges. This is at a point 10 km north of the town of Collie, in the State's south-west. The dam will supply the Great Southern Towns Water Supply system in which it will replace the existing Wellington Dam. Increasing salinity levels in the latter will restrict future use of its water to irrigation purposes.

Harris Dam is essentially an homogeneous earthfill embankment, 35m in height and 430m in crest length. A cross-section of the dam is shown on Figure 1. The dam is founded on in situ soils, generally 20 to 30m thick, formed by weathering of the underlying, predominantly granitic bedrock.

Preliminary conceptions of the site saw the foundation soils as being:

either i) relatively impermeable and likely to sustain high pore pressures during embankment construction.

or ii) permeable and requiring a positive cutoff to limit underseepage. Detailed site investigations were carried out by Authority staff to resolve these and other design issues.

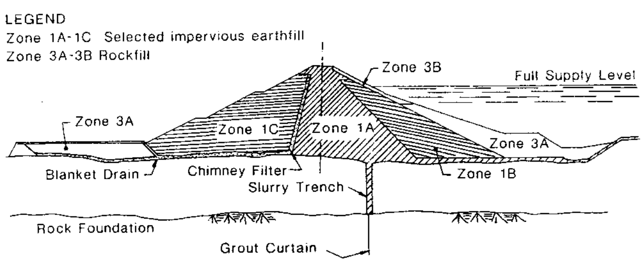

FIGURE 1 : Harris dam embankment cross section

Paper delivered to Western Australian Group of Australian Geomechanics Society at Technical Meeting on March 13th, 1990.

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SITE DESCRIPTION

The dam foundations comprise up to 30m depth of largely residual granitic soils, overlying granite bedrock dissected by dolerite dykes. Whilst completely and highly weathered materials predominate in the soil profile, less weathered materials exist locally either as occasional corestones or as thin zones above the slightly weathered to fresh bedrock. The bedrock is shallower at the abutments and outcrops on the left abutment to provide an unlined chute for the spillway. The residual soils are mostly silty and clayey sands and silts. Laterite is present as a thin mantle of slopewash gravels on the valley sides and as a bonded, ferruginous layer, 1 to 2m thick, just below the surface, mainly in the valley floor. A typical log and CPT trace of the soil profile are shown on Figure 2.

Measurements of soil permeability varied widely with the test method:

i) laboratory constant head tests averaged 1.9×10^{-7} m/s

ii) packer tests, typically on 1.5m long sections of borehole, averaged 3.3×10^{-6} m/s

iii) 'pump out' tests gave average values of 2.0×10^{-5} to 3.3×10^{-5} m/s over aquifer depths of about 20m.

The results of the 'pump out' tests were considered to be the most reliable as the test is full scale and therefore includes macro soil fabric effects such as relict joints, fissures, cracks, root channels and quartz veins. Furthermore, 'drilling induced' smearing on the sides of the drillhole, has less of an effect in 'pump out' tests, compared to packer or 'pump in' tests.

Packer test measurements of permeability in the underlying bedrock varied considerably from the impermeable ($<10^{-7}$ m/s), to greater than the capacity of the test equipment ($>10^{-2}$ m/s). Average values however were of the order 4×10^{-6} m/s and fall within the 'groutable' category.

SEEPAGE PROBLEM

A 2-dimensional seepage flow net analysis assuming a full reservoir condition and isotropic foundations, indicated that up to 3756 m^3/day could be lost in underseepage. This is equivalent to 8% of the reservoir's annual yield or about \$1.6M in terms of the capital cost of the Harris Dam. Seepage control measures could obviously reduce this loss of

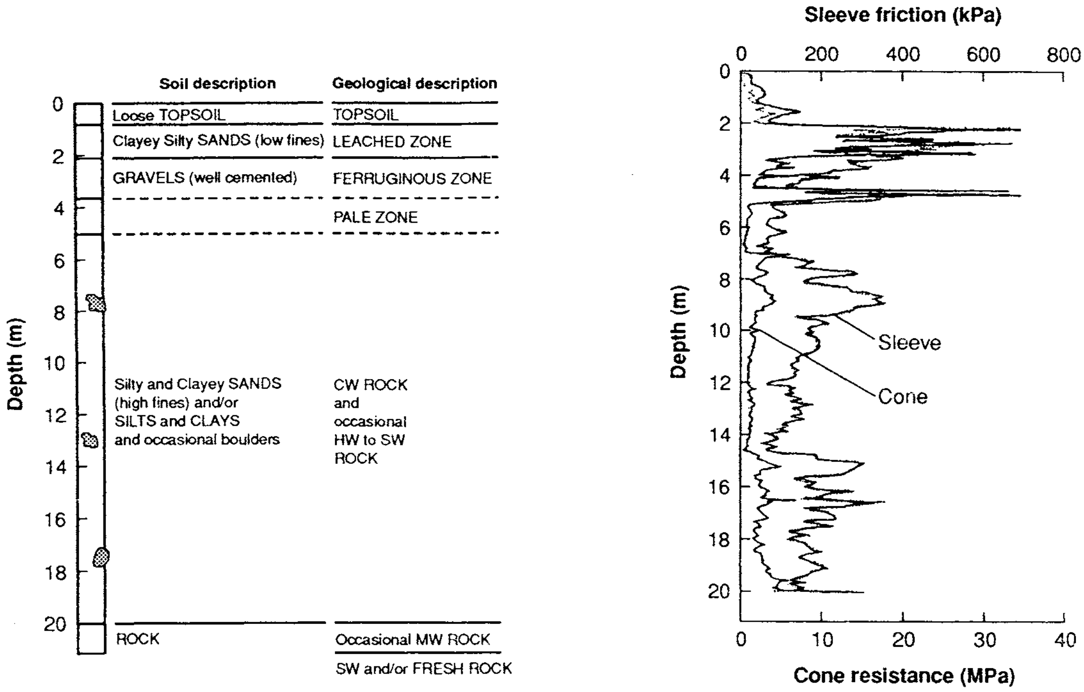

FIGURE 2: Typical soil profile in valley floor

water resource. Such measures would also permit major economies in the design of the downstream drainage blanket and/or embankment stability.

Various methods of control were considered and assessed as follows:

a) a clay filled cutoff down to bedrock in opencut excavation was likely to be expensive with problems of river diversion, dewatering, slope instability, stockpiling and disposal of unsuitable material.

b) a grout curtain through the residual soils would require the use of expensive chemical grouts and would not be as cost effective as more positive forms of cutoff.

c) an upstream impervious blanket would be limited in its overall seepage reduction potential and would be too sensitive to the presence of permeable zones such as dykes or faults, extending under the dam.

d) a slurry trench would provide a positive cutoff in the form of a continuous wall, extending some 25m down to bedrock. It would be constructed across the full width of the valley in a series of overlapping panels. Most of the trench could be excavated by grab with only the more resistant material, found locally, requiring first breaking up by chiselling. Costs were expected to be about $250/m^2$.

The slurry trench cutoff was selected as being the most cost effective method of seepage control within the soil profile. The technique avoids the site disruption and risks associated with the large opencut excavation and provides a far more positive and effective cutoff than either soil grouting or an upstream impervious blanket. To complete the seepage control measures, the slurry trench was to be 'underpinned' by a cement grout curtain within the bedrock. This was considered essential to prevent seepage bypassing the slurry cutoff through more permeable features in the underlying bedrock e.g. dykes and shear zones etc.

CUTOFF DESIGN

Whilst several slurry cutoffs have been constructed in Australia over the last twenty years, they are not common and of necessity their designs, are to some extent, site specific. (2)

There are two principal methods of slurry cutoff construction normally used. In one, bentonite slurry is employed to support the trench during excavation, following which it is displaced by a plastic concrete or soil bentonite slurry, tremied into the bottom of the trench. In the other, cement bentonite slurry, with or without retarders, is used throughout the operation, both for excavation support and final construction. The latter method was adopted for the Harris Dam as it is quicker and cheaper than the other method and avoids the risk of remnant bentonite contamination of the finished wall.

Efficiency of the slurry trench cutoff was examined by flow net analysis for various foundation and slurry permeabilities and trench widths. The results, which were later confirmed by finite element analysis, are given in Table 1 for an 800mm wide trench. For greater trench widths, the seepage will reduce proportionately.

The results clearly indicate that the key factor in reducing seepage is the permeability of the hardened slurry material. If a figure of 1×10^{-8} m/s could be achieved in the field, as some references suggest (3,4,5), then the seepage could be reduced by as much as 95%.

The trench width of 800mm was selected somewhat arbitrarily. It had to correspond to available clamshell grab sizes. In addition it had to limit hydraulic gradients across the section whilst providing some allowance for reduced width due to

Foundation permeability (m/s)	SEEPAGE (m^3/day)		
	No cut-off	Plus slurry cut-off	
		Slurry permeability (m/s)	
		$k = 1 \times 10^{-7}$	$k = 1 \times 10^{-8}$
2×10^{-5}	1832	773	129
4×10^{-5}	3756	966	132
1×10^{-6}	94	N/A	N/A

Table 1: Seepage estimates for 800 mm wide trench

misalignment at joints between panels.

The slurry trench was located about 20m upstream of the dam centre line to coincide with shallow bedrock. In this position the trench would still be subject to substantial embankment loads in the centre of the valley, where the seepage head conditions would be greatest. At the abutments, the trench was terminated on shallow rock ridges that extended above full supply level.

Throughout the length of the cutoff, the trench was intended to be keyed 500mm into underlying slightly weathered to fresh rock to provide a continuous impervious contact. A view of the trench and grout curtain in elevation is shown on Figure 3.

The grout curtain was to extend down some 10 to 20m below the base of the trench and was offset 1.2m upstream of the trench, rather than being on the same line. This was done to avoid subsequent drilling of grout holes through the trench disturbing the slurry or, if pregrouting, chisel operation at the bedrock contact, disturbing the set grout. Contact between the trench and grout curtain was to be sealed by TAM (tube a manchette) grouting over the bottom 2m of the soil profile, using the relatively fine, Type SA cement.

DESIGN REQUIREMENTS OF SLURRY

Although the target permeability for the slurry was known, other properties of the hardened slurry needed to be defined. Clearly the material needed to be stable, erosion resistant and largely free of cracks. In addition however, the stress-strain characteristics of the slurry were important.

During construction of the 35m high dam embankment, the underlying foundation soils were expected to compress by up to about 500mm of largely immediate settlement. The slurry trench needed to have similar compression characteristics. If more compressible than the adjacent soils, the slurry material would yield, shedding overburden load onto the adjacent stiffer soils. In this situation, the trench material would be vulnerable to hydraulic fracture under reservoir head. Equally, if too stiff, the slurry trench would tend to attract load and could become over stressed, leading to cracking and associated leakage.

Ideally, therefore, the slurry material needed to have stress-strain characteristics identical to those of the surrounding soil and to remain relatively impermeable over the operating range of strains. In practice, as might be expected, the in situ soils at the Harris Dam site are not homogeneous like the man made slurry, but vary greatly in their compression characteristics. A perfect match between the strength properties of the slurry material and those of the in situ soils was therefore impractical. The best one could hope for, was to select a slurry modulus that minimised differential settlements between the slurry and adjacent soils, particularly in the upper, more critical sections of the trench. In addition, however, to cope with the local high shear stresses that would be generated at the slurry soil interface, the slurry material needed to be capable of deforming without cracking over a wide range of strain.

The following properties were finally selected as the design criteria for the slurry mix:

i) laboratory permeability$\leq 10^{-8}$ m/s

ii) erosion resistance of ND1 in the pinhole test

iii) durability - remain stable

iv) elastic modulus of 40 MPa ± 10%

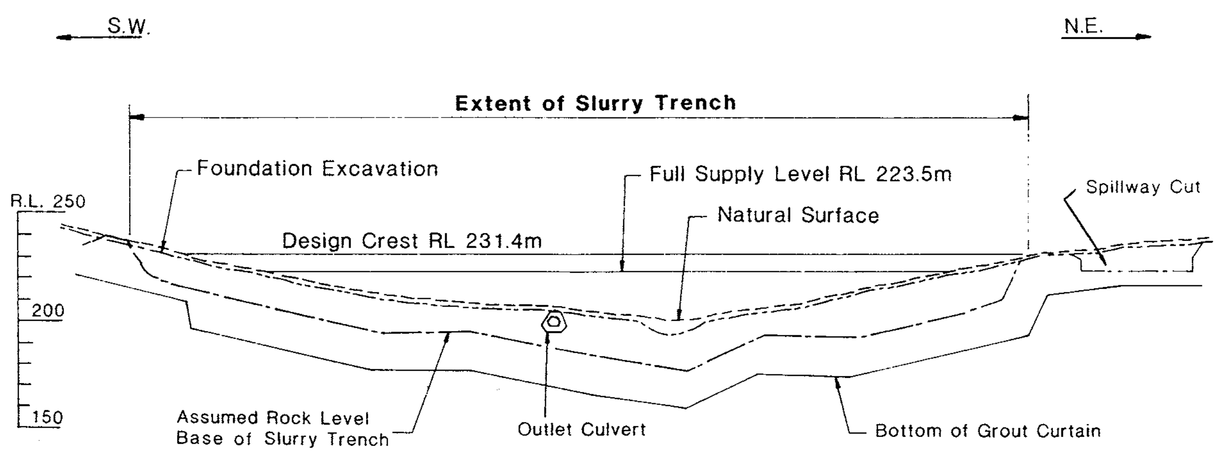

FIGURE 3 : Section along slurry trench

v) UCS ≥ 200 kPa

vi) deformability - elasto-plastic response up to a strain of 10% i.e. no cracking.

To be certain these criteria could be met in practice and to gain a better understanding of slurry properties and the effects of different cements, the Authority carried out a programme of trial mixes in the pre-contract design period. The results of this work, along with those of subsequent contract trial mixes, are discussed in the accompanying paper (1).

AWARD OF CONTRACT/PROGRAMME

Separate contracts were let for the cutoff construction including curtain grouting and for the embankment construction. Although both contracts started at the same time, in August 1988, the slurry trench/grouting works was programmed to be finished by April 1989, leaving the following summer for construction of the main embankment.

Five 'Schedule of Rates' tenders were received from a select list of contractors, at least three of whom had agreements with European specialist diaphragm wall contractors. A contract was awarded to the lowest tenderer, GFWA Pty Ltd. of Perth, for $2.6M.

TRIAL MIXES

Since fluid properties of the slurry are critical to panel construction, the contractor was made responsible for mix design. The contract therefore, included for carrying out a series of trial mixes to determine a mix that satisfied both construction and design requirements. This work was carried out by a slurry specialist from SIF Bachy, GFWA's parent company with the assistance of staff from the Water Authority's Engineering Research Station (ERS).

The contractor's tender proposed a blend of 35% OPC and 65% crushed granulated blast furnace slag (BFSC) as the cementitious material for the slurry. Pre contract investigation of slurry mixes at ERS, using various cements, including OPC with and without blast furnace slag and fly ash, had already identified the strength advantages of blended cement. The contract trial mixes were therefore able to zero in on the final design mix without investigating a wide range of trial mixes.

From a construction standpoint, the following properties of the fluid slurry were important:

- Viscosity. This is largely determined by the bentonite content, to give a stable mix that prevents cement separation, limits bleeding and yet is pumpable and allows settling out of coarser excavated material.
- Density. This must be sufficient to support the sides of the trench.
- Bleed. Significant release of water from the slurry is undesirable.
- Setting Time. This must be sufficient, with or without retarders, to allow full excavation of the trench panel, including any keying into rock.
- Filter Losses. This is water lost from the slurry into adjacent soils depending on their porosity. More significant at depth, it leads to the formation of a filter cake which increases the impermeability and strength of the outer skin of the cutoff wall.

The trial mixes indicated that increasing cement contents reduced the permeability of the slurry but tended to increase the brittle or non-plastic character of the material. Furthermore that soil 'take up' by the slurry, when placed in the ground, would probably lead to increased density, strength and modulus of the hardened slurry and a consequent decrease in deformability.

The design mix adopted contained the following:

225 kg/m^3 of blended cement
30 kg/m^3 of bentonite
913 l/m^3 of water
0.5 to 2.0% by weight of cement of retarder

CONSTRUCTION

Prior to any trenching, a series of boreholes were drilled at 5m centres using wash boring techniques to better define the bedrock profile. Any anomalies were then investigated by diamond coring and a clearer indication gained of both the likely trench termination depths and the presence of floaters or corestones.

The slurry trench was constructed in alternate primary and secondary panels, 2.7m long, corresponding to the dimensions of the grab. Primary panels were excavated first then secondary or infill panels, followed either the same day or 24 hours later, but in 3 or 4 cases, several weeks later. The secondary panels overlapped and excavated into adjacent primary panels by at least 350mm at the surface. The primary panels served as a guide or control for excavating the secondary panels. However for all panels, alignment and stability of the upper section of the trench was largely maintained by excavating between a steel guide frame, laid on the surface. Minimum overlap allowed between panels was 100mm longitudinally and 600mm laterally. A schematic view of panel construction is shown on Figure 4.

Excavation of the completely and highly weathered material which made up most of the panel, was carried out using a KL 800, 800mm wide by 2.7m long, 8 tonne, clamshell grab, twin cable operated by either an RB38 or RB61 crane. Excavation of rock to form the key into bedrock or to remove corestones or hard rock layers was carried out using a 9 tonne chisel to break up the material for later removal by the clamshell.

Construction of the first few panels quickly revealed that chiselling of the bedrock to establish a 500mm deep key was largely impractical. Not only was the operation very time consuming, it was adversely affecting the slurry mix. Repeated chiselling disturbed the trench sides causing increased loss of water and increased soil 'take up' by the slurry. In turn, workability of the slurry was greatly reduced, promoting difficulties in cleaning out the trench base. Density of the slurry was increased accordingly, with ultimately a reduction in deformability. The requirement for a key was therefore relaxed to one of establishing good contact with fresh rock over the width of the panel.

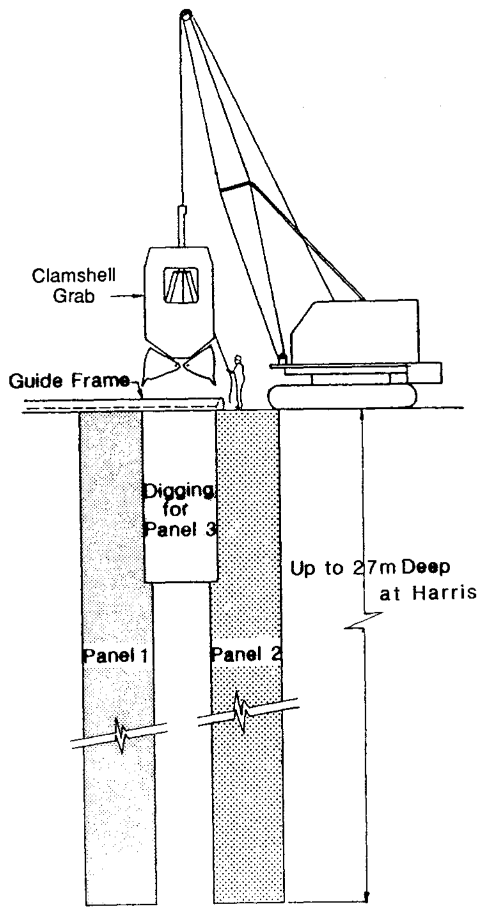

FIGURE 4: Sequence of slurry trench panel construction

Corestones or floaters of relatively unweathered rock were encountered at various levels in the completely weathered soil profile in about 20% of the panels. Most of these were satisfactorily dealt with by breaking through with the chisel. Where this was impractical, the trench was locally diverted around the obstacle.

At one location, high on the left abutment, extensive corestones could not be avoided and over 40kg of explosives were detonated in eighteen drillholes in an attempt to break up the offending rock. Although successful at shallow depth, the blasting made little impact on a deep seated stronger layer. The trench was therefore founded on this hard layer and the underlying 'window' of completely weathered material was later sealed off by TAM (tube-a-manchette) grouting with microfine cement.

The ground conditions and in particular, the floaters, presented some major headaches for the contractor when the clamshell grab became stuck or locked in the ground. This occurred several times throughout the job and techniques for freeing the grab ranged from the use of small explosive charges, special jacking equipment to a 6m long excavation shield. One of the grabs resisted all attempts to free it and, in the end, had to be abandoned.

Construction took longer than anticipated due in part to industrial stoppages and to delays in recovering grabs held fast in the ground. Excavation commenced in November 1988, working uphill on the west bank of the river and finished up on the east abutment in June 1989. Production averaged one primary or two secondary panels per day and peaked at four panels per day in shallow working, high up on the abutments. Panel depths reached 27m in the riverbed area but averaged around 15m. A total of 177 panels were constructed which provided an overall surface area of 8100 m2.

QUALITY CONTROL AND RECORD TESTING

The following parameters were tested daily as the principal means of quality control of the slurry: density, viscosity, pH, filter cake thickness, filter losses, bleed and sand content. Predictably the character of the bentonite 'master mud' (the bentonite: water mix before the addition of cement) and the slurry from the mixing plant remained relatively constant throughout the job. Slurry from the trench, however, was far more variable because of differences in the age of the samples, the ambient temperature and the amounts of in situ soil 'taken up' by the slurry.

In addition samples of both fluid and hardened slurry were taken periodically from the trench for checking against the design criteria.

Laboratory testing of these samples revealed the following:

- permeabilities close to target values
- wide variations in elastic modulus
- elasto-plastic behaviour generally up to 8-10% strains
- increased brittle behaviour with increased soil content

Typical values recorded of the design and construction properties of trench slurry are included in the Appendix.

CONCLUSION

Efficiency of the slurry trench can only be really tested under full reservoir conditions which may be experienced this coming winter. Foundation piezometers either side of the slurry trench and downstream V-notch seepage measurements

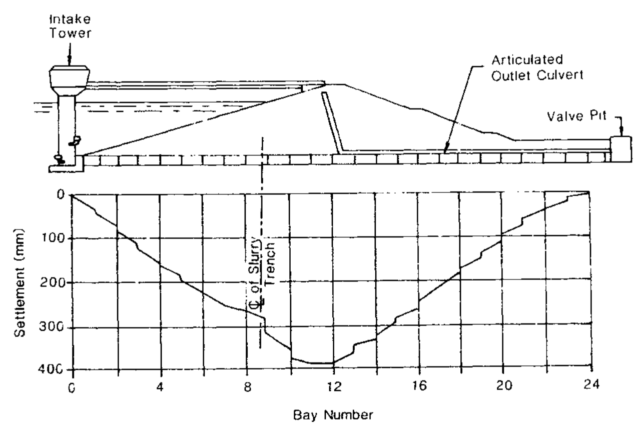

FIGURE 5 : Culvert settlements on completion of embankment

should assist in analysing the situation.

In the meantime construction monitoring has produced some promising results:

a) Measurements of settlement of the soil foundations due to embankment construction are remarkably close to predicted settlements, i.e. 475mm compared to 500mm.

b) the relatively high modulus values of trench slurry samples suggest the slurry may be somewhat stiffer than the foundation soils, certainly at shallower depths.

c) the relative stiffness of the slurry can also be seen in the settlement profiles of the articulated outlet culvert where the latter crosses over the slurry trench, in passing from upstream to downstream side of the dam. A noticeable kink in the otherwise uniform catenary shape of the settlement profile on Figure 5, marks the trench crossing. The result confirms that hydraulic fracture is an unlikely possibility.

d) pore pressures in the foundation have risen only marginally in response to embankment construction. This was expected and confirms the permeable nature of the foundation soils and provides further evidence of the need for a slurry trench.

ACKNOWLEDGEMENTS

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The views expressed herein are those of the author and may not necessarily represent those of any organisation.

REFERENCES

1. B. C. POTULSKI (1990). Harris Dam Slurry Trench: Slurry Mix Design. Aust. Geomech. J. 19, June, 1990.

2. B. A. FORBES (1986). Slurry Trench Cut-Off for Bucca Weir. ANCOLD Bulletin No. 75.

3. ICOLD Bulletin 51 (1985) Filling Materials for Watertight Cut-Off Walls.

4. S. A. JEFFERIS (1981). Bentonite - Cement Slurries for Hydraulic Cut-Offs. Proc. x ICSMFE, Stockholm. June, 1981.

5. B. P. CHAPUIS, J. J. PARE and A. A. LOISELLE (1984). Laboratory test results on self-hardening grouts for flexible cut-offs. Can.Geotech. J. 21, 185-191. 1984.

APPENDIX

KEY DATA:

(a) Materials used:

Cement: Blended slag cement from Cockburn Cement (65% blast furnace slag cement, 35% OPC)

Bentonite: Aquagel (natural unpeptized bentonite) from Baroid Aus.

Admixture: Daratard combined plasticiser/retarder from WR Grace Aus.

(b) Slurry Mix used

<u>cement grout</u>	) Slurry Mix:
cement 1190 kg/m^3	) Final Mix Proportions
water 598 l/m^3	)
<u>bentonite master mud</u>	) cement 225 kg/m^3
bentonite 37 kg/m^3	)
water 986 l/m^3	) bentonite 30 kg/m^3
<u>Daratard</u>	water 913 l/m^3
0 to 2% of cement weight	

(c) Typical Design Properties of Site Slurry:

Test carried out on 100mm diameter by 200mm long specimens. Samples taken of both hardened slurry from trench and fluid slurry from trench or feeder lines.

Lab permeability: Under hydraulic gradient of 45:1 and effective confining pressure of 200 kPa.

maximum 8×10^{-8} m/s
minimum 0.2×10^{-8} m/s
average 2.5×10^{-8} m/s

Elastic modulus: Taken as secant modulus at 50% of the yield stress on CID triaxial testing under 200 kPa effective confining pressure.

maximum 130 MPa
minimum 13 MPa
average 67 MPa

UCS: typically 300 to 600 kPa on batch plant samples 500 to 800 kPa on samples from slurry trench

Elasto-plastic behaviour commonly up to at least 5% strain, often 8 to 10% strain in CID triaxial testings as

above, under strain rate of 0.1% per minute.

(d) Typical Construction Properties of Site Slurry:

Refer to table 2 below:

Source of sample	Density g/mL	Viscosity sec	pH	Filter cake mm	Filtrate loss mL	Free water @2hrs	Sand content %vol
Bentonite master mud	1.02	31	8	1	17	-	-
Slurry mix at plant	1.17	41	13	-	-	2 mm	-
Slurry mix from trench	1.27	60	13	-	-	-	11

TABLE 2: Typical construction properties of site slurry

HARRIS DAM SLURRY TRENCH : SLURRY MIX DESIGN

by B. C. Potulski*

SUMMARY

The Water Authority of Western Australia has recently constructed a dam across the Harris River, 10 km north of Collie, a town in the south west of Western Australia. The dam is an earthfill type structure, 35 m high and 430 m long. The dam is founded on in situ soils, approximately 20m to 30m thick, overlying predominately granitic bedrock.

The design of the dam needed to address problems associated with the predicted large settlements of the structure (in the order of 500 mm), and also problems associated with possible seepage through the foundation soils with permeabilities of the order of 10^{-5} m/s.

The chosen solution to reduce the seepage through the foundation soils was the construction of a cement-bentonite slurry trench cut-off. The cut-off trench was supplemented by a grout curtain to minimise seepage through joints and fractures in the underlying bedrock.

The rationale behind the selection of a slurry trench cut-off as the means of reducing seepage through the foundation soils, as well as construction aspects, are discussed in the accompanying paper (1).

The process of designing a slurry mix capable of fulfilling the design and construction criteria is discussed in this paper.

INTRODUCTION

Slurry trench cut-off techniques have been used successfully overseas, and to a lesser extent in Australia. However, little material has been published, and guidelines on the design of slurry material are limited. This is understandable given that most solutions are site specific, depending upon site geology, foundation soil type and to a small extent on the type of construction employed (2).

The first step in designing the slurry material was to determine the optimal properties for the slurry material. In so doing the following aspects had to be considered:

- the slurry material's stress-strain and permeability-strain characteristics. In particular, interaction between the slurry trench and the surrounding in situ soil, had to be taken into account.
- construction of the slurry trench with regard to such details as the construction sequence and placement of the slurry.
- durability. The slurry trench must remain competent for the duration of the dam's design life.

The contractor was responsible for design of the slurry mix to meet specified design criteria. However, once optimal engineering properties for the slurry material were identified, a comprehensive laboratory testing program was carried out by the Authority to determine a range of suitable slurry mixes i.e. the slurry constituents and their proportions. This program was later supplemented by a series of tests designed by the contractor.

Stress-strain Characteristics

An ideal slurry material would have stress-strain characteristics identical to those of the surrounding soil, and remain practically impermeable over the entire range of strain through which it deforms. In the great majority of cases, as at Harris River, such an ideal slurry material does not exist. The geology of the site at Harris River is far too complex to enable a 'perfect match' between the strength parameters of the slurry material and those of the in situ soils.

With the realisation that a 'perfect match' between the properties of the slurry and the in situ soil would not be possible, thought was given as to what slurry material properties would result in a successful cut off trench. The requirements were that the slurry material:

a) have an elasto-plastic response over a wide range of strain so as to withstand large settlements (predicted to be in the order of 500 mm) without cracking.

b) have stress-strain characteristics similar to the 'average' characteristics of the in situ soil. In particular the Elastic Modulus of the slurry material needed to closely match the average value for the in situ soils. This requirement was to prevent large differential settlements between the slurry trench cut-off and adjacent in situ soils.

c) stay watertight under the applied stresses and resulting deflection.

Detailed analyses were undertaken to assess the optimal Elastic Modulus for the slurry material. Firstly, an 'average' profile of Elastic Modulus versus depth for the in situ soil was arrived at, via analysis of the results of triaxial tests, oedometer tests and several pressuremeter tests.

The next step was to assess how the slurry trench cut-off / in situ soils interaction is influenced by the slurry material's Elastic Modulus (Figure 1). This interaction was analysed through performing standard 1-dimensional settlement calculations to establish relative movements (Figure 2). Also a simple 2-dimensional finite element analysis was performed to give a guide to the possible stress and load transfer.

Paper delivered to Western Australian Group of Australian Geomechanics Society at Technical Meeting on March 13th, 1990.

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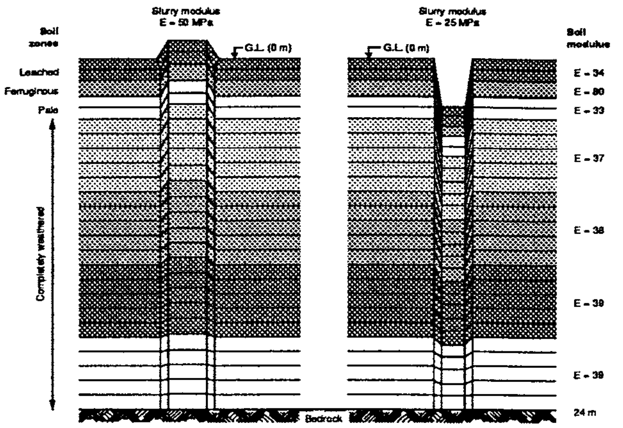


FIGURE 1: Simple model of slurry/soil deformation

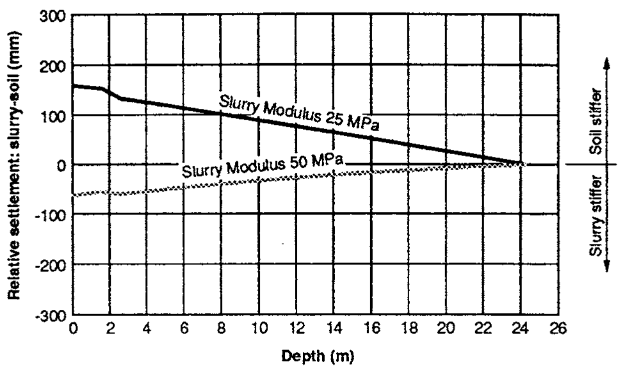


FIGURE 2: Deformation analysis of slurry/ soil settlement

The resulting design criteria established for the slurry material were (Figure 3):

Criterion 1) Elasto-plastic stress-strain response up to 10% axial strain during triaxial compression testing.

Criterion 2) Peak Unconfined Compressive Strength (Peak UCS) greater than 200 kPa.

Criterion 3) Elastic Modulus of 40 MPa (accurate to ± 10%).

Criterion 4) Permeability of less than or equal to 10^{-8} m/s over the full range of strain.

Construction Aspects

From a construction viewpoint, the slurry material must remain at an acceptable viscosity during handling and placement, have a 'setting time' compatible with the construction sequence, and suffer minimal bleeding (loss of water during hydration) or shrinkage.

The resulting construction criteria established by the contractor and the Authority for the slurry material were:

Criterion 1) Viscosity of 45 to 50 seconds Marsh to give a stable pumpable mix that prevents sedimentation of the cement in suspension and limits bleeding of the mix.

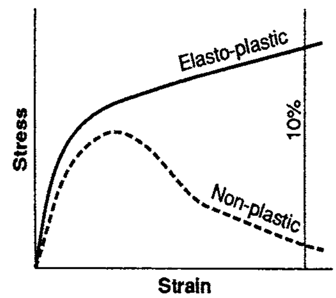
Criterion 2) Density of the slurry mix should be sufficient to stabilise the trench.

Criterion 3) Bleed of no more than 3%

Criterion 4) Setting time should be adequate to allow for panel excavation. Application of 0.5% to 2% of Daratard retarder extended the setting time up to 24 hours.

Criterion 5) Water loss from the slurry mix (filter losses) into adjacent permeable soil should be limited to prevent mix 'thickening' due to the decrease in water/cement ratio.

. Permeability $\leq 1 \times 10^{-8}$ m/s
. Elastic modulus = 40 MPa ± 10%
. Elasto plastic response up to strain of 10% i.e.



. UCS ≥ 200 kPa
. Pinhole class = ND1

FIGURE 3 : Design requirements of slurry

To satisfy the construction criteria, the slurry mix had to fall into a fairly narrow range as shown on Figure 4

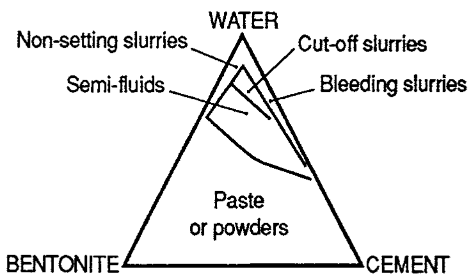


FIGURE 4 :Principal regions in the bentonite-cement-water slurries (3)

DESIGN OF THE SLURRY MIX MATERIAL

On the basis of published data, the decision was made to construct the slurry trench from cement-bentonite slurry, the major constituents of which are cementitious material and bentonite. It was recognised that the blend of Ordinary Portland Cement (OPC) and Blast Furnace Slag (BFS) had advantages over usage of OPC on its own (3). It allows the slurry to be worked for longer periods without affecting the setting properties (increase in excavation time). It also produces material with a higher strength and a lower permeability than mixes made with OPC alone. The same data indicated that a cement-bentonite slurry with a water :cement ratio between 2.85 and 5 could be suitable for the Harris Dam.

Literature on designing slurry mixes, with respect to the choice of constituent materials and mix proportions, contains very limited detailed information (2,3). Therefore it was decided to commence the testing program with a series of Unconfined Compressive Strength (UCS) tests on laboratory prepared samples of slurry material to quantify the influence of cementitious material type, cementitious content (per m^3 of slurry), bentonite content, and sample curing time on slurry strength.

Stage 1 - Initial UCS Testing Program

Different cementitious materials tested during the initial UCS test program were Ordinary Portland Cement (OPC), blends of OPC and ground Blast Furnace Slag (BFS), and blends of OPC and Fly Ash (FA). Cementitious contents of the samples ranged from 110 kg to 350 kg per m^3 of slurry. Bentonite contents ranged from 30 kg to 60 kg per m^3 of slurry, and sample ages at the time on testing ranged from 3 days to 28 days.

As shown in Table 1, UCS test results indicated that of the slurry samples tested, for given cementitious and bentonite contents, samples with cementitious material consisting of 65% BFS and 35% OPC had the highest peak UCS stress (i.e. were the 'strongest'). It is possible, indeed likely, that similar blends containing a higher proportion of BFS are even stronger. However such blends were not tested as it was felt that a stronger slurry material would be too brittle (non-plastic).

TABLE 1

Selected UCS Test Results

Cementitious Blend	28 day UCS Stress (kPa)
100% OPC	420
65% OPC / 35% BFS	1200
75% OPC / 25% FA	150
50% OPC / 50% FA	50

Note: Each sample consisted of 300 kg of cementitious material and 30 kg of bentonite per m^3 of slurry.

Furthermore and as expected, the UCS test results also indicated that slurry strength increased with increases in either cementitious content, bentonite content or sample age.

Primarily on the basis of the results of preliminary UCS testing and published data, it was anticipated that a blend of BFS and OPC would be appropriate for the Harris Dam cut-off trench. This selection also took into account that such a mix is desirable from economic and availability viewpoints, and that the trench construction contractor had experience with similar slurry materials.

Stage 2 - Triaxial Compression and Permeability Testing Program

The next stage of the testing program involved determining the mix proportions that would yield a slurry which satisfies the design criteria. This stage entailed performance of drained triaxial compression tests, permeability tests and additional UCS tests.

The principal variables investigated during the program were cementitious content and bentonite content, with the cementitious material of all samples consisting of 65% BFS and 35% OPC. The laboratory prepared samples tested had cementitious contents ranging from 110 kg to 300 kg/m^3 and bentonite contents ranging from 30 kg to 60 kg per m^3 of slurry.

The important properties of the slurry material are now looked at one at a time.

(i) Permeability

Satisfaction of the permeability design criterion proved to be the most important factor in designing the slurry material. The permeability test results are presented on a plot of Permeability versus Cementitious Content on Figure 5. The plot shows that slurry permeability decreases with increasing cementitious content, and with increasing bentonite content.

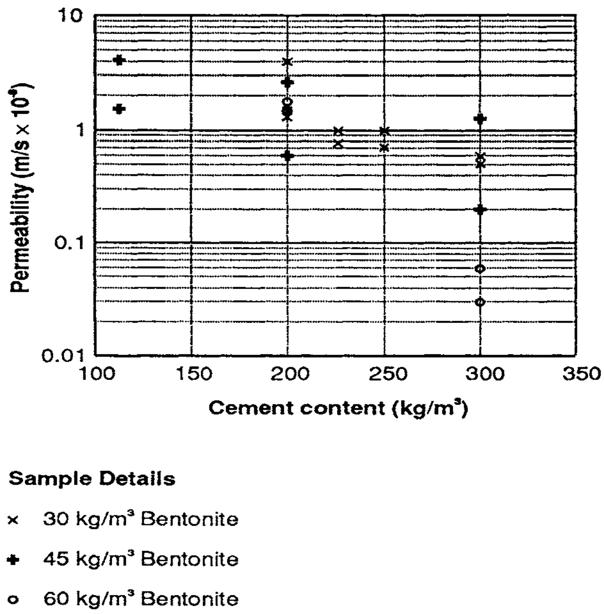


FIGURE 5 : Laboratory Permeability Test results

Furthermore, the results indicated that slurries with a cementitious content of 225 kg/m^3 (or greater) and a bentonite content of 30 kg/m^3 have a permeability of less than 10^{-8} m/s. Since it is desirable to use minimal bentonite in the slurry (so that slurry viscosity is optimised, as well as slurry cost) the results showed that a bentonite content of 30 kg/m^3 would suffice provided that cementitious content of the slurry is 225 kg/m^3 or greater. In fact it was found that within the tested range (30 kg to 60kg/m^3), the bentonite content had no significant effect on slurry material permeability.

The influence of sample strain on permeability was also investigated — the worry being that permeability increases with increasing strain. Results showed there to be no significant trend of increasing permeability with increasing strain, even after straining to 10% axial strain.

(ii) Post Yield Behaviour

The dependence of post yield behaviour on cementitious

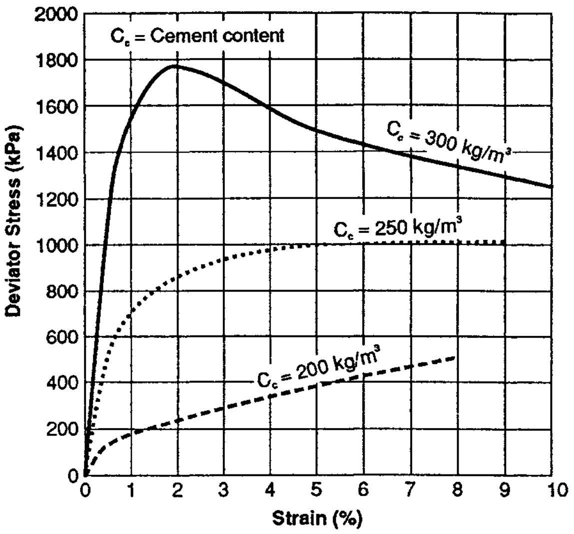


FIGURE 6: Triaxial (CID) test results — Deformability vs deviator stress (30kg/m³ bentonite)

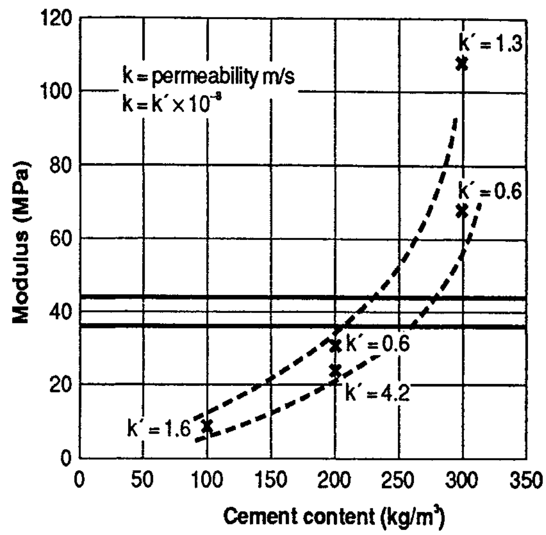


FIGURE 7: Triaxial (CID) test results — Modulus vs cement content (30kg/m³ bentonite)(5)

content of the slurry samples is illustrated on Figure 6 — which shows drained triaxial compression test results on a plot of Deviator Stress versus Axial Strain. Samples with cementitious contents of 250 kg/m³ or above, displayed brittle post-yield behaviour when strained to 10% axial strain. In contrast, samples with cementitious contents of 225 kg/m³ or less, displayed elasto-plastic behaviour, or post yield work hardening behaviour.

(iii) Elastic Modulus

Elastic Modulii of the samples tested in drained triaxial compression tests are shown in Table 2 below.

TABLE 2

Summary of Slurry Elastic Modulus Results

Cementitious Content kg/m³	Elastic Modulus MPa
110**	14 to 38
200	30 to 55
225	36 to 100
250	112 to 160
300	150 to 231

NOTE:

1) Samples had bentonite contents of 30 kg/m³, except for (**) for which bentonite content was 45 kg/m³.

2) Samples had been cured for 28 days prior to testing.

The results indicate that in order to satisfy the design criterion that the slurry must have an Elastic Modulus of between 36 and 44 MPa, the cementitious content of the sample would have to be of the order 200 kg to 250 kg/m³ (Figure 7).

Stage 3 - Durability Testing Program

Lastly, a program of drained triaxial compression tests and permeability tests was carried out on slurry material cured for 60 days and 90 days to determine whether the slurries' strength or permeability properties deteriorate with time. Undrained triaxial compression tests are also being carried out on these samples, in order to further enhance understanding of the slurry materials behaviour. The results of this test program are still pending and will be published once fully analysed.

Miscellaneous Tests

A number of tests were performed to determine the suitability of the slurry material from the erosion resistance and construction viewpoints. They were pinhole dispersion (an erosion type test), bleeding and shrinkage tests, viscosity and several tensile strength tests.

The samples tested for resistance to erosion were classified as ND1, the class for maximum erosion resistance by the pinhole dispersion test (AS1289). Laboratory bleed and initial shrinkage tests showed volume reductions of less than 2%. Testing of slurry viscosity was limited, as the contractor was confident that there would be no problems in pumping and placing the slurry. Initially, immediate viscosity and bleed tests were used to determine optimum bentonite content for best workability with minimum of free water. Once this optimum was established (approximately 30 kg/m³ of slurry), several mixes containing 150 kg to 250 kg of cementitious content were prepared and subjected to a full range of tests as were other (preliminary) mixes. Determination of slurry 'setting time' was left to the site engineers, as a suitable laboratory method could not be found (Shear vane testing was tried, but was unsuccessful due to inadequate resolution within the measured range).

Further details of laboratory testing program are given in the Appendix.

As mentioned before, work is continuing to provide data on durability of the design slurry. It is intended to report the results of this work at a later date.

CONCLUDING REMARKS

The testing program completed to date on 'laboratory prepared' mixes of cement-bentonite slurry material, as well as testing of the in situ 'as built' slurry trench material, allows us to draw the following conclusions and recommendations for future program of slurry mix design:

1. The construction criteria should be taken as a starting point for the possible range of mixes.
2. The design process should be based, whenever possible, on soil parameters derived from both laboratory soil testing and also on the in situ testing (penetration, pressuremeter or seismic testing).
3. Slurry material preparation which includes curing condition should mirror as closely as possible the 'as built' processes and procedures. Chemical composition or impurities of curing water and temperature may have significant effect on material properties measured during laboratory testing.
4. Once the possible slurry mixes are narrowed to the size of a 'handleable group', the tested materials should be subjected to a wide array of standard soil testing to provide comparative data and better understanding of material's 'soil like' behaviour and performance.
5. Particular attention should be paid to the quality assurance of the field mixes by adhering to established procedures. That is important because once in place, inferior cut-off trench material cannot be improved. Replacement of the inferior material is the only practical option.

ACKNOWLEDGEMENTS

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REFERENCES

1. C E BRADBURY (1990). Harris Dam Slurry Trench. Design and Construction. Australian Geomechanics Journal No 19 June 1990.
2. ICOLD Bulletin 51 (1985). Filling Materials for Watertight Cut-Off Walls.
3. S A JEFFERIS (1981). Bentonite - Cement Slurries for Hydraulic Cut-Offs. Proceedings X ICSMFE, Stockholm. June, 1981.
4. Harris Dam. Slurry Trench. Precis of Laboratory Test Results. Water Authority of Western Australia. November 1988 (internal report).
5. Y LACOUR (1988). Harris Dam River. Cut-Off Wall. Slurry Test Programme. First interim report. November 1988.
6. Hydraulic Barriers in Soil and Rock. Proceedings of Symposium on Impermeable Barriers for Soil and Rock. Denver June 1984.

APPENDIX

Laboratory Testing Details

The sample preparation technique chosen in the laboratory testing programme modelled a full scale in situ procedure.

Samples were prepared by firstly mixing prehydrated bentonite with cementitious material using a high speed paddle mixer. Once mixed, the slurry was poured into 100 mm diameter x 220 mm high split moulds. As much air as possible was then expelled, the moulds sealed, and the samples cured in a temperature controlled lime saturated water bath until they were required for testing. The samples were then removed from their moulds and trimmed down to their test length of 200 mm. The practice of casting overlength samples and then trimming to size was found to be necessary to prevent 'end effects', such as entrapped air bubbles, from affecting test results.

Drained triaxial compression tests were originally carried out on samples cured for only 7 days. However problems were encountered during such tests due to blockage of the drainage tubes (pore lines), thought to have been due to hydration products and unbound bentonite depositing in the lines.

Development of a soft mantle on the outer surfaces of the samples was a problem when curing in lime saturated water for 28 days or more. Such a mantle could well have had an effect on the results of triaxial compression and permeability tests. It is thought that the mantle was due to some form of chemical reaction with the lime in the curing water. Further samples were cured in water from Harris River and ordinary water from the water mains. No growth of soft mantle was observed.

Drained triaxial compression tests samples were carried out using effective cell pressures of 200 kPa, and straining at a rate slow enough to ensure that pore pressure did not build up.

Permeability tests were performed in triaxial test cells, using effective cell pressures of 200 kPa and (generally) applying hydraulic gradients of 45:1 across the samples.

GEOTECHNICAL CENTRIFUGE MODELLING AT THE UNIVERSITY OF WESTERN AUSTRALIA

[This article has been prepared jointly by all those involved in centrifuge model testing at The University of Western Australia. A list of research personnel, in alphabetical order, is given below.]

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1 INTRODUCTION

The main requirement for model testing in any area of Engineering is to ensure that key dimensionless groups are equivalent in model and prototype. In fluid mechanics, an area where dimensional analysis was pioneered, there are a number of such groups (Reynolds number, Froude number and so forth) which determine the performance of the prototype, and which need to be matched correctly in any small-scale modelling of the prototype. In solid mechanics, for problems where the self-weight of the material is important, the primary dimensionless group is the ratio of self-weight stress to the strength of the material. This principle needs to be taken even further in geotechnical research, where the response of soil and rock depends critically on the past and current stress state. Under these circumstances, it becomes necessary to simulate the full prototype stress field in any model experiment, distinguishing carefully between the ambient stress state prior to any perturbation, and the stress changes that take place subsequently during the experiment.

It is now widely accepted that centrifuge modelling provides the most versatile technique for obtaining stress conditions that are homologous in model and prototype. Other techniques – such as using a downward hydraulic gradient – may also be used, but none of these offers the same scope as centrifuge modelling. For element tests, or where the variation of stress through the event to be modelled is small (such as in modelling of a very deep tunnel, or the stress changes around an advancing cone), it is sufficient to use triaxial cells or larger so-called 'calibration chambers' where the boundary stresses are controlled. However, in problems where not just the ambient stress, but the *gradient* of ambient stress is important (such as an embankment on soft clay, or the performance of foundations on sand), centrifuge modelling becomes the optimum approach.

This paper describes the centrifuge facility that has been developed in the Department of Civil and Environmental Engineering at The University of Western Australia, and also presents brief descriptions of the current research projects being undertaken using the facility. The paper concludes with a summary of additional research areas where centrifuge modelling can play an important role both in fundamental research and in site-specific design studies.

2 CENTRIFUGE MODELLING

The primary aim of a centrifuge model test is to obtain similitude of stress and strain in model and prototype. This is achieved by accelerating a scale model, where all linear dimensions are reduced by a factor, N, to an acceleration of magnitude N gravities (g), effectively increasing the self-weight by a factor of N. Soil of density identical to the prototype is used in the model, so that the vertical overburden stress at a depth of z/N in the model is equal to that at a depth z in the prototype. Thus, for example, the stress distribution through a 0.5 m thick layer of soil in the centrifuge, at 200 g, is equivalent to the stress distribution in the same soil in the field over a depth of 100 m.

In general, model tests are carried out on soil which is similar in grain size to the prototype soil, in order to achieve similar stress-strain properties. Relative to the scale of the problem being modelled, the grain size of the model soil will therefore be a factor of N greater than in the prototype situation. Experiments have shown that this produces no measurable 'scale effect' provided that the ratio of grain size to the smallest significant dimension of the problem is less than about 3 - 5 %. For example, if a 1 m diameter pile were to be modelled at a scale of 100:1, the model pile would be 10 mm in diameter, and the grain size should be maintained below about 0.3 mm. In some cases, it may be necessary or appropriate to reduce the size of the soil grains in the centrifuge model tests, although it then becomes necessary to assess what difference such reduced grain size may make to the engineering properties of the soil.

2.1 Scaling Relationships

Centrifuge scaling relationships have been extensively described elsewhere, for example, Arulanandan et al (1988). The main principle is similarity of stress and strain, and the reduction of linear dimensions by a factor of N. The scaling factors for common quantities are summarised in Table 1.

Table 1

Scaling Factors for Centrifuge Modelling

Parameter	Dimensions	Scaling Factor
Acceleration	LT^{-2}	N
Seepage velocity	LT^{-1}	N
Length	L	1/N
Stress	$ML^{-1}T^{-2}$	1
Strain	-	1
Force	MLT^{-2}	$1/N^2$
Bending moment	ML^2T^{-2}	$1/N^3$
Time (diffusion)	T	$1/N^2$
Time (cyclic period)	T	1/N
Time (creep)	T	1

It is clear from Table 1 that correct scaling of time presents a problem - one that is common to any form of geotechnical modelling. Where consolidation is to be modelled, the time scale is reduced by a factor of N^2, which permits many years of prototype consolidation (or other form of diffusion) to be modelled in a few hours. However, if events such as liquefaction under cyclic or earthquake loading are to be modelled, then it becomes necessary to adjust the effective permeability of the soil (for example, by increasing the viscosity of the pore fluid) in order to match both cyclic period and diffusion time scale.

2.2 Centrifuge Facility at UWA

The centrifuge that has been installed at UWA was made in France by Acutronic, who were the manufacturers of the centrifuge at Nantes, one of the largest geotechnical centrifuges in Europe. The size of a centrifuge may be expressed as the product of maximum payload and maximum acceleration level, expressed in g-tonnes. The UWA centrifuge is a 40 g-tonne machine, with a maximum payload of 200 kg at an acceleration level of 200 g. Proportionally heavier packages, up to a maximum of 400 kg, may be tested at lower acceleration levels.

Acutronic make a range of geotechnical centrifuges, ranging from the Model 661 (as installed at UWA) up to the Model 680 (the Nantes centrifuge) which is a 220 g-tonne machine. A 600 g-tonne machine is currently being designed.

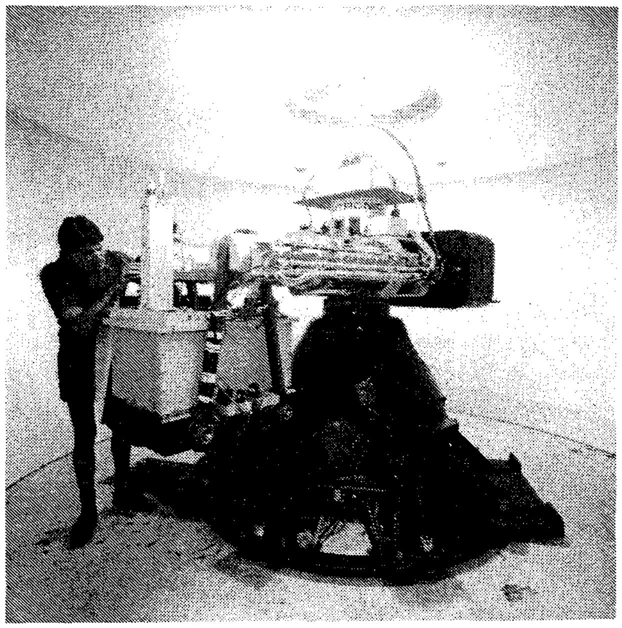


Fig. 1 Model 661 Acutronic Centrifuge

The Model 661 centrifuge, shown in Figure 1, has a swinging platform at a radius of 1.8 m, and a nominal working radius of 1.55 m. At maximum acceleration of 200 g, the rotational speed is 340 rpm, with a platform velocity of 64 m/s (230 km/h). The platform has a usable area of 500 mm by 700 mm, and can hold containers up to 500 mm high. Equipment mounted on the containers – such a cone penetrometer for 'in-flight' site investigation – is restricted to a height of 950 mm above the platform.

The centrifuge is housed at ground floor level in a specially-constructed circular reinforced concrete chamber. The chamber is located immediately outside an existing laboratory, and has direct access to that laboratory, which offers a spacious area for sample preparation. A novel feature of the housing is a 250 mm thick inner lining of high-density polystyrene, which has very good energy-absorbing characteristics, and also provides thermal and acoustic insulation. The chamber is completed by a curved fibreglass door, fitted with rubber seals to reduce air loss, and hung on a double pivot which allows sufficient access to remove the centrifuge if the need were to arise. At present, plans are being prepared to air-condition the chamber, in order to maintain a constant temperature throughout long tests, thus avoiding the large diurnal temperature variations which can cause significant zero-drift of instrumentation.

Communication with the centrifuge is via power and instrumentation slip-rings that are housed within the main axis, with the fixed cables exiting downwards, through the centre of the gearbox. This feature of the Acutronic design leaves the top of the central axis clear, thereby providing valuable additional space in the 'low-g' region. As may be seen in Figure 1, this space is used to house a 286 microcomputer, which will eventually handle all A/D data conversion and multiplexing, and also process-control of actuators on the package. The number of instrumentation slip-rings was

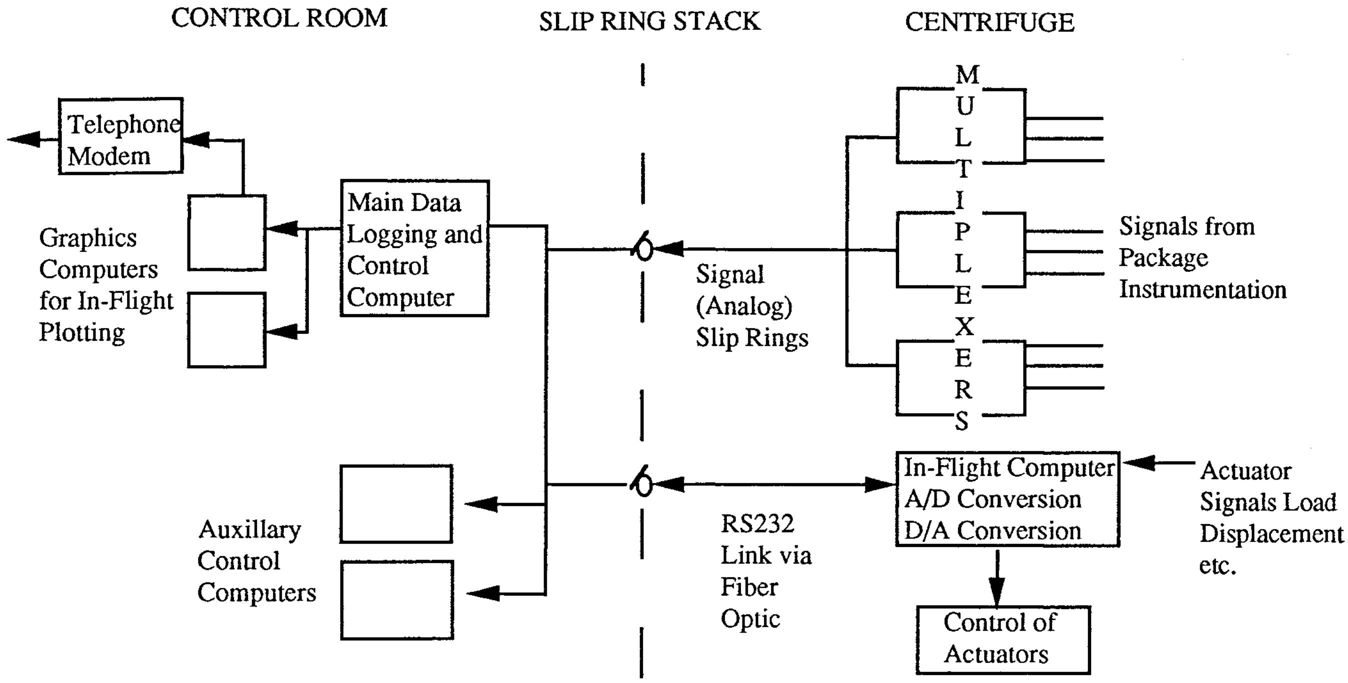


Fig. 2 Schematic representation of Centrifuge data logging and control system

deliberately limited to 23, since it was planned to multiplex signals on the arm. Thus, a total of 38 data channels are currently available, with plans to expand the system to over 100 as new multiplexing units are developed. In addition to the electrical slip-rings, there are two video slip-rings and a fibre optic slip-ring capable of transmitting data at up to 50 Mbps. The fibre optic link will eventually provide the primary mode of transmission for data digitised on the arm.

At present, a single hydraulic slip-ring has been added, with the hydraulic hose routed around the central computer (see upper part of Figure 1). This link will be replaced shortly by a 4-channel hydraulic slip-ring, to facilitate experiments on pollutant transport.

Two additional features that have proved most valuable are (a) miniature (20 gramme) CCD video cameras (Panasonic Model WV-CD1) that sit directly on the package, and (b) laser-operated remote sensing displacement transducers. The former devices permit close-up views of the test, with sufficient resolution to obtain quantitative measurements (using appropriate image processing software). The laser displacement transducer may be used to measure surface settlement of soft clay, or lateral movement of a pile, avoiding physical contact that may interfere with the measurement.

A number of packages and accessories have been developed since the centrifuge was installed. To date, most of the experiments have been performed in rectangular boxes, with internal dimensions 390 mm by 650 mm by 300 mm high. The boxes weigh about 70 kg, and permit models with up to 250 mm of saturated soil to be tested at the maximum acceleration level of 200 g. Electrical actuators have been developed for conducting cone penetrometer and foundation tests. Each actuator has two degrees of freedom, which allows relocation of the cone in-flight (between successive tests) and combined horizontal and vertical loading of foundations. The actuators are lightweight (10 - 12 kg), with a design specification of 10 kN at a maximum loading rate of 3 mm/s. Loading rates are controlled through software, and the above specification may be altered by changing either the motor, the reduction gear, or the recirculating-ball lead-screw.

2.3 Data Acquisition

The data acquisition system employed at the UWA centrifuge facility has been developed with the underlying philosophy of producing a fully automated, but versatile, control and data collection system. The principle of this system is shown schematically in Figure 2.

The system consists of a main data acquisition and control computer which is responsible for logging all data returning via the centrifuge slip-rings to a hard disk. This computer is also in communication with the computer mounted on the low-g central platform of the centrifuge, which provides control of actuators such as the cone penetrometer and foundation loading devices.

Rather than assigning all control, data-acquisition and graphing functions to a single computer, which would necessitate sophisticated and complex software development, these various tasks are distributed among auxiliary computers. The auxilary computers may each be programmed to perform specific tasks - such as operating solenoid valves for water level control, controlling hydraulic valves, or just graphing data.

The data logged by the main microcomputer are also passed to two graphics computers which can be programmed to display time records of selected channels, or to plot two or more channels against each other, during the test. One of the graphics computers also contains a modem, allowing the centrifuge user to monitor the test progress by telephone link, whilst the machine is running unattended - e.g. during long overnight consolidation periods.

The system outlined above requires a relatively large input from the centrifuge user - as each software package for the control and graphics computers must be set up for a particular test. However, each component is relatively simple to develop, and yet the whole system is extremely versatile.

2.4 Current Research Projects

The centrifuge at UWA has now been in operation for a year (since June, 1989). Use of the centrifuge has grown rapidly, and it is now heavily booked for the next six months. A list of major projects being undertaken are listed below:

- Shallow foundations on calcareous soil
- Embankment loading of piles.
- Response of rectangular box culverts.
- Studies of pollution migration through soil.
- Consolidation at low effective stresses.
- Modelling jointed rock.
- Mining induced subsidence.

These projects, which are supported through grants from industry and from the Australian Research Council, are described in more detail in the following sections.

3 SHALLOW FOUNDATIONS ON CALCAREOUS SOIL

3.1 Introduction

The geotechnical centrifuge at UWA is being used to explore the behaviour of shallow foundations in calcareous soil, with application to the design of offshore structures. Current design methods for shallow foundations on calcareous soil are based primarily on traditional bearing capacity calculations, which fail to take account of the high compressibility of calcareous soil. The study aims to examine various factors that affect foundation performance, namely: density (or void ratio), degree of cementation and layering of the soil, eccentricity and inclination of applied loading, and cyclic loading.

The project was one of the first to be carried out on the centrifuge, and necessitated considerable development of equipment and testing techniques. The main equipment consisted of actuators to permit 'in-flight' cone penetration testing and combined vertical and horizontal loading of the model foundations.

Calcareous soil recovered from the sea-bed on the North-West Shelf of Australia is being used for the model tests. The soil is uncemented, and is oven-dried and sieved (to remove the larger shell fragments) prior to use. Techniques are being developed to re-cement the soil, using calcium hydroxide, in order to simulate the cemented layers that are encountered in practice. Such cementation, and the intervening layers of calcareous muddy silt that exist in the field situation, are likely to be critical in determining punch-through and liquefaction failure of shallow foundations.

3.2 Equipment and Sample Preparation

The model tests have been carried out in a centrifuge 'package', that consists of a strong box upon which sit two actuators (see Figure 3). The taller of the two actuators comprises a cone penetrometer, which may be relocated across the width of the box between successive penetration tests. The cone is 10 mm in diameter, with a load cell at the tip and another load cell at the top of the shaft. The maximum penetration rate is 3 mm/s, with a maximum range of 30 MPa.

The shorter actuator provides loading for the shallow foundation, with independent control of vertical and horizontal displacement. Software allows particular loading paths to be followed, including maintaining a constant vertical load, during horizontal cyclic loading. In the early tests, cyclic loading was limited to slow, essentially drained, loading, with periods of 1 - 20 s. The system is currently being developed further, in order to achieve cyclic loading rates in the region of 10 Hz.

In the tests performed to date, a total of five cone penetrometer tests and two foundation tests have been carried out on each sample of soil. The number of tests is a compromise between maximising the amount of data from each sample, and yet avoiding too much interaction between tests. In each sample, tests were carried out at two acceleration levels (100g and 140g), in order to assess the validity of the model test results.

To date, a total of five soil samples have been prepared, as follows:

- A sample, (1), prepared by dry pluviation and densification by vibration. This produced a relatively dense sample.
- A sample,(2), prepared by dry pluviation leading to a less dense sample.
- Two identical samples, (3 & 4), prepared by mixing soil with silicon oil under vacuum, and placing the soil as a slurry. Silicon oil, with a viscosity one hundred times that of water, was used as the pore fluid in order to increase drainage times. Increasing the

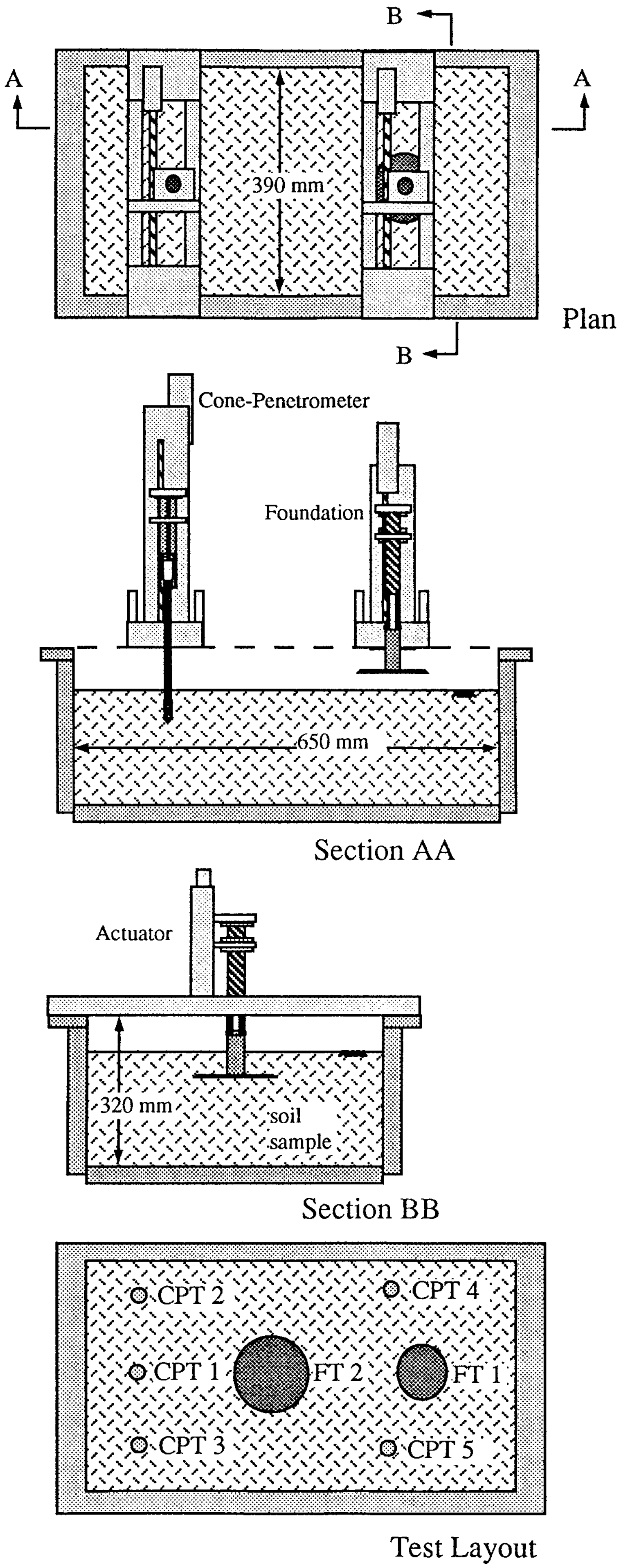


Fig. 3 Layout of Foundation Test Package.

loading rate by a factor of a hundred and reducing the effective permeability of the soil by the same factor allows correct simulation of excess pore pressures between model and prototype. (Note that, if water was used instead of silicon oil, the loading rate would have to be increased by N^2.) This technique, commonly used in modelling earthquake events at small scale, is particularly useful for modelling liquefaction under cyclic loading.

- A 'cemented' sample, (5), prepared by combining soil with calcium hydroxide and water, then flushing with carbon dioxide and saturating under vacuum. This produced a chemical reaction resulting in inter-particle bonds of calcium carbonate, thus producing a sample with a cemented micro-structure.

Using these samples, a total of ten foundation tests and twenty cone-penetration tests have been carried out.

3.3 Test Results

Differences in sample properties due to the different methods of preparation are reflected in the profiles of cone resistance, as shown in Figure 4(a). In general, the lateral variation in soil properties within each sample was minimal, as evidenced by the results of different cone tests in the same sample; Figure 4(b) for example shows the results of three such tests at different locations in sample 3.

Also shown in Figure 4(a) is an approximate envelope of cone resistance obtained from a typical site on the North-West Shelf. It is clear that, while the cone resistance of the upper 6 m may be matched reasonably well by the model soil, the layers of very low cone resistance (for example, in the range 7 - 14 m) are not reproduced. Inspection of the borehole logs shows that the low cone resistance is generally associated with a high fines content - calcareous muddy silt - compared with the silty sand that has been used in the model tests. The inclusion of layers of finer material in the model tests is an essential part of future work aimed at evaluating the potential for punch-through and liquefaction types of failure.

The model foundations consisted of flat circular footings, of diameters 70 and 100 mm. At acceleration levels of 140 and 100 g respectively, these represent a prototype foundation of diameter 10 m, which corresponds to a typical offshore foundation for jack-up rigs or small gravity-based platforms.

A typical response is shown in Figure 5(a) for a test on a 100 mm diameter footing (at 100 g) in the second soil sample. The bearing envelope shows no clear failure load, but rather a continuously increasing bearing capacity with increasing penetration. A bearing stress of just under 500 kPa is required for a settlement of 10 % of the footing diameter.

The test included three unload-reload loops, and also two stages of horizontal cyclic loading. Both stages of horizontal cyclic loading, at a shear stress amplitude of ±50 kPa, gave rise to significant vertical settlement of the footing. However, on subsequent vertical loading, the original bearing envelope was rejoined.

Foundation response from different sized models tested at different accelerations, but modelling an identical prototype, compared well, as shown in Figure 5(b). Thus the modelling technique is validated.

3.4 Future Tests

The actuator control is currently being developed further in order to increase the rate of cyclic loading to about 10 Hz. This, together with the silicon oil pore fluid, will allow simulation of pore pressure build-up during typical storm loading.

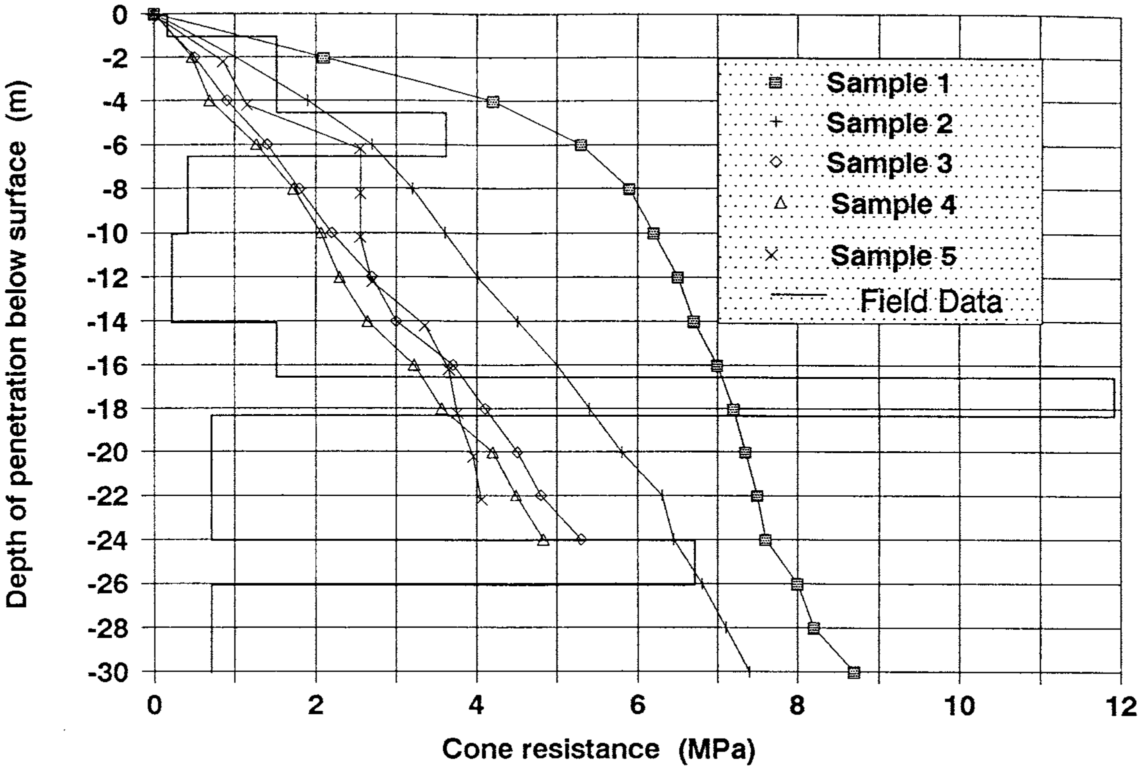
Comparison of the cone resistance profiles from the model and prototype situations indicates that the bearing response of the calcareous soil is significantly affected by the particle size distribution. In particular, a layer of finer material close to the surface may prove critical in the design of prototype shallow foundations. This aspect will be investigated in future testing.

It is also planned to include additional instrumentation in the soil in order to provide data on the transfer of stress. Miniature pore pressure transducers and earth pressure cells will be embedded beneath the model foundations, and the information used to develop analytical models of bearing capacity in calcareous soil.

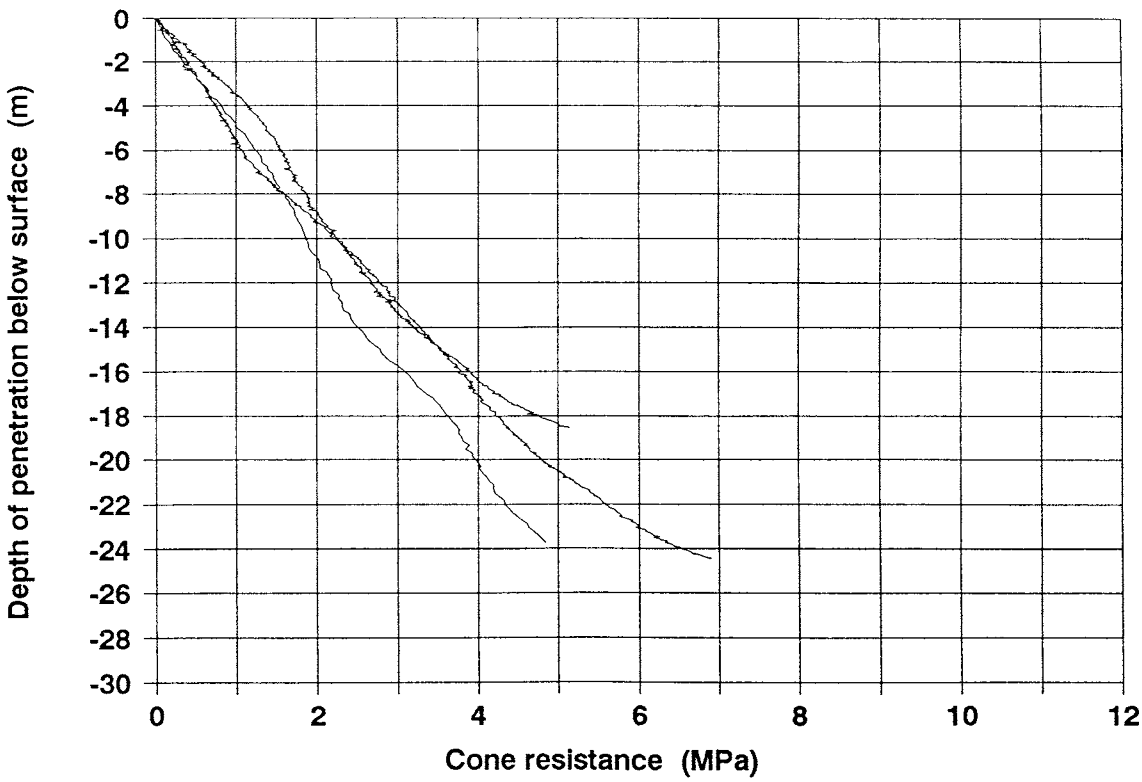
4 LATERAL LOADING OF PILES IN SOFT CLAY DUE TO NEARBY EMBANKMENT CONSTRUCTION

Construction of embankments on soft clay results in the development of significant time-dependent embankment settlements, and both horizontal and vertical movements within the soft clay. Where a bridge approach embankment is founded on soft clay, construction of the bridge itself is sometimes commenced before full settlement of the embankment has occurred. Piles supporting bridge abutments adjacent to such embankments may therefore experience significant lateral forces from horizontal soil movements. These lateral forces induce bending moments and deflections in the piles which may lead to structural distress or failure of the piles or bridge structure.

Design of piled bridge abutments subjected to such forces has in the past been based on both theoretical (Poulos, 1973) and empirical (DeBeer and Wallays, 1972) analyses. However, in a limited number of full scale field trials, the forces predicted by various approaches have in general been in poor agreement with measured values. Hence, prediction of the bending moments and deflections induced in abutment piles in this situation remains problematic. Because of this, a conservative design incorporating caissons to shield piles from lateral soil displacements may be adopted, or

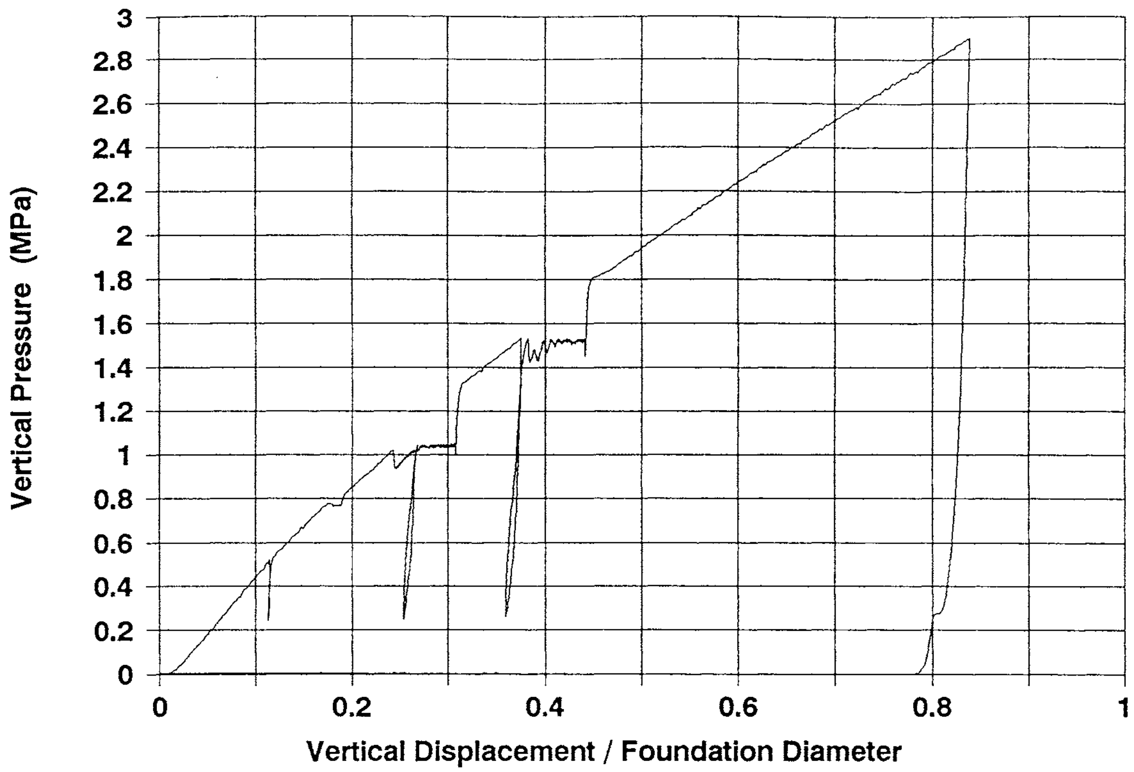


(a) Cone-Penetration Tests. Typical Profiles.

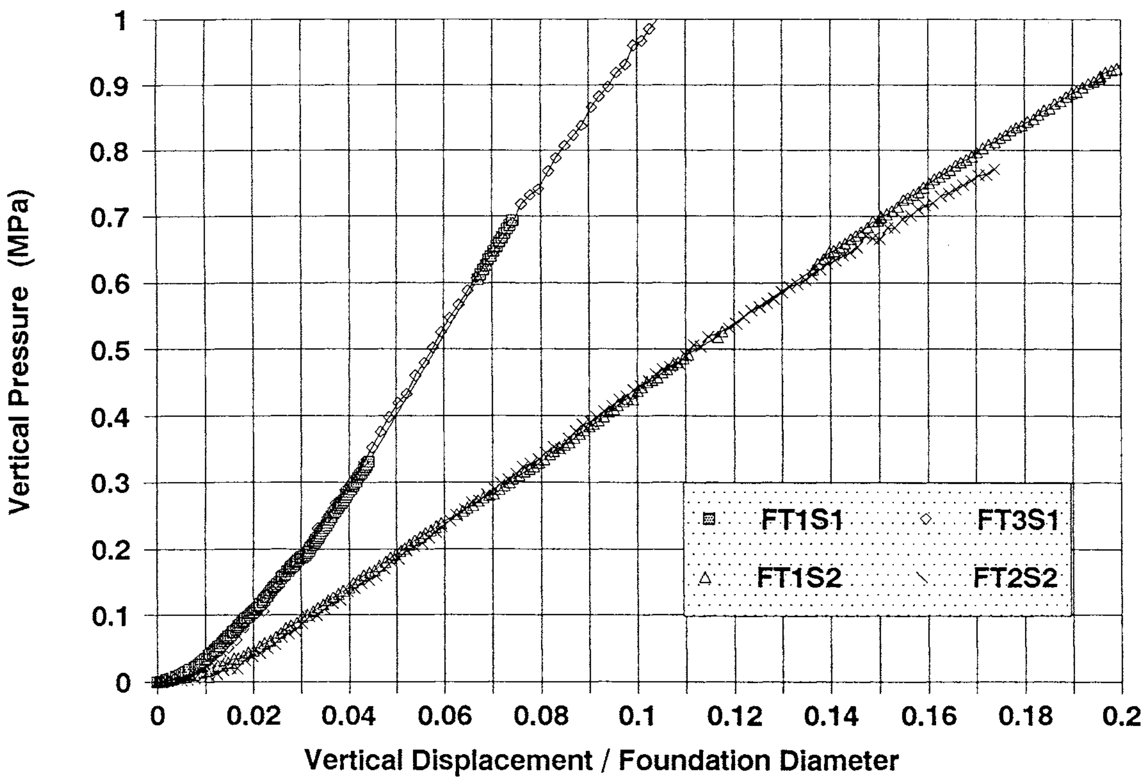


(b) Cone-Penetration Tests. Sample 3.

Fig 4. Cone-Penetration Data in Calcareous Soil.



(a) Bearing Pressure versus Penetration, Foundation Test 2, Sample 2.



(b) Bearing Pressure versus Penetration, Samples 1 and 2.

Fig 5. Foundation Test Data in Calcareous Soil.

bridge construction may be delayed until full settlement of the approach embankment has occurred. If the bending moments and deflections induced in the piles can be estimated accurately, then more cost effective construction procedures may be implemented.

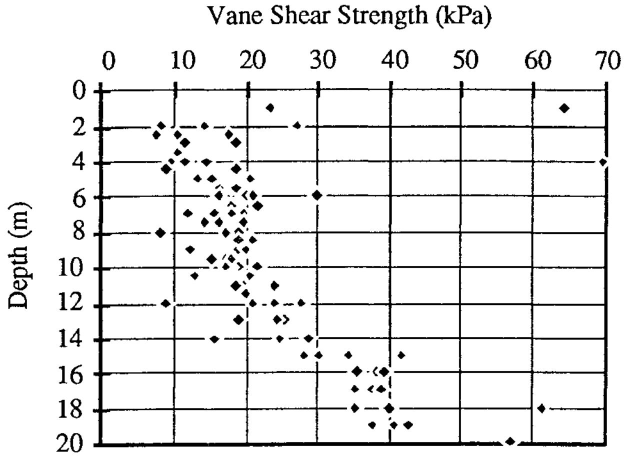


Fig. 6(a) Vane Shear Strength Test Results - Burswood Bridge Abutment

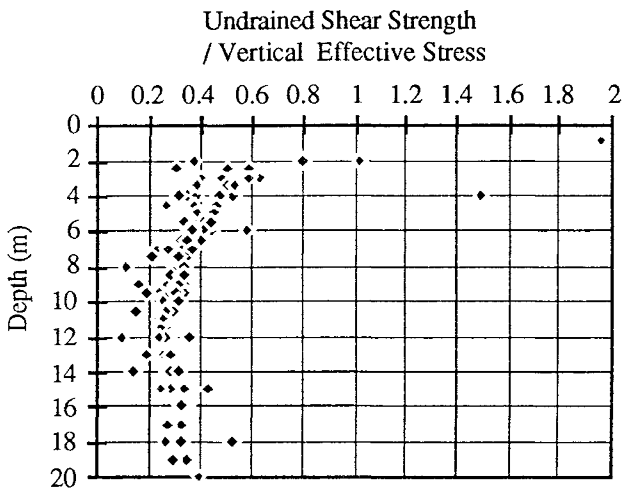


Fig. 6(b) Vane Shear Strength Test Results - Burswood Bridge Abutment

Centrifuge model tests are being conducted at UWA to obtain data on the interaction between an embankment on soft clay and piles driven nearby. This project is being sponsored by the Main Roads Department of Western Australia. It is directly relevant to the design of the proposed Burswood Bridge across the Swan River in Perth. The proposed abutment site on the Burswood Peninsular is underlain by an 18 m deep layer of soft to firm organic clay deposited in a meander of the Swan River (Geidans and Kilvington, 1984). Field vane shear test results from the proposed abutment location are shown on Figure 6(a) as shear strength versus depth, and on Figure 6(b) as the ratio S_u/σ'_v versus depth. The ratio S_u/σ'_v gives an approximate indication of the variation in OCR with depth, showing that the upper 10 to 12 m of the clay is overconsolidated.

Centrifuge models for this project are prepared by consolidating a slurry of kaolin under a surcharge of 60 kPa, before transferring the sample to the centrifuge for final self-weight consolidation at 110 g. The strength profile of the models, estimated from the results of CIU triaxial tests, is shown on Figure 7(a) with the field vane results superimposed for comparison. The stress history of the samples results in the overconsolidation profile indicated on Figure 7(b). These figures show that strength and overconsolidation profiles similar to the prototype may be achieved in the models by selection of an appropriate stress history.

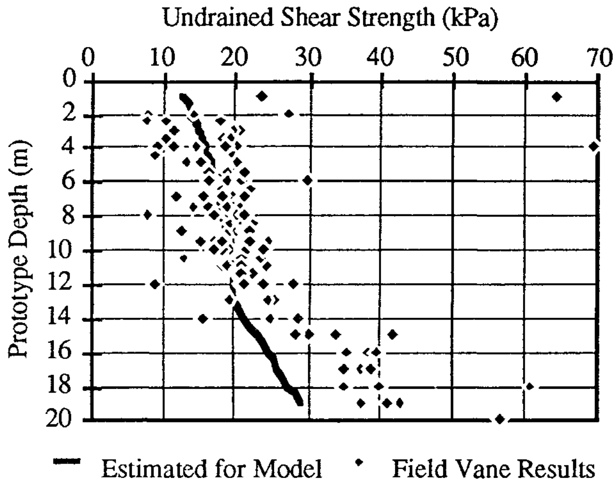


Fig. 7(a) Shear Strength and OCR profiles for Centrifuge Models

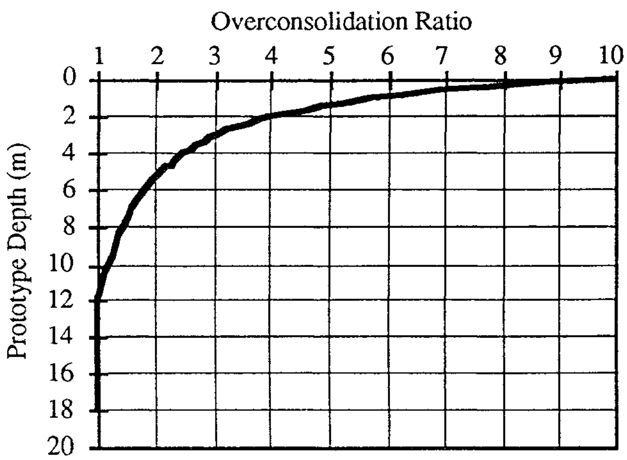


Fig. 7(b) Shear Strength and OCR profiles for Centrifuge models

The layout of the centrifuge models is illustrated on Figure 8. Model piles have been constructed from 3.2 mm square hollow brass sections to replicate the bending rigidity of the 310 UC 158 steel sections proposed for the prototype piles. The model piles are strain gauged externally to measure bending moment at ten levels, and their deflection above the soil surface is measured with a non-contact laser displacement sensor.

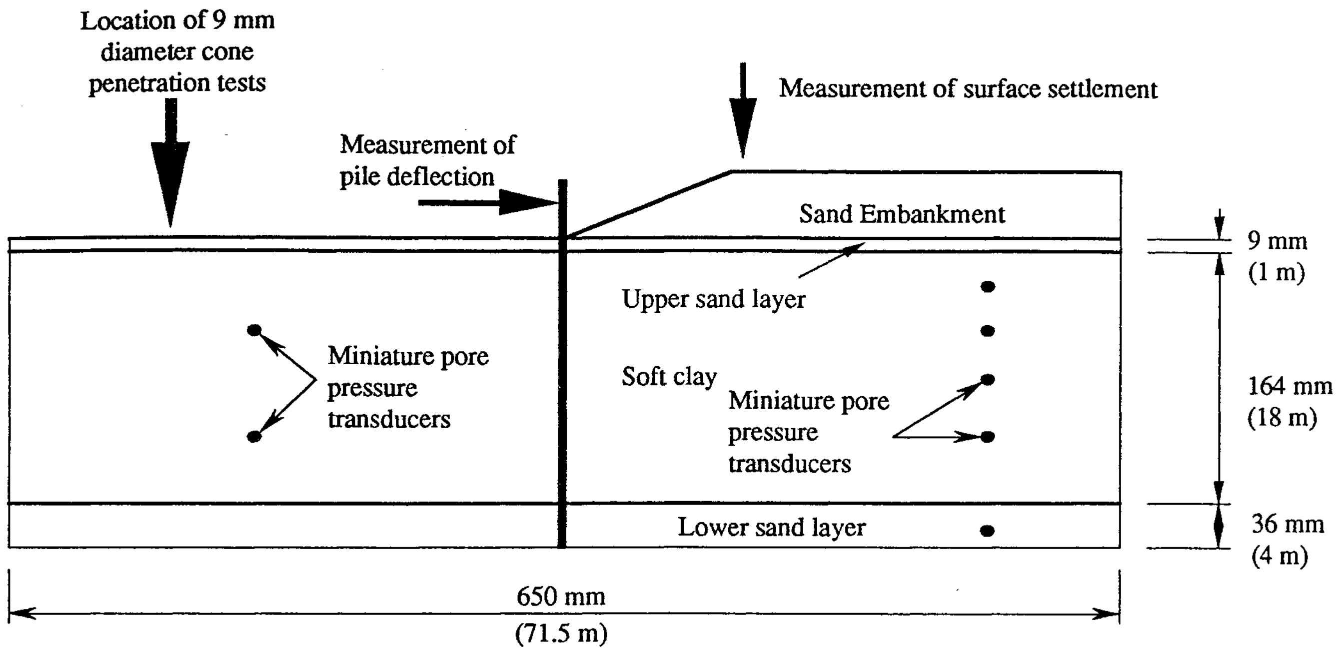


Fig. 8 Layout of Centrifuge Models

To date, the undrained shear strength of the soft clay has been assessed by performing cone penetration tests 'in-flight'. A sensitive 9 mm diameter cone penetrometer has been developed for this purpose. However, a major limitation of the cone penetrometer lies in the empirical factors used to correlated cone resistance with undrained shear strength.

A new device is currently being developed for the centrifuge, to enable shear strength to be measured more directly. The device comprises a cylindrical bar mounted at right-angles (to form a T) at the end of a vertical shaft. The T-bar is pushed into the soil and the penetration resistance measured by a load cell situated immediately behind the bar.

The results of the test are interpreted making use of the plasticity solution for flow of soil around a cylinder (Randolph and Houlsby, 1984), which enables the shear strength to be deduced directly from the measured penetration force.

The new device combines the advantages of the cone penetrometer (giving a continuous profile of 'strength') with the vane shear tests (which gives an 'exact' measure of shear strength). Experience so far has shown a significantly better correlation with shear strength than obtained with the cone penetrometer, independent of the overconsolidation ratio of the soil.

In preliminary testing, a sand embankment was constructed in several stages adjacent to a single pile by stopping the centrifuge, building the embankment by hand, and then restarting the centrifuge. Such an approach induced significant errors in the test results, as the pile was able to straighten when the centrifuge was stopped, but served to test the instrumentation systems and general modelling procedures.

The pile bending moments measured during one stage of embankment construction in a preliminary test are shown on Figure 9. Only seven levels of strain gauges on the pile were functional during this test. The bending moment data were fitted with a polynominal curve, from which lateral pressure and deflection profiles were derived using simple beam theory. These profiles are also shown on Figure 9.

To enable accurate modelling of embankment-pile interaction, it is imperative to construct the embankment 'in-flight" in the centrifuge, A sand hopper has been designed which will allow sand to be poured through more than 200 small valves onto the surface of the model during flight. Each individual valve takes sand from a separate compartment in the hopper, so that different amounts of sand may be poured through each valve. This will enable three dimensional embankments of virtually any shape to be constructed in-flight. Embankments of up to 12 m prototype height will be able to be constructed.

5 RECTANGULAR BOX CULVERTS

5.1 Introduction

In recent years, modelling of the soil-structure interaction of buried, flexible, circular culverts has been performed by numerous research groups. As a result, the soil-structure interaction problems of such culverts are better understood, and new design and analysis procedures are being proposed. In the meantime, research on rectangular box culverts has remained limited, and at present little information is available on the response of these structures to loading either by axle loads at low

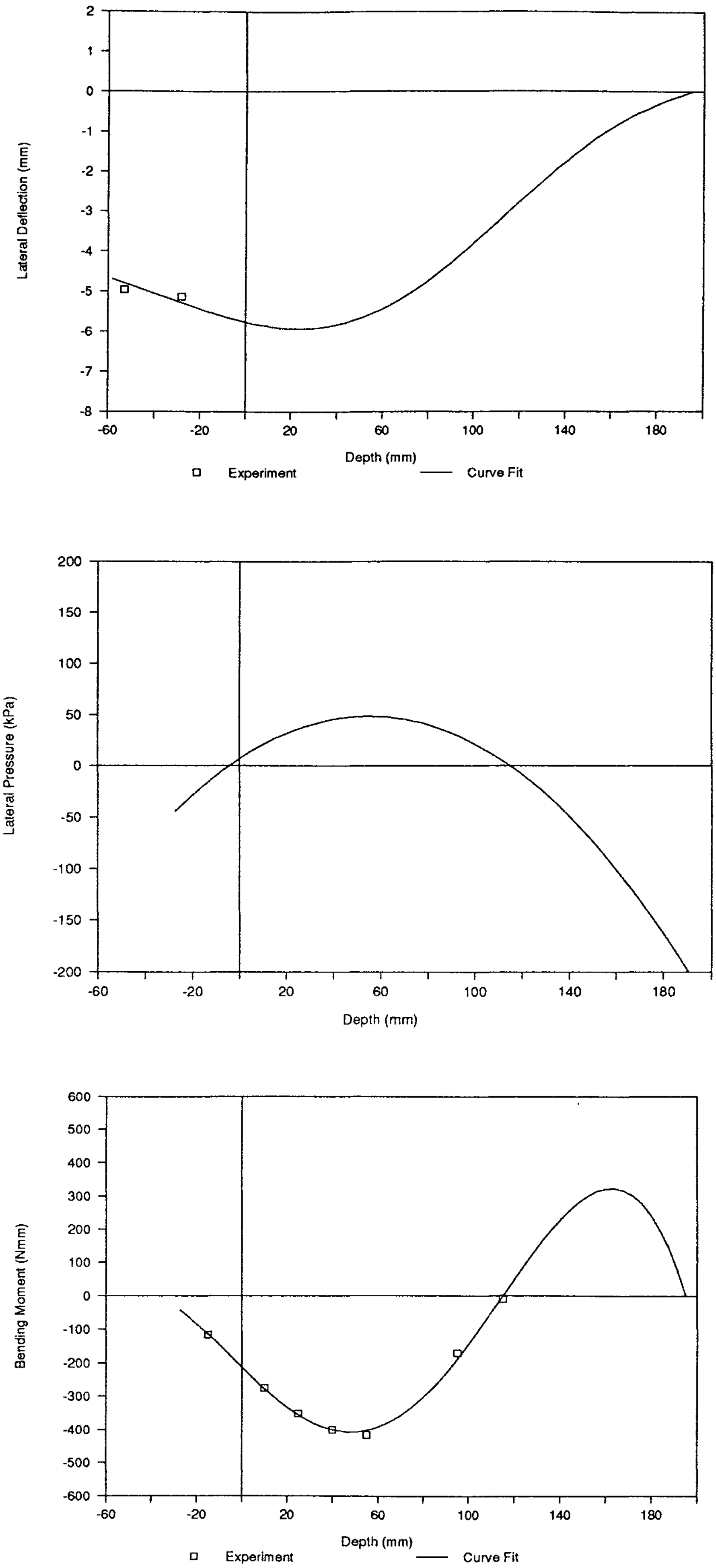


Fig. 9 Preliminary Test Results - Pile Response at 2 m Embankment Height

depths of cover, or by soil loads at high depths of cover. Such information is urgently required in order to assess present building code and safety standards, and to validate current methods in use for design of rectangular culverts.

The response of rectangular box culverts under different loading conditions is a problem which is difficult to solve theoretically or analytically, but may be examined conveniently using centrifuge testing techniques.

5.2 Current Research

The Geomechanics Group at UWA is presently embarking on a study of the pre-failure response of rectangular box culverts under two loading conditions. During the study it is planned to examine the relationship between the load carried by a buried rectangular culvert, and the relative soil structure stiffness.

Four model culverts will be manufactured from mild steel and instrumented at 12 points to measure bending or axial strain. Each steel culvert will have the same internal dimensions. However, wall thicknesses will be varied between the culverts, in order to produce a wide range in structural stiffness over the four models. All model culverts will be incorporated in homogeneous centrifuge samples constructed from dry silica sand.

The behaviour of each model culvert will be examined under two loading conditions, as shown in Figure 10. The first condition corresponds to a uniform soil load under a high depth of cover (Figure 10(a)), while the second condition corresponds to a live load, applied as a uniformly distributed strip load along the longitudinal axis of the culvert, under a low depth of cover (Figure 10(b)).

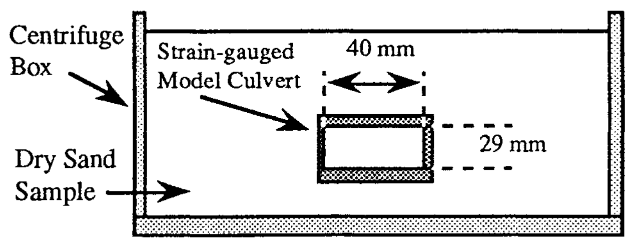


Fig. 10(a) Model Configuration for Soil Load at a High Depth of Cover

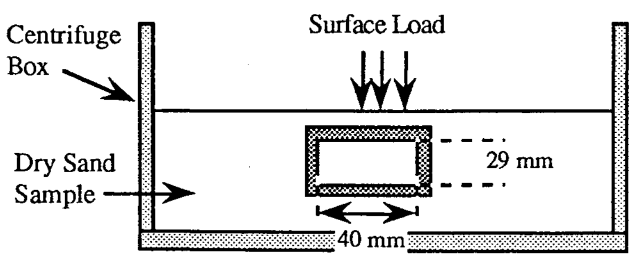


Fig. 10(b) Model Configuration for Surface Load at a Low Depth of Cover

Results from the test series will be used to establish a relationship between the load carried by a rectangular culvert and the soil-structure stiffness ratio. This will enable evaluation of current methods in use for rectangular box culvert design, and will provide valuable insight into the general behaviour of buried, flexible structures under different loading conditions.

6 MODELLING OF CONTAMINANT TRANSPORT PROCESSES IN SOIL

6.1 Introduction

During the past decade, much emphasis has been placed upon the importance of quantifying transport processes in the subsurface. A better understanding of the mechanisms which govern mass, heat and contaminant transport in soils is essential, if environmentally sound strategies are to be developed for controlling subsurface contamination.

At present, there exists a critical need for physical observations of pollutant behaviour in soils. Such observations are required both to validate existing mathematical transport models, and to aid in developing improved conceptual models of fundamental transport processes.

Controlled field experiments and laboratory column tests have traditionally provided the bulk of experimental data on pollutant behaviour in soils. However, researchers have recently come to recognise that a geotechnical centrifuge can provide a powerful testing tool for the physical modelling of transport phenomena in porous media.

Geotechnical centrifuge modelling supplies a means of carrying out small scale physical modelling of many transport problems, under repeatable and controlled laboratory conditions. It is a feature of centrifuge model tests that transient processes, such as groundwater contaminant migration, that occur in long prototype times, may be reproduced in a centrifuge model in short model test times. For example, a 1/100 scale model in a relative centrifugal field of 100g, is capable of reproducing almost 30 years of pollutant transport data, in only 25 hours of model test time.

Because the product of depth times acceleration is the same in centrifuge model and corresponding prototype, the stress distribution throughout the model will be identical to that throughout the prototype. This means that gravity dependent phenomena, such as free convection under non-isothermal conditions and physical migration due to density gradients, can be correctly replicated in a reduced scale centrifuge model: correct replication of these effects is not possible outside a centrifugal field, unless a full scale field test is carried out.

The Geomechanics Group at UWA is currently involved in two research projects entailing the centrifuge modelling of transport processes in porous media.

These projects are discussed under separate headings below.

6.2 Modelling of Absorption and Dispersion Processes

Although much progress has recently been made in the centrifuge modelling of transport processes, two fundamental areas of uncertainty remain associated with this field of research. The first concerns modelling the dispersivity and heterogeneity of a porous medium, in particular the observed dependence of field scale dispersion upon a 'length-scale' factor. The second concerns the modelling of sorption and retardation processes. Both factors must be addressed before modelling of site specific problems can be attempted on the centrifuge. The complexity of modelling sorption processes on the centrifuge demands the necessity of interdisciplinary co-operation, and the possibility of a collaborative research project with experts in this field is being investigated. A study of problems associated with the modelling of dispersion processes is already underway.

In order to replicate all dispersion processes correctly in a reduced scale centrifuge model, both macroscopic and microscopic prototype lengths must be scaled. This requires scaling of prototype particle sizes in the centrifuge model. In order to investigate prototype particle scaling on the centrifuge, a series of centrifuge tests is planned using models constructed of different gradings of silica sand. Three sands with particle diameters of 0.1 mm, 0.45 mm and 0.9 mm have been identified as suitable modelling materials. These sands will be incorporated in homogeneous centrifuge models of a pollutant transport problem, Figure 11, and tested at suitable g-levels. A comparison of scale will then be made between experiments. This will enable the feasibility of scaling lengths in a centrifuge model to be established, and will also provide valuable insight into the influence of particle size on microscopic dispersion processes.

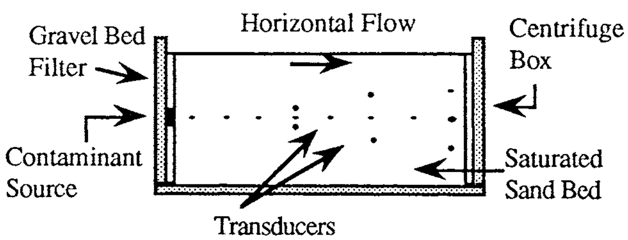


Fig. 11 Model Configuration for a Centrifuge Study of Dispersion Processes

At a later date, centrifuge models will be constructed which incorporate different layers of each modelling material in the same sample. This will enable the influence of macroscopic heterogeneities on mesoscopic (field-scale) dispersion processes to be assessed

It is proposed to use a standard sodium chloride solution as the model pollutant in all tests. The progress of the contaminant plume as it moves through the centrifuge model will be mapped by an array of buried resistivity probes, which are currently being developed at UWA.

All centrifuge model tests will be compared with theoretical predictions from a suitable numerical code. The work described above is being supported by funding from the Australian Research Council, and Alcoa Australia Ltd.

6.3 Coupled Physical Modelling of Heat and Contaminant Transport

To date, few experimental data exist on the subsurface migration of contaminants under non-isothermal conditions. Collaborative research between Cambridge University and The University of Western Australia has been initiated in this area, with small scale physical modelling of a coupled heat and contaminant transport problem.

A series of centrifuge model tests is to be performed in a rectangular strong box, using different gradings of fully saturated silica sand. High temperature sodium chloride solution will be introduced into each model during centrifuge flight, along a buried horizontal porous pipe, Figure 12. Contaminant transport and temperature rise in the soil surrounding the pipe will be monitored throughout each test by combined miniature resistivity and temperature probes, which are also being developed at UWA.

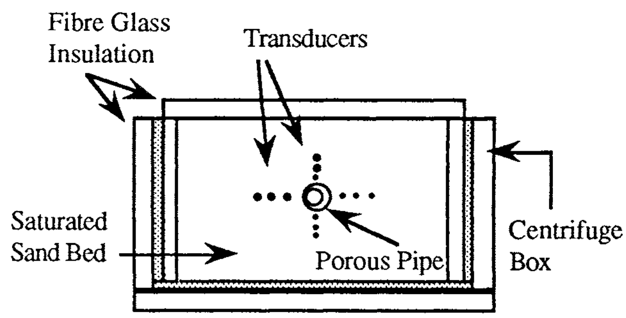


Fig. 12 Model Configuration for Coupled Heat and Contaminant Transport

During the centrifuge test series, the influence of both particle diameter (microscopic heterogeneity) and waste temperature on transport processes will be investigated. All centrifuge data will be compared with theoretical predictions from a suitable numerical code.

6.4 Future Research Projects

Funds are currently being sought for two further projects involving the centrifugal modelling of contaminant transport processes.

The first project will entail a study of dispersion in the presence of small vertical density gradients. Numerous field experiments performed in the presence of vertical density gradients have indicated that plumes in predominantly horizontal flow, tend to spread more in the lateral horizontal direction than the vertical direction. This seems to suggest that the dispersion tensor depends upon more than the two most commonly used terms: longitudinal dispersivity and lateral dispersivity. More experimental data are needed in order to quantify dispersion processes under such conditions. The Geomechanics Group at UWA have proposed a collaborative project in this area with the CSIRO Division of Groundwater Research.

Funds are also being sought to initiate a three year study into problems associated with the land disposal of waste effluents. During this project, the integrity of landfill liners under the action of real contaminants will be examined, and subsurface contaminant transport and recovery processes will be investigated in the presence of complicated flow and geological boundary conditions. Close collaboration with industry is anticipated at all stages of this project, with results from the study forming a much-needed bridge between fundamental research and waste management practice.

7 CONSOLIDATION OF SLURRIED TAILINGS

7.1 Introduction

The safe and economical disposal of mine tailings is one of the problems which confronts the mining industry, especially now with the increasing pressures being brought to bear on all aspects of mining by the environmental lobby. In many cases, mine tailings are deposited as dilute or thickened slurries in containment areas which may be natural topographical features or may be engineered specifically for the purpose. If the liquid entrained in the slurry either has the potential to pollute the underlying soil or groundwater, or has a commercial value, it may be desirable to contain the liquid completely in the disposal area and prevent any loss into the underlying formation, and/or to recover the liquid to feed back into the process stream.

For proper management of tailings disposal areas, both during the active life of the area and during any subsequent rehabilitation period, it is essential to understand the processes which control the movement of water in the tailings, the rate and final amount of settlement of the surface, and the changes in the strength profile through the depth of the tailings. This involves understanding the processes of initial sedimentation, self-weight consolidation under various drainage boundary conditions and formation of surface 'crusting' due to the combined effects of water table lowering in the tailings and surface dessication.

The current project in this area, funded by the ARC, is concentrating initially on understanding the process of self-weight consolidation. The approach which has been adopted is to develop numerical tools to model the processes, and to "calibrate" these tools with centrifuge modelling.

7.2 Numerical Modelling

The consolidation of slurried tailings is a classic example of large strain consolidation – that is, the changes in geometry and soil properties from the initial state to the final state are such that the original consolidation theory of Terzaghi, as traditionally applied, is not an appropriate means of analysing the problem. Thus, to deal with this problem, it is common to resorted to using the large strain consolidation model proposed by Gibson *et al.* (1967), which incorporates the features essential to deal with tailings consolidation: changing geometry of the layer and changing permeability and stiffness with reduction in voids ratio. An alternative approach has been adopted in the current project. Using numerical (finite element) techniques and fast modern computers, the traditional formulation of Terzaghi can be used for large strain consolidation problems provided the geometry and soil parameters are updated after each increment of consolidation; in effect, the small strain formulation is applied to small steps of consolidation, and the large strain aspect dealt with by updating the problem after each increment. In any practical application, this approach has the advantage that the updating phase after each consolidation increment can include addition of fresh tailings at the surface, which would occur in practice during the active life of the disposal area.

In the self-weight consolidation problem applied to tailings disposal, the drainage boundary conditions may be any of the following:

- surface drainage only
- surface and base drainage, but with the base water table being the same as the surface water table; this would correspond to the case of an impermeable disposal area with a filter layer at the base, but with no pumping from this layer.
- surface and base drainage, with the water table in the base being lower than the surface layer; this would include the case of complete underdrainage, with the water table in the underlying layer being kept below the base of the tailings
- combinations of the above, applying at different stages of the life of the area.

Numerical models, once formulated, require some means of verification or "calibration" if they are to be used with confidence in practice. It is possible to do this calibration using a combination of laboratory consolidation tests and observations of the performance of full-scale disposal areas. However, a much more satisfactory approach, at least in the early stages of development, is to use centrifuge modelling.

7.3 Centrifuge Modelling

Centrifuge modelling is the ideal method of investigating self-weight consolidation of soft soils. The scaling laws summarised in Table 1 indicate that consolidation takes place N^2 times faster than for the equivalent prototype. Thus, 1 day of consolidation on the centrifuge, at an acceleration of 100 g, is equivalent to 100^2 days (27 years) consolidation at prototype scale. The various drainage boundary conditions of interest, listed above, can easily be implemented on the centrifuge, and measurement of pore pressures and settlements throughout the layer is relatively straightforward on the centrifuge. After completion of consolidation, the strength profile can be determined using the in-flight site investigation tools already described. After the test, when the centrifuge stops, the layer can be sampled quickly to establish the final moisture content and dry density profiles.

An initial proving set of tests has already commenced using kaolin prepared at a water content equal to twice the liquid limit. Four drainage boundary conditions are being examined: top drainage only; top and bottom drainage with the bottom water table at the same level as the top water table; top and bottom drainage, but with both boundary water tables at the mid-height of the sample; and top and bottom drainage with both boundary water tables at the base of the container. Miniature 'Druck' pore pressure transducers are placed in the slurry to monitor the progress of consolidation, and surface settlements are measured using an LVDT resting on a small surface plate. Due to the increase in self weight with increasing acceleration, all transducers must be counterweighted using pulley systems to prevent them sinking through the slurry during the early stages of the test while the slurry is still soft. To date, tests of up to 36 hours have been run.

The results obtained from this testing indicate that the techniques adopted result in well-controlled tests which provide the quality of data necessary for calibration of the numerical model. A parallel programme of laboratory consolidation tests is now underway to determine independently consolidation parameters for the soil at all stages from a slurry to the maximum consolidation stress. The model will then be used to predict the performance in the centrifuge tests using these parameters.

Of primary interest in this programme is the 'crusting' effect of the low water table induced by complete or partial underdrainage. In theory, underdrainage induces suction in the pore water above the water table, with the final suction value determined by the height above the water table. If the soil remained fully saturated above the water table, these suctions would result in high effective stresses being induced in the soil right to the surface, giving much greater strength increase than would be achieved by self-weight alone. However, in practice, this effect is muted due to desaturation at some height above the water table. The centrifuge tests permit this whole process to be examined in a controlled way.

8 MODELLING OF JOINTED ROCK

The centrifuge facility is presently being used to study the response of reinforced and unreinforced blocky rock masses that are common to many mining, and some civil engineering, surface excavations.

The centrifuge study forms the physical modelling component of a research program aimed at defining a suitable design methodology for the reinforcement of unstable blocks of rock that form in the periphery of the excavation. The physical modelling is being conducted in conjunction with field work and analytical and numerical modelling.

Historically, the centrifuge has not been used widely in rock mechanics studies, partly due to limitations of even the largest machines to achieve sufficiently high stress levels in the rock. For surface excavations, where stresses are considerably lower than those encountered underground, the UWA centrifuge can model a maximum 'rock' slope height of about 50 m. Surcharging on the upper surface may be used to augment the stresses in the model material.

In a preliminary test, a model rock slope was built out of 14,000, 15 mm x 15 mm x 15 mm blocks. The blocks were assembled as a steep rock slope in a vertical, 'brick wall pattern' in both plan and section. The model arrangement is shown in Figure 13. The model blocks were lightweight (unit weight of 9.5 kN/m^3), and gave a Class 2 type uniaxial behaviour with a uniaxial compressive strength of around 0.75 MPa. The model was designed to simulate a rock mass with an inherent strength approaching that of a continuum.

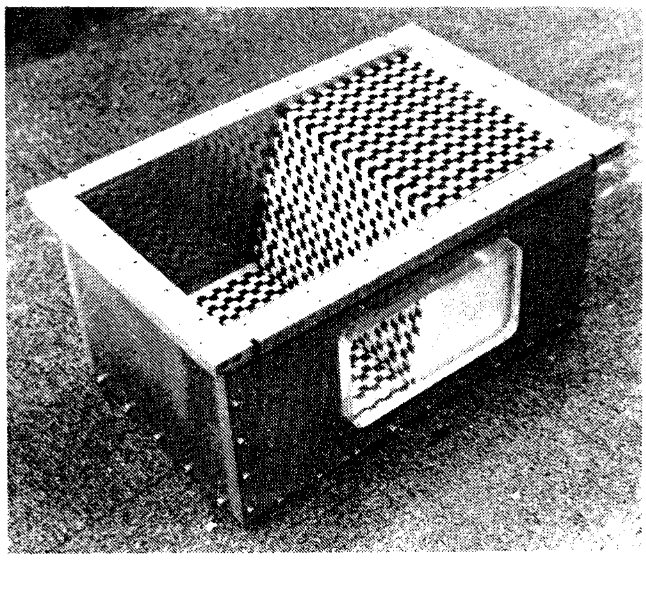


Fig. 13 Model arrangement of 'brick wall pattern'

The model rock slope had a free height of 275 mm, representing a prototype height of 55 m at 200 g. The stress level at the level of the toe of the slope was 0.52 MPa, with no surcharge, and 1.12 MPa with a surcharge of lead shot on the upper surface of the rock slope. These stress levels are probably insufficient to cause crushing of the model blocks near the toe, where the confining effect of adjacent blocks will increase the compressive strength beyond 0.75 MPa. No failure of the slope was observed, even with the full surcharge applied at an acceleration level of 200 g.

As predicted the initial model test did not lead to failure of the slope, mainly due to the high strength–to–weight ratio of the model blocks. However, variations in the geometry of the block assembly – for example from a vertical 'brick wall' assembly that is interlocked in two directions to, say, sloping stacks of blocks – will quickly convert the problem from one approaching a continuum to one approaching that of a simple sliding or toppling block on a plane. In these types of problems it is not the intact block strength that controls stability but rather the peculiar disposition of the mutually intersecting discontinuites. This is the type of problem that will be studied with the centrifuge. Here, the 'weights' of the blocks formed between the discontinuities may be increased by acceleration until a block type of failure mechanism results.

The possible failure mechanisms are in fact quite numerous and very complex, because each usually involves a multiple, and successive, block collapse, with each block capable of a distinct rigid body motion. To monitor these mechanisms, the centrifuge set-up allows video recording through a side window, laser scanning measurement and stereo-photogrammetric displacement profiling of the slope face during flight.

The types of problems to be studied are very difficult to simulate numerically and almost impossible to study in the field. This is especially true when the influence of model reinforcement is added. Also, in other physical modelling methods, it is very difficult to obtain correct similitude for the model reinforcement stiffness. It is hoped that this can be avoided at the high stress levels achieved by centrifugal modelling.

The analytical and numerical modelling methods include simple force-displacement style analytical procedures, finite difference, finite element, boundary element, discrete element and hybrid computational schemes. When these procedures are applied in rock mechanics, two fundamental problems exist in assessing their validity. The first concerns correctly describing the rock mass structural geometry of the field problem, the second concerns correctly describing the geometrical response of the rock mass to excavation. It is difficult, if not near impossible, to achieve an appropriate precision in these areas simultaneously. Thus, the validity of some discontinuum procedures is still to be verified for design purposes. The only rigourous way of checking the predictions of these methods is to carefully fabricate and construct a precise model, then subject that model to a uniform acceleration field and measure its response accurately. Verification of these numerical procedures in the area of slope design will be attempted in the centrifuge at UWA.

9 MINING-INDUCED SUBSIDENCE

9.1 Introduction

Ground subsidence can lead to expensive and potentially hazardous damage to surface and near surface structures: for example, the cracking of buildings and dams and rupturing of pipes and services. Furthermore, damage to lining systems employed in tailings ponds or landfill depositories may result in the escape of leachates which could pose a threat to ground water supplies.

Of particular interest to the resource development industry of Australia is the problem of mining-induced subsidence resulting from the extraction of minerals at depth. It is well known that the final surface disturbance resulting from mineral extraction will depend on the overlying strata and depth of soil cover. For example, expressions such as escarpments, plug settlement zones or areas of general subsidence may occur.

The aim of the proposed research is twofold and involves:

- a fundamental study of the propagation of discontinuities through a soil deposit, and
- the modelling of site specific cases for use in the calibration of numerical and predictive models.

9.2 Fundamental Study

In this study a series of centrifuge model tests will be performed to investigate the mechanisms involved in the propagation of induced boundary displacements through a soil deposit, and the resulting surface and near surface disturbances which result. In particular the following factors will be investigated:

- the effect of soil cover thickness
- the effect of stratified soil layers of differing properties
- the effect of different modes of induced displacement (i.e abrupt 'fault' type displacements which contain discontinuities of both slope and displacement, and displacements which contain discontinuities of slope only).

A versatile 'trap-door' displacement system will be mounted at the base of the centrifuge strong box such that a false floor containing an arrangement of rotating flaps and solid units will permit various modes of base deformation to be induced. The development of displacement and strain fields within the model will be obtained by image processing of the recorded movements of discrete markers placed in the model.

9.3 Site-Specific Modelling

An application in collaboration with the Geomechanics Division of the CSIRO has been made to NERDDC to fund a separate project where a series of centrifuge tests will be performed to model coal extraction techniques. During this study, specific mineral extraction processes, which allow for the controlled collapse of subterranean workings behind the advancing face, will be modelled. By modelling site specific cases in the centrifuge, it will be possible to calibrate numerical models formulated to predict surface disturbance in a controlled environment, and thus their application to field situations can be made with greater confidence. The Geomechanics Division of the CSIRO have a strong record of involvement in this area, and have access to surface subsidence records from coal mining areas which will permit comparisons with the centrifuge model tests to be made.

The two projects outlined above will be run in parallel, and it is anticipated that the data obtained from the fundamental study will be incorporated into the development of the numerical and predictive models used in the site specific study.

10 SUMMARY

This paper has described the new geotechnical centrifuge facility at The University of Western Australia, and summarised the various research projects that are currently being undertaken using the centrifuge.

In addition to the projects already described, there are many other areas where centrifuge modelling may prove useful. The centrifuge, by its nature, is ideal for any problems where self-weight loading is important. Such problems may involve static loading, as in retaining walls, reinforced soil, soil nailing, and the stability of slopes and excavations, or may involve dynamic loading where inertial and flow effects become important.

Centrifuge modelling has played a key role in research on earthquakes, and events such as blast loading, where inertial effects are critical. There is also a useful role to play for problems involving flow of granular materials, for example in the mining industry (transport, stockpiling and retrieval of ore and waste material) or in silo design.

The range of geotechnical problems where centrifuge modelling may prove useful is enormous, and it is likely that other geotechnical centrifuges will be established in Australia over the next few years. In the meantime, the centrifuge at UWA is seen as a national facility, and it is hoped that researchers and design engineers throughout Australia will make use of the facility.

11 REFERENCES

ARULANANDON K., TOMPSON P.Y., KUTTER B.L., MEEGODA N.J, MURALEETHARAM K.K. AND YOGACHANDRAN C. (1988). Centrifuge Modelling of Transport Processes for Pollutants. *Journal of Geotechnical Engineering*, ASCE, 114 (2) 185-205

DeBEER E.E. & WALLAYS M. (1972). Forces Induced in Piles by Unsymmetrical Surcharges on the Soil Around the Piles, Proc. 5th ECSMFE, Madrid, Vol 1, p 325 - 332.

GEIDANS E.P. & KILVINGTON D. (1984) Shear Strength of Estuarine Muds of the Swan River, Proc. 4th ANZ Conf. on Geomechanics, Perth.

GIBSON R.E., ENGLAND G.L. & HUSSEY M.J.L. (1967). The Theory of One-Dimensional Consolidation of Saturated Clays. I. Finite Non-Linear Consolidation of Thin Homogeneous Layers. *Geotechnique* **17**, No.3, 261-273.

GIBSON R.E., SCHIFFMAN R.L. & CARGILL K.W. (1980). The Theory of One-Dimensional Consolidation of Saturated Clays. II. Finite Non-Linear Consolidation of Thick Homogeneous Layers. *Canadian Geotechnical Journal* **18**, No.2, 280-293.

POULOS H.G. (1973). Analysis of Piles in Soil Undergoing Lateral Movement, JSMFE ASCE, Vol 99, No SM5, p 391 - 406.

RANDOLPH M.F. & HOULSBY G.T. (1984) The Limiting Pressure on a Circular Pile Loaded Laterally in Cohesive Soil. *Geotechnique* **34**, No.4, 613-623.

The geotechnical consultant

A B Phillips, B J Douglas, R Fell, M McMahon, P J N Pells, C Thorne, R Turner, B Walker

Introduction

Geotechnics, or geomechanics, is that branch of engineering which relates directly to the ground: its natural behavior, the way it responds to artificially-changed conditions and the way it interacts with man-made structures.

Geotechnical input is required for most civil, structural and mining engineering projects, and yet the ground is highly variable and its behavior difficult to predict. This presents the geotechnical consultant with problems of imprecision and uncertainty which are unfamiliar to many of his, or her, colleagues in allied technical disciplines, and are not often appreciated by the layman.

To address the implications of this, the following paper has been prepared by members of the Sydney Group of the Australian Geomechanics Society.

It is hoped that it will clarify the public perception of the geotechnical consultant, place in perspective the services which can be supplied, explain why most geotechnical work has to pass through a number of basic stages, and provide useful advice on briefing, engagement and professional liability.

With technological advancement, our ability to predict the way in which the ground behaves in a wide range of circumstances has improved enormously over the last 20 or 30 years.

There is every reason to assume this process will continue in future with resulting financial benefits to the community as a whole, and an increased confidence among engineers and clients alike that the ground-related aspects of their projects can be dealt with effectively.

Geotechnical consultants

Many engineers have a limited understanding of geotechnical principles, since they have to apply them only rarely during their professional lives. As a result, geotechnical specialists have an important part to play in the planning, design and construction of most projects.

Geotechnical consultants come from a range of backgrounds including soil mechanics, rock mechanics, engineering geology and hydrogeology. Singly, or as a team of professionals they can investigate a site and provide advice on probable ground behavior under a specific set of circumstances. Geotechnical consultants need a good general knowledge of engineering principles, mining and civil construction methods, so they can communicate effectively with their colleagues and clients. They work in private consulting practice, in large contracting organisations and in government authorities and generally hold university degrees in civil engineering or engineering geology. Many also hold more specialised postgraduate qualifications. Even so, relevant experience is of paramount importance and can only be gained "on the job".

Briefing and selecting a geotechnical consultant

Many misunderstandings and problems arise through inadequate briefing of geotechnical consultants and poor communications during the work. It is not realistic to require that an investigation should find out "everything" about a site. Neither is it reasonable to instruct that specific holes should be drilled at specific locations to specific depths.

When briefing a geotechnical consultant, it is appropriate to:

- Describe clearly the project and define performance requirements, including structural loads, levels, tolerable ground movements, flow rates and other relevant criteria
- Seek the consultant's advice on the type of investigation that is needed, the amount of field and laboratory work, the overall time scale and the scope of the final report
- Request a written proposal setting out the services to be provided and an estimate of total costs and fees
- Provide all available survey and underground services information and inform the consultant of any restrictions which may be placed on his work.

The brief for a geotechnical investigation should always be flexible. Since so much depends on what is actually encountered on the site there must be scope to modify the program should conditions be different from those anticipated. Failure to incorporate this flexibility can result in a loss of quality, with potentially serious consequences if difficult ground conditions are encountered later.

Consultant selection based on cost alone, without a detailed brief and without any attention being paid to the proposed scope of work, will normally result in the consultant who proposes to undertake the smallest amount of work being appointed, that is, the fewest boreholes, least testing and least effort being put into understanding the site. This inevitably increases the risk of unexpected difficulties arising as the project progresses with a consequent risk of increased costs. When such costs occur they are frequently many times the saving on the geotechnical investigation.

For larger and, particularly, more complex jobs it is reasonable to obtain proposals from more than one consultant. This gives an opportunity for innovation in investigation method, but it is in the interests of the project that selection should be based first on technical merit and on cost only when two proposals appear to be equally sound. Under these circumstances it is appropriate to notify all consultants ahead of time, that they are in competition and preferably with whom.

It is always advisable to ask for a record of the firm's relevant experience and for details of key personnel who are likely to work on the job. This information should be considered alongside the technical and financial proposal when making an appointment.

Geotechnical investigation

To give more than general advice a geotechnical consultant normally has to undertake an investigation to gather specific information about the site within which development is proposed. The steps in an investigation may include some or all of the following, depending on the availability of information and the development requirements:

1. Form a model of the anticipated site geology from published information, field exposures, the regional setting and, in some cases, remote sensing data
2. From the known project requirements, select those matters which require definition such as excavation characteristics, foundation capacity, settlement behavior and overall stability
3. Confirm or modify the geotechnical model by exploratory drilling, excavation and perhaps the use of geophysical techniques
4. Obtain values for the engineering properties of the materials onsite and the groundwater regime from insitu and laboratory tests and observations.

Sampling and testing of the ground disturbs its natural properties, so interpretation of test results requires care and experience. Advances in insitu and laboratory testing are helping to overcome this problem. Even so, the most sophisticated, modern techniques, which tend to be too expensive for routine engineering work, are not totally reliable.

Site investigation is, however, the key to understanding most geotechnical problems. the scope and nature of the investigation and analysis required will depend on the characteristics of both the site and the proposed development. The advice of an experience geotechnical professional should be sought on these matters, since he will usually be in a better position to know local ground conditions and problems and the relative costs of different investigation methods. He will also ensure that investigation is directly relevant to the project in hand.

Holes which are drilled without reference to both the project and the site geology will often be of limited value, so planning is a critical stage in the investigation process. On larger, or more complex, projects it is often cost-effective to stage field work, so that general conditions on site are understood first and, later, more specific investigation is undertaken as the requirements of the project become better defined.

Recording of field data must be accurate. It should be realised, however, that even the best field report is a subjective interpretation of the facts which comprise only the samples recovered, the numerical results of tests and observations such as drill performance, or water levels at specific times.

Field staff should be able to notice when conditions are different from those anticipated, or when the finding appear to be inconsistent. Such circumstances may demand a modification of the investigation program. To react to this requires a professional understanding of the consequences for the project as a whole. It is not something that should be left to the drilling contractor, unless he has appropriate engineering qualifications and is also aware of the details of the job.

Even the most intensive and well-managed investigation samples only a small percentage of the total site. There will always be the potential for unforeseen geological factors to emerge later as "problems" when features not met in boreholes become apparent.

Data and analysis

Assessment of geotechnical data involves interpretation of the geology in relation to field and laboratory test results. Data is not always consistent so judgement and experience have to be exercised in deriving an appropriate geotechnical model for analysis.

The mathematical tools at our disposal are not perfect, but generally more precise than the quality of our site investigation data. Where data is unreliable, even the most sophisticated analysis is likely to give the wrong answer. Choosing an appropriate analytical technique, given the quality of the available data and the required accuracy of the final answers, is a matter for experienced professional judgement.

Reporting and design

The geotechnical report should cover all aspects required by the brief. It will not normally attempt to design civil or structural works, but should provide sufficient advice and parameters for design engineers to do so.

Regardless of the adequacy of the report, engineering design is often greatly assisted by experienced geotechnical input. This will reduce the risk of inappropriate solutions being selected and can streamline the overall design process considerably.

Some clients believe that once the geotechnical report has been presented any further geotechnical involvement should be unnecessary. In most cases, this is not so: interaction between the geotechnical consultant and the designers will often lead to better solutions being adopted than the alternatives which might otherwise have been considered. This is where cost savings can frequently be made.

Discussion of alternatives is normally part of the design process and discussion is generally more effective if people with different expertise participate. To reduce the possibility of subsequent contractual problems, such discussions should be sufficiently formal for the matters discussed to be recorded.

Tendering

Preparation of tender documents follows design. In addition to writing the site investigation reports, the geotechnical consultant is often the best person to define terms, procedures and quantities if no standard specification, or method of measurement, is being used.

A contractor will normally benefit by seeking geotechnical advice while preparing his tender, just as a client for a project with complex ground works will benefit from tender assessment by a geotechnical consultant. In either case, there is less chance of inappropriate techniques being adopted to deal with the particular ground conditions of the site. For the same reason it is often beneficial for a geotechnical consultant to be involved in precontract discussions with tenderers, since it is only at that time that the contractor's assumptions can be discussed in an atmosphere free from prejudice.

Construction

Construction, to the geotechnical consultant, is the last stage of investigation and need not be too late to influence design, or choice of construction methods, if an appropriately flexible contractual situation exists.

Excavation is frequently undertaken which produces large exposures of the site geology for the first time. The consultant's presence on site at this stage will enable design assumptions to be verified and assist with overcoming unexpected geotechnical problems, if they arise.

Geotechnical consultants are often asked to "certify" that the works will "perform satisfactorily", "be suitable for a particular purpose" or some similarly worded requirement.

This form of certification amounts to a warranty for which consultants are not insured. It is not, therefore, in the clients interests to require such certification. The consultant can, however, certify that work has been carried out generally in accordance with the design intent, or that testing has been carried out in accordance with the specified standards.

Performance monitoring is to be encouraged whenever possible, since it provides a check on the satisfactory behavior of the works as they are built, and because it warns of construction problems before they get out of hand and provides data which can be invaluable for other projects in the same area.

This interpretation of the facts is least often available to geotechnical consultants and yet is invaluable data if it can be obtained.

Limitations and risks

The accuracy of geotechnical engineering advice is limited by geological variations, sampling and testing difficulties, theoretical uncertainties about behavior and the amount of data that can be collected given a project's budgetary constraints. All of these lead to the possibility that actual conditions and behavior encountered may differ from those anticipated.

Risks can be reduced with more intensive and higher quality work, but never completely eliminated. The "risks" in his regard are generally those relating to construction costs, or reduced performance of a structure, rather than to overall stability or to life and limb, even though these situations can unfortunately arise.

It is in the client's interest that advice should not be overly conservative. Good advice, often leading to significant cost savings, can be given provided the potential limitations are accepted by all concerned and the risks are knowingly shared by those with financial interests in the particular project. Contractual arrangements which recognise the inherent uncertainties in geotechnical engineering and allow for adjustment to cope with variable conditions should be aimed for. If unexpected conditions are met on a site, immediate involvement of the geotechnical consultant will help to achieve a satisfactory solution to the problem.

The engineering profession as a whole is conscious of the conflicts imposed by owners who seek minimum costs of construction, but are ready to sue at the instant something goes wrong. It is appropriate that "risk tolerance" should be discussed with the geotechnical consultant as part of the initial briefing – this will enable the budget for the site investigation to be balanced against risk expectations.

Conclusion

Competent geotechnical input can provide significant cost savings during the course of a project, both at the feasibility and design stage, and during construction. The preparation of reliable and comprehensive site information will increase contractor confidence and reduce the risk of construction claims.

There needs to be recognition of the fact that geotechnical engineering is not an exact science and that a site investigation report cannot be a guarantee of trouble-free construction. Risks will, however, be very much reduced by engagement of competent, experienced personnel and a budget appropriate to the scale of work and the anticipated difficulties of the site conditions.

Acknowledgement

This paper has been produced by the joint effort of the following:

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R Fell (University of New South Wales)
M McMahon (McMahon Associates Pty Ltd)
P J N Pells (Coffey Partners International Pty Ltd)
C Thorne (Coffey Partners International Pty Ltd)
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B Walker (Jeffery & Katauskas Pty Ltd)

These authors formed a subcommittee of the Sydney Group of the Australian Geomechanics Society. The views presented by the authors do not necessarily represent the views of all members of the Society nor the companies represented. ❑

AUSTRALIAN STANDARDS ARE WE BEING SERVED?

In the course of their everyday work, most practising engineers rely in one way or another on the Standards and Codes published by Standards Australia (SA). The reason these documents are relied upon will vary according to the nature of the problem being addressed and on the individual or firm undertaking the task at hand. These reasons probably include:-

- a belief that the requirements of the Standard are mandatory
- comfort is gained from the belief that if the guidelines of the Standard or Code are followed, accepted reasonable practice is being adopted
- the recipe-book approach presented is convenient.

However, few of the users of our Standards and Codes are aware of the background to the documents, or even the process by which they evolve and are published. Over recent years concern has been expressed at various times by the Australian Geomechanics Society and elsewhere, that the process by which our Standards are produced may not always adequately reflect good practice, consistent with that which is or should be applied across our country. Australia is a large sparsely populated country with widely varying climatic and geological requirements to be addressed and accommodated in design. It therefore is essential that any document purporting to be an Australian Standard is genuinely applicable in all parts of the country, or at least recognises those areas to which it may not be applicable.

Where design processes are straightforward 'recipe-book' matters (if such things exist), this may not be a problem. However, in geotechnical engineering, judgement and local experience play a significant part in any design process. To try to incorporate such matters into a Code or Standard would be very difficult, if not impossible. It therefore is considered we have to be very circumspect about how codified we allow ourselves to become. Even more so, we must be conscious of the variable conditions with which we have to deal and avoid/ prevent the publication of Codes or Standards which fail to recognise the diversity of conditions on our continent. The converse of this is ensuring we do not end up with Codes or Standards which are sufficiently general as to be of little value to anyone except the legal profession when a dispute has arisen.

From involvement in several committees of Standards Australia, the writer has some concern that the existing process is flawed by the funding policies of Standards Australia. Under present SA guidelines, no funding at all is provided for committee members, so that not only do active participants to SA committees have to forgo considerable amounts of time (which may translate to income) they also have to meet direct costs of participation such as travelling expenses, accommodation and the like.

The result of this policy is that committees tend to be dominated by members from Sydney and Melbourne, due to their large population bases and relative proximity to each other. In this way if meetings alternate between Sydney and Melbourne, the costs of attending meetings are curtailed. Even so, there are many committee members who find themselves in the invidious position of not being able to attend "out-of-town" meetings because they or their organisation are unwilling to meet the direct costs involved.

The consequence of this is that continuity of committee participation often is restricted. Furthermore, it is believed there are many in the wider engineering community who have both the experience and time to be positive contributors to SA committees, but due to retirement or other financial restrictions, are unable to offer their direct services. On the Australia-wide scene, rarely does Western Australia, northern Queensland or the Northern Territory have the opportunity to contribute actively because of the distances and associated costs involved. However, these areas have geotechnical problems not encountered in south-east Australia, but which may not be incorporated in a prospective Standard through ignorance on the part of the authors of the document.

An additional disadvantage of this policy of non-funding of committees is that a number of committees end up being dominated by self interest groups whose committee members are fully funded to attend all committee meetings and "push" their particular barrow. Not only does this have the potential to result in a biased Standard, but technical merit may be compromised in order to satisfy the (perhaps conflicting) demands or requirements of these interest groups. As demands on our time increase and the costs of travel continue to rise, it is suspected the dominance of self interested parties in the preparation of Standards may increase, and technically "poor" Standards may result. Regrettably the content of many Standards is often made mandatory, and if we are not careful we may find we end up with mediocrity which has become compulsory.

Balanced against this view that the present system does not encourage Australia wide participation, by those most suited to the preparation of a technically relevant document, the view of Standards Australia should be put. It is understood Standards Australia is a Secretariat incorporated by Royal Charter to maintain an unbiased position in the market place. Its funding is principally from publication sales and membership subscriptions with only nominal funding from the Federal Government.

It is further understood that Standards Australia takes the view that Standards are written by industry for themselves, and therefore those who will "benefit" most from the Standards should pay for, or at least contribute significantly to their preparation. The Committee process established by Standards Australia is designed to allow affected industry bodies to be represented, and with representatives who are able to both reflect the technical expertise required and the relevant interests of that body. Unfortunately, this is too altruistic in our real world, and it is considered many committees end up not having a fair representation of the community's interests and technical expertise.

No immediate solution to this perceived problem is apparent. However, it is considered the Australian Geomechanics Society and more generally the Institution, should periodically reflect on the way in which our Standards are produced. If we as a profession are united in wanting reform of this system, some rationalisation of it may be possible.

N. de Plume

M.AGS

Ed. Note

An issue not raised in the above letter is the current Standards Australia practice in respect to Public Review of Draft Standards. No longer does SA automatically forward copies of public review drafts to interested subscribing members, but rather the availability of a Public Review Draft is published in "The Australian Standard", the monthly publication of SA, and a charge is made for the supply of a draft (the draft site investigation code costs $20). An obvious consequence of this state of affairs is that only a limited number of people become aware of the existence of a draft and even less go to the trouble of paying for a copy to review. This could easily result in a limited public review process, with the subsequently published Standards inadequately reflecting the views of the profession. This is particularly pertinent, given the concerns raised by Mr de Plume regarding representation on committees.

We would welcome any letters from interested readers on their experiences, good or bad, in relation to Australian Standards.

Engineering Principles of Ground Modification

by Dr Manfred R. Hausmann

Published by McGraw-Hill Book Company

This new book by Dr Manfred Hausmann, of the University of Technology, Sydney, deals broadly with the topic of ground modification and covers such topics as compaction, hydraulic modification, dewatering, physical and chemical modification (including stabilization and grouting) and modification by inclusions and confinement.

The more conventional topics, such as compaction, dewatering and preloading are given a thoroughly modern and comprehensive treatment, however I felt that the real strength of the book was that it broke new ground by dealing with a wide range of specialist topics and modern ground modification techniques. Such topics include soil reinforcement, the use of geosynthetics for drainage and the reinforcement of embankments, pavements and railway foundations, soil nailing, thermal modification, ground freezing, electrokinetic dewatering, and the design of crib and gabion walls. Today, geotechnical engineering covers a wide range of disciplines, and it is becoming increasingly important for engineers and engineering geologists to have access to specialist texts and for this reason alone, I am most pleased to have a copy of this book on my bookshelf.

Although written for an international readership, one pleasing aspect of the book is that it has an Australian flavour, with references to Australian Standards, case studies and experience. Where Australian experience is lacking, examples are given from overseas and this is often helpful when deciding which of the new technologies may be successfully introduced into Australia.

At the end of each chapter, there are numerous example questions and problems (with answers) so that the reader can test his knowledge of the preceding material. I have had experience of the use of the book with students, and having had to work out the solutions myself, have found this a most useful way of learning new design techniques.

In summary, I found this a most useful and interesting book, and at the price of $123.95 for the hardback edition, I would recommend it to any practicing engineer. At over $100, it is unfortunately out of the price range of most students, however if it appeared in a softback edition, it would make an excellent text for assisting them with their studies and would be a book that they would want to keep after graduation.

John Small

The University of Sydney

NEXT ISSUE

MORE PAPERS - MORE FEATURES

The next issue of "AUSTRALIAN GEOMECHANICS" shall include feature papers on embankments on soft clay, expert systems in geotechnical engineering, side resistance on piles and pile design by authors such as Poulos, Lee, Small, Goh, Tan and Johnston.

We will also feature a review of the issues raised by the "Coledale Case" as well as the regular features:

* GEONEWS
* BOOK REVIEWS
* GRAVEL RASH
* PRESS INTERFACE
* GEODIARY
* NATIONAL COMMITTEE REPORT
* GROUP REPORTS

CONTRIBUTIONS

Contributions are welcomed to any of these regular features. Please try to meet the deadline and format details set out on page 2.

LETTERS TO THE EDITOR

The editorial panel will welcome contributions in the form of letters on any topic related to the practice of geotechnical engineering, Many topics evoke much private discussion and complaint. Avail yourselves of a much wider audience and set out to change or incite by jotting down a few lines in time for the next edition. If you really don't have anything else to write about then just tell us what you think of ***Australian Geomechanics.***

WASTE MANAGEMENT FEATURE

The June 1991 issue of Australian Geomechanics will include a feature on waste management and related aspects of environmental engineering.

This area of geotechnical engineering is growing in Australia as we discover the legacies of some of the mistakes of the past and set out to better control the products of our present industrialisation.

These aspects present a major challenge for our generation and the life style of future generations may well depend upon just how well we handle this challenge.

Articles, papers, views, case histories and general comments are sought for this special edition on the technical, legal and liability aspects of this very significant topic.

Anyone wishing to submit a major article is asked to forward a brief abstract to the Editorial Panel by 28 February 1991 so that the panel may ensure that a wide range of aspects of the topic is presented.

ADVERTISING

To ensure that Australian Geomechanics may maintain a high standard of production we are actively seeking the support of advertisers. If you are involved in the general geotechnical scene, support "Australian Geomechanics" and get your message across to MOST of the geotechnical practitioners in Australia.

For rates and other details please contact Peter May c/- I.E.Aust, 11 National Circuit, BARTON, ACT. 2600. Telephone:(06) 270 6555 or Facsimile:(06) 273 1488

BOOK REVIEWS

Geotechnical Engineering Techniques and Practices

by Roy E Hunt

Published by McGraw-Hill Book Company, 729pp.

Also titled Geotechnical Engineering Analysis and Evaluation

Recently, I was asked the following question:

"If you had to go to a deserted tropical isle and you could only take one reference book on geotechnical engineering, what reference would you select?"

Books like Winterkorn and Fang (1975), Tomlinson (1980) or even Bowles (1982) immediately spring to mind. A more recent candidate is a book published in 1986, written by Roy E Hunt entitled 'Geotechnical Engineering Techniques and Practices'.

The author, Roy Hunt, received his B.Sc. in Geology from Upsala College in 1952 and his M.A. in Soil Mechanics and Foundation Engineering from Columbia University in 1956. He has worked variously as a project geologist, chief of a soil mechanics laboratory, project engineer, and the principal of a firm. His work experience has encompassed such diverse projects as dams, roads, foundations, tunnels, large land area development, environmental conservation, retaining structures, and slope failures.

His book, published by McGraw-Hill, is divided into three major sections covering (1) engineering properties of geological materials, (2) site planning, ground improvement, earth structures, and foundations, and (3) slopes, retaining structures, and excavations. The book's strength lies in its elucidation of the techniques and practices applicable to the many scenarios and problems likely to be encountered in geotechnical engineering. The book contains a wealth of information within its 800 odd pages on many aspects of geotechnical design and evaluation. The person expecting to find a "cookbook" approach to engineering design may be disappointed.

The book more correctly serves as a convenient starting point for design and supplies a useful reference list should further detailed information be required.

This is one of the few books that I have come across where rock and soil are treated as equals in geomechanics. So many texts that claim to be GEOtechnical mainly concentrate on one, usually soil, and pay lip service to the other. This is symptomatic of, firstly, the general division of geomechanics into the two related sciences of rock and soil mechanics, and secondly, the fact that most geomechanists (and hence potential authors of geomechanics texts) become experts in only one of the two sciences. Perhaps, the engineering geology background of the author (Roy Hunt) and his varied work experience have provided a more balanced view of geomechanics. Such an outlook is to be commended and encouraged amongst all practitioners.

The figures and illustrations throughout the book are full of information; perhaps too detailed in some cases as the author seeks to convey as much information as possible in the available space. I like the section on lateral pile analysis, a convenient and useful summary, and the section on retaining walls is a good one. The description on laboratory testing of soil and rock is informative although I found that this section made for dull reading unfortunately.

One aspect that annoyed me throughout the book, is the different units used (e.g. tsf, ksf, kPa, kg/cm^2, psi for stress), which appear to be derived from whatever reference source it was that the original information was obtained. I know that we live in a world where different systems of units are used, and tolerance is a virtue to be admired in an individual. That being stated, it may have shown foresight to have standardised the units, as in SI units (Pa, kPa, MPa) and American Imperial (either tsf or psi, not both). Adoption of SI units is recommended as it is a recognised international standard. American Imperial should be used only to ensure sales of one's book in the U.S., since it serves no other useful purpose. This last statement shows that humility and tact are other virtues to be admired in an individual.

I wonder why the book has two different titles. The dust jacket gives the title as 'Geotechnical Engineering Techniques and Practices'. The inside cover page has 'Geotechnical Engineering Analysis and Evaluation' as the title. Both titles accurately describe the book's content. Perhaps that's something for Mr or Ms McGraw-Hill to explain.

The book is a useful one to have, for both the young graduate and the 'not-so-youthful' experienced practitioner. Published by McGraw-Hill Book Company, the book is available from all discerning bookstores. EA Books price as of 19/4/90 is A$155 plus $6 postage. It's a bit pricy for a personal copy (???), so get the office to buy one (but then again you get a book with two titles for the price of one). It may not be THE geotechnical text to take with you onto a deserted tropical isle, but then again I haven't come across too many recent texts either.

Elio Novello.

PROFESSIONALS IN THE DOCK

THE COLEDALE EXPERIENCE TO BE REVIEWED

The legal aspects of the "Coledale case" ended when the four accused employees of the NSW State Rail Authority (SRA) were found not guilty of a variety of criminal charges. The charges arose from a 1988 mudslide in which a railway embankment collapsed onto a house and killed two people.

Among those acquitted was Mr David Christie, the principal geotechnical engineer with the SRA. The case has therefore created more than a little interest among geotechnical engineers across the country. The premature end to the case has been welcomed, but the principal questions regarding the reasons why Mr Christie was charged in the first place have still not been answered.

The case has come at a time of increasing awareness and concern throughout the engineering profession in relation to professional liability. Many geotechnical engineers are acutely concerned about the issues raised by this case and the increasing responsibilities which society is pressing onto professionals.

So that the issues may be better understood, we have solicited an in depth analysis of this case from prominent practitioners who have followed the saga closely. This analysis will appear in a future issue of "Australian Geomechanics".

GEONEWS,
PEOPLE AND ORGANISATIONS

RECENT APPOINTMENTS

Keith Seddon has joined consulting geotechnical engineers M.P.A. Williams and Associates Pty Ltd, based in their Melbourne office. He will work in the areas of mine waste disposal and groundwater, as well as more conventional geotechnics. Keith was previously employed by Pak-Poyand Kneebone Pty Ltd.

Queensland Group extends its congratulations to two members:

Peter Hollingsworth on his election as Vice President of the Institution of Engineers, Australia.

and

Professor Ted Brown on his appointment as Deputy Vice-Chancellor at the University of Queensland.

Professor Harry Poulos has this year become ISSMFE's Vice-President for Australasia. He takes over from John H.H. Galloway who held the position from 1985 to 1989.

Professor Poulos was appointed Professor of Civil Engineering at the University of Sydney in 1982, filling the chair made vacant by the death of Professor E.H.Davis. In 1984, he became Head of the School of Civil and Mining Engineering and Director of the Centre for Geotechnical Research at the University of Sydney. In 1989, he joined consultants, Coffey Partners International, as Director of Advanced Technology, whilst still retaining a Chair of Civil Engineering at Sydney.

He has written over a hundred papers and three books and is a world authority on the behaviour of piles and piled foundations. In 1987 he was selected to present the E.H.Davis Memorial Lecture by the Australian Geomechanics Society. In 1988 he received the same society's quadrennial John Jaeger Memorial Award. In 1989 he was invited to give the 29th Rankine Lecture for the British Geotechnical Society.

AUSTRALIAN GEOMECHANICS INDEX

A cumulative index 1971-1990 for Aust. Geomechanics Journal, Aust. Geomechanics News and Aust. Geomechanics is now available on DBase III. Copies can be obtained from Robert Smith, Senior Geotechnical Engineer, Australian Construction Services, Tivoli Court, 239-241 Bourke Street, Melbourne, Victoria 3000. Telephone (03) 652 8282..

AS 3725 - LOADS ON BURIED CONCRETE PIPES

The new Australian Standard AS 3725 was introduced at a series of seminars held throughout the country in June and July. The seminars were jointly sponsored by Standards Australia and the Concrete Pipe Association. Changes introduced in the new standard include higher bedding factors which permit the use of higher fills and larger diameter pipes.

CHANGE OF ADDRESS

Members are requested to notify the Australian Geomechanics Society, as well as the Institution, of any changes of address.

CONFERENCE WATCH

Details of forthcoming conferences etc are given under the GEODIARY heading, but a few conferences of note are worthy of special comment.

The conference with the most appeal for some will surely be the International Seminar on Soil Mechanics and Foundation Engineering of Iran to be held in November 1990. The theme covers the usual range of good soil mechanics topics but the attraction lies in the fact that the seminar organisers undertake the travel and accommodation expenses of individuals whose papers are accepted for oral presentation. This should surely ensure a major task for the reviewing committee. Unfortunately the due date is now past.

The award for the conference with the most unlikely topic for the particular venue must go to the May 1991 Conference on Computer Methods and Advances in Geomechanics. One of the topics for discussion is Ice Engineering which should provide a pleasant diversion from the sunshine of the Cairns coast.

The 'BIGGEST' award goes to The Second International Symposium on Mine Planning and Equipment Selection in Calgary, Canada. The advertising lists 25 distinct topics on almost every conceivable aspect of mining and geotechnical engineering. If you miss out on a free ticket to Tehran you could hardly miss acceptance here on the grounds of non-conformity with the theme.

For a trip nearer to home, the Ninth Asian Regional Conference on Soil Mechanics and Foundation Engineering to be held in Bangkok on 9-13 December 1991 may be attractive. However, for anyone wanting to present a paper at the conference, abstracts had to be submitted not later than October 1990!

For those who must limit the conference round to Australian shores, don't forget the conference on Engineering in Coral Reef Regions to be held on Magnetic Island (4-7 November 1990). The latter definitely seems to have a certain attraction.

We anticipate that the next issue will include reports from one or other of the Australian delegates to the Retaining Wall conference in New York held in June and the Glasgow conference on Reinforced Soil to be held in September. Brief (or not so brief) notes on conferences will be welcomed for inclusion in future issues.

LIBRARY FACILITIES

All publications received by the Society are placed in the CSIRO Division of the Geomechanics Library, PO Box, Mt Waverly, Vic 3149. Tel:(03) 881 1355, Fax:(03) 803 2052 . Copies of the library accession lists are sent periodically to the secretaries of Geomechanics Society groups in each state. Books are available for loan to Society members and photocopies of papers or extracts may be obtained on request for a fee.

NATIONAL COMMITTEE MATTERS?

As I trust all of you are aware, the affairs of the Australian Geomechanics Society are organised and co-ordinated, on a national basis, by the so-called National Committee. This committee is composed of one ex-officio and 19 elected members, and its membership is currently as follows:

Chairman [1]	Dr Neil Mattes
Deputy Chairman [1]	Max Ervin
Immediate Past Chairman	Dr Peter Mitchell
A'asian Vice President ISRM	Prof Ian Johnston
A'asian Vice President IAEG	John Braybrooke
A'asian Vice President ISSMFE	Prof Harry Poulos
AusIMM Nominee #1	Denis McMahon
AusIMM Nominee #2	Frank Kaesehagen
IEAust Nominee #1	Garry Mostyn
IEAust Nominee #2	Mike Thom
Victorian & O'seas Members' Reps	Tom Flintoff
	Marc Kurzeme
NSW & ACT Members' Reps	Bob Carr
	Dr Tony Phillips
Queensland Members' Rep	Paul Wallis
SA & NT Members' Rep	Brian Richards
Tasmanian Members' Rep	Tom Bowling
WA Members' Rep	Charles Waterton
Secretary (ex-officio)	Peter May.

The National Committee meets twice a year, usually alternating between Melbourne and Sydney. The matters discussed by the committee are wide ranging and the meeting lasts an entire day. (Even then, some matters receive less than the airing they deserve, particularly after lunch!). In view of the important, but generally unseen role of the National Committee, the Editorial Panel has decided to include a regular column in Australian Geomechanics with discussions, outlines, etc. of those items raised at the previous National Committee meeting. Matters to be raised will be those that may be of interest to the general readership of this magazine or, alternatively, may require an airing that is more widespread than is generally available through the medium of the National Committee meeting minutes! A typical example of such a matter would be discussions relating to Australian Standards. The article on this subject in this issue indicates the relevance of this particular topic.

<u>Matters Arising from National Committee Meeting on 6 April 1990</u>

o Membership of the AGS is currently 730 and initiatives to attract new members are under consideration.

o A Five Year Plan setting out the objectives and future directions of AGS has been prepared and distributed to State Groups.

o John Braybrooke (Regional VP, IAEG) will undertake to foster unification of engineering geologists, who currently are variously represented by AusIMM, AGS, Geological Society of Australia, Assoc. of Engineering Geologists, etc.

o Nominations have been called for the 1991 W H Warren Medal and for the 1991 E H Davis Lecturer.

o The AGS will continue to provide its membership list (for a fee) to "appropriate" organisation for direct mailing purposes.

o A Bid Document is now being prepared for Melbourne's bid for the XIV ICSMFE in 1997. The committee agreed that the theme of the Australian bid should include"The Decade of Disaster Reduction". The bid Coordinator, Max Ervin, welcomes any input or suggestions for format of the bid, which should be ready by November 1990.

o Planning for the joint AGS/Structures Panel conference "Structures in the Ocean", which is currently scheduled to be held in Perth in 1992, has lost a good deal of momentum and the conference appears to be in danger of becoming a "non event".

o Tentative bookings have been made in Adelaide for the staging of the Sixth ANZ Geomechanics Conference in Adelaide in 1996.

o Next National Committee meeting will be in Sydney on 5 October 1990.

Should anyone have any comments/queries on the matters reported above or if there are any matters that you would like brought up at the next meeting, please contact your state representative or the National Secretary.

[1] Elected by last year's National Committee

PRESS INTERFACE

The Engineering profession, along with the other professions associated with the Geosciences, suffers from a lack of public awareness of significance of the role we play in society. This is reflected by the status which is afforded those dabbling in the "earth sciences", compared to that afforded to Doctors, Accountants and Lawyers for example.

Various and continuing efforts are being made by professional bodies to rectify this situation with apparently limited success. One such effort on the part of the Western Australian Division of the Institution of Engineers involves informing the media about forthcoming technical meetings and other events.

In the latter months of last year, this particular project at least resulted in a response from the press. Len Findlay's regular column in The West Australian bore the headline **"ENGINEERS GET THE URGE TO INTERFACE"**. Len then led into a "very tongue in cheek" article by referring to Engineers' apparent need to be loved and understood being demonstrated by the fact that the local "Institute" (He didn't even get the name right) of Engineers had deigned to send to the press "flyers" advertising forthcoming meetings. The Structures Panel's - *"Mr Trevor Osborne's delightful The Interface of Structures With Ground With Particular Regard to Retaining Methods"* and Geomechanics Society's - *"Professor C.P.Wroth's snappy address on Control of Ground Movements Around a Large Excavation in London"* both rated special comment.

Mr Findlay did go on to say *"For my part, I think it would be a shame to miss either."* However he did not appear to be in the audience for either performance.

This article was followed up a week or so later, by another article written in response to a mass of letters which Mr Findlay received. This time we Engineers again bore the brunt of Mr Findlay's wit.We are acknowledged as being *"wonderful, imaginative, talented, hard working, skilful and, for all I know, underpaid."* (We don't disagree with Mr Findlay on any of these points.) However by this stage our columnist had concluded that Engineers are also ***"thin skinned."***

In all of this Mr Findlay appears to deny that there is a need for journalists to be better informed about our professions. It has therefore been interesting watching the press in recent times for clues to suggest that the nation's'" journos" are really so well informed that they have nothing to learn from geotechnical practitioners.

This isn't all that rewarding. About the only time matters to do with geomechanics make the press, is when there is a problem. The NSW State Rail Authority employees' court case gained quite wide coverage, as has a series of mine failures in recent times.

One such item published in The West Australian recently reported upon a slope failure in a gold mine in Kalgoorlie.

Under the headline **"MEN DASH CLEAR OF PIT LANDSLIDE"** we are informed of the lucky escape for a *serviceman* and a *driller* as about a million tonnes of ore collapsed onto a drill rig.

The article concluded by reporting that "a rock mechanic" was on his way to the scene. This description conjures up a delightful picture of our man in grease stained overalls, spanner in hand, rushing to the site to put the pieces of rock back together and hopefully getting the "darned thing going again" before too long.

The news isn't all bad of course, as evidenced by Carmelo Amalfi's recent article, also in the West Australian, under the headline **"ROUND ABOUT WAY TO MAKE BUILDING SAFER"**. Carmelo has presented a well written and informative few paragraphs about the newly installed geotechnical centrifuge at the University of Western Australia.

The article was informative even for those of us "in the trade" particularly to learn that the demand for centrifuge modelling is so high that *"it had been booked out for the next eight months at a daily charge of $1000 to $2000."* With this revenue ($258,000 calculated), the unit certainly represents an excellent investment for the hallowed halls of UWA.

This leads to the point of this column and the fact that we invite submission of articles "of interest" concerning the geosciences which may be gleaned from the national press. Please forward your cuttings to the Australian Geomechanics Editorial Panel for inclusion in future editions.

"Fin Lendlay"

GRAVEL RASH

All engineers would be aware that the status of the profession in the eyes of the public is not all that it could be. (However, there is an interesting contradiction between the dropping of engineers from the list of professions that can sign passport applications and the criticism that was levelled at Australia's immigration policy recently that we only take "engineers , doctors , etc." from Thirld World Countries - It would seem that Australians recognize the status of Third World engineers above those practicing here).

The Institution is currently endeavouring to raise the status of engineers and this involves, to a considerable extent. the improvement of the image of the professional engineer. A major component of this is the projection of the practice of engineering in a positive light, in a manner that can be appreciated by the general public.

Several different strategies have been implemented to carry this out and there has even been a suggestion that the IE Aust might produce a "soapie" tentatively called "The (Young?) Engineers"! This might seem a little over the top, but no one could argue that "Young Doctors" (which my daughter, who is a nurse, claims to watch for "educational reasons"!) does anything but enhance the public profile of the medical profession. So maybe we can look forward to "living and loving" with the team at the sewage treatment plant, or even "real life human drama" in the core logging shed on a remote dam site!

Another measure of our public persona can be gauged by references to our profession in the literature. Medical doctors, architects, lawyers, teachers, accountants abound, but where are the geotechnical engineers? My only recollection of a geotechnical profession cropping up in a novel is a book called "Orange Wednesday", written by Leslie Thomas, who also wrote "Virgin Soldiers". There was an Israeli geotechnical engineer in this book, but the profession was hardly presented in the best light (imagewise) because his wife had an extended affair with the hero, with the engineer being portrayed as very boring, preferring to be scratching around in the desert rather than being with his wife!

All this notwithstanding, I believe that we in the geotechnical profession have a particular duty to "lift our game" and present our important, interesting and sometimes even exciting profession to the general public. I am sure we have all experienced the blank glazed look that comes over people's faces when the following drama is enacted:

GLVFLG[1]: (in response to EXTREMELY funny joke told to her by Witty, Sophisticated, Bright, Good Looking, Young Man ("WSBGLYM"))

"Ha - Ha - Ha - Ha - Ha" (+ tears rolling down her cheeks)

"Tell me, Pete/Bill/Fred/Ian/etc...." (Delete where appropriate)

".....What do you do for a living?"

WSBGLYM:(proudly) "Oh, I'm a geotechnical engineer"

GLVFLG:(with puzzlement) "What's that?"

WSBGLYM proceeds with explanation

GLVFLG:(with boredom) "Oh, that's nice...Must go now, Bruce has invited me to go and watch some paint dry!"

(Exit stage left, yawning).

So what are we going to do about it?

---

1 "Gorgeous Looking, Vivacious, Fun Loving Girl"

STATE GROUP REPORTS

QUEENSLAND GROUP

Report of the March Meeting

At a combined meeting with the Geological Society of Australia, Jenny Hacker and Peter Reman presented a paper on Recent Coastal Sediments and Climatic Changes. Recent development of the coastal plan has highlighted the variability of coastal sediments. The authors showed that much of this variability is due to sea level change resulting from climatic shift. The most recent shifts resulted in low sea levels at around 18 000 BP and 135 000 BP and high sea levels at the present time and 120 000 BP. Sediments deposited during the last inter-glacial when sea levels were high were subjected to sub-aerial weathering during the following period of low sea level. In contrast, those that were formed since the last low sea level have not undergone such substantial change. Sediments deposited 5 000 to 6 000 BP when sea levels were 1 - 2m higher than present show some significant differences compared to other recent sediments. The authors pointed out that knowledge of the details of recent coastal sedimentation in Queensland is still incomplete and that thorough investigation is desirable from both a geological and geotechnical point of view

Future Meetings

MAY:

Mini Symposium - "Earthworks Compaction"

Thursday 17 at 4.30pm.

JUNE:

"Geomechanics Japanese Style"

Monday 25 at 6.15pm - Dr D Williams.

Venue: Hawken Auditorium.

JULY:

"The Geotechnical Engineer and the Law"

Thursday 19 at 6.15pm

Messrs J Doyle, D Starr.

Venue Hawken Auditorium.

AUGUST:

Dinner Meeting - Topic & Speaker TBA.

Thursday 16 at 7.00pm.

Venue Johnsonian Club.

OCTOBER:

Mini Symposium - "Geomechanics and the Environment"

Thursday 18 at 4.30pm.

Venue Hawken Auditorium.

NOVEMBER:

Annual General Meeting - Retiring Chairman's Address

Thursday 15 at 6.15pm.

Venue TBA.

DECEMBER:

End of Year Dinner

Thursday 6 at 7.00pm.

Venue TBA.

VICTORIAN GROUP

Report of Meetings

SAA Draft Earthworks Code

14 February 1990

M Ervin of Golder Associates outlined the provisions of the draft Australian Standard DR 89204 "Guidelines on Earthworks for Commercial & Residential Developments". The document includes guidance on testing procedure and frequency, minimum standards of compaction and documentation.

Recent Experiences with Rock Stress Measurements Throughout Australia

14 March 1990

The two stress measurement techniques most frequently used by CSIRO Division of Geomechanics, overcoring and hydraulic fracturing, were described by J Enever.

The recording of very high stresses in the Newcastle district and in the south-west of Western Australia before the 1989 earthquake was of interest. Other measurements for new tunnels beneath Sydney and in Tasmania were reported.

Stabilized Clay Grouting and Application to the Australian Environment

11 April 1990.

S Hancock of Australian Groundwater Consultants outlined his Churchill Fellowship tour of the USSR and Eastern Europe where the technology is widely used. Mining examples from Australia were also described along with its potential application to waste containments.

Future Meetings

9 May 4-8pm

Seminar on the Design & Construction of Pavements for Container Handling Depots and Airfield.

(Date to be advised)

1989 E H Davis Memorial Lecture - M Khorshid

18 July

Beyond the Theory of Landslides T McKinley & I Pedler.

A joint meeting with graduates and students.

Venue Monash University

8 August

Draft SAA Piling Code - I Lee

5 September

Cable Bolting Theory & Practice P Fuller

10 October

Geomechanics Research from Monash University

14 November

Seminar "Investigation and Assessment of Contaminated Sites" followed by the Annual Dinner

Venue Melbourne University

TASMANIAN GROUP

Reports on Recent Meetings

Toxic Waste Disposal Seminar

12th March 1990

Speakers:Mr Frank Brown, Senior Chemist, Department of the Environment, Tasmania

Mr Alex Eadie, Engineering Geologist, Dames & Moore, Melbourne.

Mr Peter Stevenson, Engineering Geologist (recently retired) ex Department of Mines, Tasmania.

The Seminar was held at the Hydro-Electric Commission's Cafeteria and was attended by over 40 participants representing some quite diverse groups including waste disposal companies, councils and local environmentalists.

The speakers for the seminar had been briefed by Dr Fred Baynes in order that each would cover some fairly specific subject matter plus details of some distinct recent case histories after the buffet tea.

Frank Brown discussed the legal and political issues involved in the safe disposal of toxic wastes. He contrasted 'Toxic' wastes as a subset of 'Hazardous' wastes. In Tasmania these materials were covered in legislation in more than 60 acts which generally required updating to ensure consistency and the use of modern methods in dealing with these substances.

Frank presented details and examples of conflicts in the legislation which is currently in force. He outlined improvements which should be introduced and indicated that public involvement would demand the introduction of tighter controls on the disposal of toxic and hazardous substances.

Alex Eadie dealt with what he termed 'Hazardous Waste Clean-up Technology'. He described the sequence in which projects were handled, ie from 'investigation', to 'design', to 'tendering', to 'contract' and 'quality control and quality assurance'. In this process a range of different disciplines including engineers, geologists and chemists were involved.

Two options were generally available to treat a toxic waste disposal problem, ie insitu treatment or removal and treatment elsewhere. Insitu treatment options included 'volatilization', 'biodegradation' (usually with aeration to speed up the process, synthetic microbes were being used at times), 'leaching' and 'chemical reaction', (including soil washing), 'vitrification' (radiation wastes), 'passive remediation' (let nature run its course), and 'isolation and containment' (where water inflow was controlled and outflow barred). Non-insitu techniques were generally similar but provided more opportunities for solidification eg with cement, lime or other binding agents.

Practical problems associated with toxic waste disposal included the non-uniformity of contamination over an area, which often caused problems in 'representative' testing, and the concentration of toxicity which might be considered acceptable. The latter usually came down to a consensus of those involved including the 'owner', the 'workers' on the site, the 'government' and the 'public'.

Peter Stevenson briefly quoted some basic facts, ie the amount of waste generated by the metals industry in the U.S.A. currently comes to approximately 3m tonnes per year. He then detailed the geological requirements for a toxic waste disposal area saying that low permeability of the soil and rock mass was particularly important together with the hydrological aspects, ie low rainfall and little ground water (ie the converse of what is required for a water supply area). The disposal site should also be accessible, though secure, it should be on government controlled land in order that it can be readily managed and monitored for extended periods, possibly centuries. The site should also be aesthetically acceptable and its location should be well documented.

After the buffet tea break the three speakers presented details of case histories.

Frank Brown described the 5 B's of waste disposal:- Bury, ie storage in landfill, Burn, ie construction of a suitable incinerator which is currently being discussed for the disposal of medical wastes in Australia, Bog, ie disposal into the sewage treatment system, Barge, ie disposal at sea, and Bunker, ie establishing a storage facility such as is proposed for radioactive wastes. All the above disposal methods were being practiced to various degrees and all had their pros and cons.

Alex Eadie presented details of work at the former BP Oil Terminal site in Melbourne which was being developed as a canal residential area but was contaminated with hydrocarbons. He briefly described the investigation phase and the proposed method for cleaning and washing the sand to remove the hydrocarbons, the quality assurance system to be used and the acceptance criteria which were being specified.

Peter Stevenson described an investigation into suitable areas for disposing of toxic wastes in southern Tasmania. He stated that the tops of dolerite plateaus were generally considered good as they were capped with impermeable dolerite clay and were areas of low ground water movement. Deposits of similar clays at or near sea level also provided possible sites. Areas which had been assessed included the Styx River Area - deep dolerite clays but too far away, Snug Tiers, Mt Wellington and Mt Dromedary were too accessible to the public, Brown Mt was too far away, Margate tip - near sea level on deep dolerite clays, the University Farm near Richmond - below sea level with deep dolerite clay, and Sandfire Island in the Coal River Estuary.

Although the University Farm appeared to be one of the most suitable areas, and much of the toxic waste originated from the University, the NIMBY (not in my back yard) philosophy prevailed and the waste was being sent to Victoria for disposal, ie for the time being at least.

In the discussion which followed the presentations by the three speakers a number of issues were explored such as:-

The safety of transporting toxic wastes; not considered any more problematic than say the transport of substances like petrol and sulphuric acid.

Treating the problem at its source; ie required industry to reduce the amount of waste being produced or to recycle the waste products.

Problems with the establishment of a central national incinerator facility somewhere in Australia; ie would the 'nimby' syndrome tend defer such a project ad infinitum.

Problems in quality assurance and quality control of ensuring that contaminated soil is sufficiently cleaned-up; ie are a large number of samples required to satisfy clients and/or inspecting authorities.

Education of the public in the need to establish clean-up strategies and to counter the 'nimby' syndrome; it was noted that the public generally feared chemical names as opposed to common names, eg paint thinners is quite an acceptable substance, xylene and toluene are suspect.

Biological methods for treating toxic materials and the problems being investigated in this field.

The seminar concluded after a vote of thanks to the speakers was moved by Dr Fred Baynes.

As a postscript to the above seminar the Group Committee has decided to hold a special committee meeting to follow up on some of the issues raised by the seminar. It was felt the Group should seek to raise awareness amongst its membership of the wide range of opportunities which could be provided by a general community requirement for higher standards of waste disposal and environmental control.

CONCRETE FACED ROCKFILL DAMS

3 May 1990

Speaker: Mr Rick Boyle, Hydro-Electric Commission.

This meeting was held in conjunction with the Tasmanian Division of the Institution of Engineers and was chaired by Max Maynard, the current Division Chairman. The meeting was attended by approximately thirty members and visitors.

Mr Rick Boyle is the Commission's Construction Manager for the West Coast and is based at Queenstown. He has over twenty years experience in the construction of concrete faced rockfill dams in Tasmania.

In his lecture Rick Boyle presented an outline of the construction methods developed for C.F.R. dams within the Commission starting with the Wilmot, Cethana and Paloona Dams in the Mersey-Forth Power Development, and through to the Pieman River Power Development Dams onto the dams currently under construction on the King River at Crotty and in the Anthony-Henty Scheme.

He showed how construction methods for this type of dam had been improved during the last twenty years.

In the Mersey-Forth Scheme two types of concrete plinth design had been used, ie the 'Cethana' type and the 'Paloona' type plinth. In the first type the excavation for the plinth apron extends deeper into the foundation rock so that the top surface of the apron is normal to plinth reference line which enables grouting equipment to be positioned squarely onto the apron. In the second type the apron is parallel with the valley wall. This cuts down on foundation excavation but makes the positioning of grouting equipment much more complicated. From experience it was found that the 'Cethana' type plinth is more economical and is now used exclusively for C.F.R. type dams in the Commission.

In the placing and compacting of the rockfill a number of changes had been introduced. For the earlier dams the Commission had used a fleet of large capacity 'Haul-Pacs' which required haul roads at fairly flat grades. Currently the Commission was using a hire fleet of smaller articulated dump trucks which were much more manoeuvrable and could handle steeper grades. To compact the rockfill smaller self propelled vibrating rollers (6 tonne) were now being used instead of 10 tonne towed units. Tests had shown that practically the same degree of compaction could be obtained with the self propelled rollers. The large 10 tonne rollers, which are nowadays no longer readily available, are still being used to compact the upstream and downstream faces of the rockfill, At Crotty dam glacial gravels were being used as fill instead of the customary shot rock. Although the gravels were required to be washed to enable placing in wet weather once in place they performed very well. Settlement measurements at Crotty dam indicated high rockfill modulus values for the gravels which formed the core of the dam. The upstream and downstream zones were being constructed using shot rock from the nearby power tunnel excavation.

In the construction of the upstream concrete face slab numerous construction improvements had been developed over the years. These ranged from improved waterstop installation details, eg the use of stainless steel instead of copper, to the use of much lighter slipform equipment which improved construction efficiency considerably.

The lecture was well illustrated with diagrams and colour slides. At the conclusion of the lecture a fifteen minute video on the construction of the Reece dam on the Pieman River was shown.

Bram Knoop, A.G.S. Tasmanian Group.

SYDNEY GROUP

Meetings during the first half of the year received good attendances, particularly the May 9 meeting on "Coledale Remedial Measures".

The April 11 meeting was changed to:

-PERFORMANCE VERSUS PREDICTION -

TRIAL HIGHWAY EMBANKMENT IN MALAYSIA

Professor Harry Poulos - Coffey Partners International Pty Ltd.

Four geotechnical engineers were invited to predict the behaviour of a test embankment section in the Muar Valley south of Kuala Lumpur in Malaysia. The embankment was located on a deep deposit of alluvial clay and will eventually form part of the north-south highway which crosses Malaysia from the Thailand border to Singapore.

The predictors were asked to predict the following:

i) embankment height at failure.
ii) excess pore pressures in the clay
iii) settlement profile at the soil surface
iv) horizontal movement profiles

The lecture will describe the prediction approach adopted by the author and his two co-predictors at the University of Sydney. The other three predictions will also be summarised. The four predictions will then be compared with the measured behaviour, which was revealed well after the predictions were made.

Total group members also submitted their Class A predictions with about the same scatter in values.

Although the talk to be given by Dr Arthur Penman on May 30 was postponed (to a date yet to be fixed) an additional meeting was held on 29th June 1990 by Dip. Ing. Leonhard Zanier from Austria, entitled "Dynamic Compaction and Dynamic Replacement". The presentation included a film showing the soil improvement works for runway, taxiways and aprons of the new Changi International Airport, Singapore.

Future Meetings

Technical meetings scheduled for the last half of the year are as follows:

July 11, 1990

GEOTECHNICAL RESEARCH AT CSIRO

Dr Bruce Hobbs - Commonwealth Scientific Industrial Research Organisation

The emphasis of this talk is on new instrumentation, techniques and services available to investigate, monitor and alter the in-situ mechanical properties of rock masses. Ten individual examples are presented. These techniques are important not only because of their intrinsic use in geotechnical investigations, but because they form the basis of a new industry in Australia.

The examples are:

* Ground probing radar
* Down hole nuclear techniques
* 20 vision systems for size analysis on conveyor belts and muck piles, and for RQD determination from surface exposures.
* Technology for methane drainage from coal seams.
* Grouting and artificial cementation techniques
* Rock reinforcement
* Rock stress and displacement monitoring, including stress monitoring using tomography
* Blast vibration monitoring with emphasis on transducer mount techniques
* Computer aided mine and tunnel design packages with emphasis on rock stability and support
* Automated and 'smart' mining and tunnelling machinery.

In each example, the aim is to demonstrate that it is now possible to determine and monitor specific geomechanical properties, often remotely and sometime in-situ, over greater volumes of rock than has been readily possible in the past. In addition, the potential is there to alter selectively some of these properties using techniques such as reinforcement or artificial cementation so that construction methods are facilitated.

These techniques, with others, form part of a package of technology that can be the basis for a new manufacturing industry in Australia built around automated and 'smart' mining and tunnelling machinery and services. A strategy for developing this new industry is outlined.

August 8, 1990

WASTE MANAGEMENT AND GEOMECHANICS

Mr William McClenney - Kinhill Engineers Pty Ltd

Environmental management has gradually become more involved over the past decade with geotechnical engineering and this trend can be expected to continue, and perhaps the largest overlap in these fields is in the waste management area. In the past, waste management strategies have often neglected the effects on the environment of fugitive releases of waste products.

With new environmental controls, there is a need for clean-up of continuing and disused operations and for design of improved waste management for new and continuing operations. In the design and implementation of the necessary measures, extensive geotechnical inventories of the soil, rock and groundwater conditions at the sites involved and their interaction with wastes are basic requirements.

The needs of these studies have demanded improved assessments of movement and contaminants through soil and rock by percolation and diffusion of the pore gas and water, and of the interaction of ground materials and wastes. These assessments involved traditional geotechnical disciplines, such as hydrogeology, together with chemical and allied disciplines.

As legislative controls become more strict, engineering controls for environmental management will become more exotic to deal with them effectively. Therein lies the challenge.

September 12, 1990

A GEOTECHNICAL ENGINEER'S CASEBOOK - PANEL DISCUSSION

Chairman: Mr Don Douglas - D J Douglas & Partners Pty Ltd

Panelists:

Mr Colin Thorne - Coffey Partners International Pty Ltd
Mr Jack Hodgson - J D Hodgson Consultants Pty Ltd

Three senior engineers will present brief case histories of several past projects that 'might have been done better', with specific reference to the lessons which can be learned from them.

October 10, 1990

TRIAL LOADING OF A FAILED SECTION OF RIVER BANK, FISHERMAN ISLAND WHARF, PORT OF BRISBANE

Mr Colin Thorne - Coffey Partners International Pty Ltd

In the early hours of 30th November, 1985, 180 metres of the partially completed bank of the new Fisherman Island's No. 3 berth collapsed into the Brisbane River. At the time of the failure the natural bank had been dredged and a supporting wall of rock and sand fill had been placed. In the course of the failure shearing had occurred in the river deposits with displacements of up to about 4 metres. At the toe, the dredging had resulted in over consolidated conditions and laboratory reversing shear box tests had shown low residual friction angles. Analyses for remedial measures showed that the costs of such measures were very dependent on whether residual strength behaviour would apply, on the extent of pore pressures remaining from the failure, and on pore pressures which would be developed during reconstruction. There were also indications that in the un-failed area there could be re-moulded conditions near the toe.

To provide information on these matters, it was decided to carry out two trials to near failure by loading two sections of the bank. Monitoring consisted of numerous piezometers, inclinometers and movement markers to track the deformation and pore pressure build-up as loading progressed. The major findings were that in the failed and disturbed areas the operational strength was essentially the critical state value, and that substantial pore pressures were developed near the toe because even at relatively high factors of safety against the overall failure, the area near the toe became highly stressed and experienced pore pressure increases approximating the maximum total principal stress increase, ie Skempton's 'A' = 1. Near the rear of the slope, the pore pressure increase approximated the increase in total bulk stress, ie Skempton's 'A' = 0.33.

November 14, 1990

CHAIRMAN'S ADDRESS

Dr Tony Phillips

FAILURES AND THE GEOTECHNICAL ENGINEER

The geotechnical engineer bases most of his designs on the results of limited site investigation and uses experience and judgement to interpret geology and test data. In addition his analysis normally relies on the application of simplified theoretical models of ground behaviour, which may or may not be appropriate to conditions on site.

Fortunately, the approach works more often than not, but factors can come into play which are not appreciated until problems arise. These may be a direct result of some feature of the geological history of the site, or of changes brought about by the development itself.

Lessons can be learnt from case histories involving failures. A number of examples will be given to illustrate some of the problems that can arise. The effectiveness of post failure investigations will also be considered to determine what information is needed to undertake effective back analysis.

All meetings are held in the Auditorium of the Institution of Engineers, Eagle House, 118 Alfred Street South, Milsons Point, between 6.00pm and 7.30pm. Light refreshments are available from 5.30pm.

A sub-committee of the Sydney Group has been formed to examine the problem of "Professional Liability". Any comments or suggestions may be forwarded to Colin Thorne, Coffey Partners International Pty Ltd.

WESTERN AUSTRALIAN GROUP

Reports on Recent Meetings.

The airline strike restricted our programme in the latter half of 1989 with a number of meetings having to be cancelled or deferred when speakers' transport arrangements were disrupted. A brief review of meetings which did proceed, is presented below.

5 October 1989 - Dr M Khorshid

'GEOTECHNICAL EXPERIENCE ON THE NORTH WEST SHELF'

The North Rankin 'A' platform was installed in 1982 on the north west shelf of Australia and is located in a water depth of 125 m. The platform was designed to resist cyclonic weather conditions. The proposed Goodwyn 'A' platform will be installed in 1992. It is located approximately 23 km to the south west of North Rankin 'A' in similar water depth and on similar foundation materials.

This presentation discussed the geotechnical programmes undertaken for both North Rankin 'A' and the proposed Goodwyn 'A' platforms. These included:

- the experience with the initial design of North Rankin 'A', including measures which were to be implemented if the design assumptions could not be verified during installation; the realisation that following the implementation of these remedial measures, the capacity was still insufficient; the development of an extensive field and laboratory test programme to enable the design of further primary and secondary remedial measures and the implementation of the selected option which consisted of an end bearing solution.

- the selection of drilled and grouted piles for the proposed Goodwyn 'A' platform foundations; the site investigations both offshore and onshore; the development and results of the large scale pile load testing programme and the modification to the design codes which included effects of long term cycling.

23 November 1989 - Professor CP Wroth

'CONTROL OF GROUND MOVEMENTS AROUND A LARGE EXCAVATION IN LONDON'

One of the many developments taking place at present in London is the construction of a large office block to serve as headquarters in Europe for one of the leading American banks. Planning permission restricted the height of the building, so that in order to maximise the investment in the site, four basement levels are being constructed below existing ground level.

The resulting excavation is essentially rectangular in plan, with dimension 145 m x 45 m x 18.5 m deep. The method of construction is 'top-down' with initially a deep slurry trench diaphragm wall formed around the site; the ground floor slab is cast and excavation proceeds under this slab to the next level. Each floor in turn acts as a stiff horizontal diaphragm to restrain the lateral movements of the retaining wall and minimise possible damage to neighbouring buildings.

The talk included description of the site investigation and soil testing (cone penetration testing, self-boring pressuremeter tests and special triaxial tests), the finite element calculations of the movement of the wall and the performance of the wall as monitored by accurate survey and inclinometers.

The 1990 programme has included the following technical meetings:

13 March 1990 - C Bradbury & C Potulski

'SLURRY TRENCH CUTOFF FOR THE HARRIS DAM'

High labour costs and improved safety standards rule out traditional methods of constructing deep seated cutoffs in soil foundations of modern dams. The talk reviewed the selection of a cement bentonite slurry trench cutoff for the Harris Dam and discussed the problems encountered in mix design and cutoff construction.

15 May 1990 - D Smith (Joint meeting with Structures Panel)

'THE ENGINEERING PROPERTIES OF PERTH SANDS'

Mr. D. Smith presented a paper which included the following topics:

- types of Perth Sands
- their strengths and densities
- settlement and bearing capacity
- effect of groundwater
- Perth Sand Penetrometer tests in sand
- Electric Friction Cone tests in sand.

21 June 1990 - C Windsor

'ROCK MECHANICS AND STRUCTURED SLOPES'

In recent years a number of advances have been made in rock mechanics that have direct application to surface excavations in structured rock. These include investigation and analysis techniques in structural geology, design and analysis techniques for single block stability assessment, continuum and discontinuum computation techniques for analysing progressive collapse mechanisms and new reinforcement hardware. Individually, these advances are substantial but when integrated, yield a powerful methodology for the design and analysis of unreinforced and reinforced surficial excavations.

5 July 1990 - J Doyle

'THE APPLICATION OF CROSS HOLE SEISMICS IN THE CIVIL AND MINING INDUSTRIES'

Mr John Doyle of BHP Engineering presented a paper to the W.A. Group of the Australian Geomechanics Society on The Application of Cross Hole Seismics in the Civil and Mining Industries. The main part of presentation concentrated on the use of cross-hole seismic techniques to obtain a complete two-dimensional picture of the ground conditions between a pair of boreholes. The speaker used the analogy of brain scanning using X-ray tomography to explain the process. In seismic tomography, a string of geophones is lowered into one borehole, and a string of charges lowered into another. Ideally, the string lengths are the same order as the borehole spacing. The charges are fired in sequence (usually starting from the bottom of the borehole), and each geophone records the arrival time of the compression wave from each explosion. Thus, if there are n geophones and n charges, an array of n x n arrival time values are recorded in the whole test.

By dividing the area between the boreholes into a rectangular grid, the average compression wave velocity Vp of each cell can be uniquely determined, with the only limiting assumption being that the fastest travel path between source and geophone is a straight line. Having obtained values of Vp for each cell, contouring of Vp is carried out automatically by the computer software, and the results displayed in colour on the screen. This is the so-called tomograph of the area between the boreholes. The final stage is to interpret this contour plot to relate the contours to the geology of the area.

During subsequent discussion, questions from the audience related mainly to the effect of the assumption of straight-line travel (especially where the object was to search for cavities), to the cost of the process, and to the scale of operations in which it was most ideally used.

9 July 1990 - Professor B Whittaker

ROCK SLOPE STABILITY AND MINING SUBSIDENCE

Professor Barry Whittaker from the University of Nottingham addressed a well attended lunchtime meeting on subsidence and slope stability. He presented many examples and a facinating array of slides to illustrate the dramatic affects of subsidence and slope failure. The topic obviously attracted a wider audience than usual with many new faces.

Future Meetings

30 August 1990 - C Orr (Luncheon Meeting with Mining Club)

SUBJECT: OPEN PIT ABANDONMENT

The recent increase in the open pit mining within the State will result in between 60 and 100 open pits being abandoned over the next few years. The talk will deal with the existing guidelines on safety bund walls around abandoned open pits, types of pit failures observed in a sample of approximately 70 pits and analytical techniques used to predict the long term stability of pit slopes.

4 September 1990 - Dr PJ Hensley

SUBJECT: CENTRIFUGE MODELLING OF CONTAMINANT TRANSPORT IN SOILS

Centrifuge modelling is an ideal tool for studying problems associated with transport of pollutants through soils. This talk will present work from a number of model tests completed using the geotechnical centrifuge at the University of Western Australia to investigate a variety of subsurface contaminant transport problems. The principles of centrifuge modelling processes governing transport phenomena and scaling laws will be briefly discussed. Tests will be described and results presented including comparisons with theoretical data from existing numerical transport codes. Conclusions and an outline of future work will be presented.

NEWSLETTER

The group has continued its practice of circulating a non technical newsletter with 2 editions so far this year. Contributions should be forwarded to: G. Cocks c/- Coffey Partners International, P O Box 517, Subiaco WA.

'OSBORNE GEOTECHNICAL'

in the West

We announce the formation of a new consulting practice based in Perth. The principal is Trevor Osborne and the firm trades under the banner of Osborne Geotechnical. Trevor has decided to change roles after over 20 years in the specialist contractor field with Grouting and Foundations and the Cementation Co (Aust) Ltd.

The practice is specialising in construction aspects of Geotechnical Engineering with particular emphasis on foundations, grouting, soil retention and soil stabilisation.

Trevor Osborne is also developing particular expertise in the field of soil reinforcement and the firm has established links with Dr Richard Jewell at Oxford University and Prof. Colin Jones at Newcastle upon Tyne. Richard and Colin are regarded as two of UK's leading practitioners in the field of reinforced soil design and construction.This field includes the use of reinforcing techniques for soil structures and embankment construction and the developing area of soilnailing.

The client base for the firm already includes representation from other specialist consulting firms, structural consultants, contractors, mining companies and government departments.

With the practice working within a very specialised area it is well placed to provide practical advice to compliment the services of other geotechnical consultants.

GEODIARY

CONFERENCES, COURSES, SEMINARS, SYMPOSIA, WORKSHOPS, ETC

Brief details of conferences,courses, seminars, symposia, workshops, etc will be entered in Geodiary without charge as a service to members of the Society. Advertisements giving more prominence and carrying greater detail may be inserted in any issue of Australian Geomechanics.

AUG 6-10, 1990

Amsterdam, The Netherlands.

6TH INTERNATIONAL CONFERENCE OF THE IAEG.

Topics: Engineering geological mapping; Remote sensing and geophysical techniques; Hydro-engineering geology; Surface engineering geology; Underground engineering geology; Engineering geology of land and marine hydraulic structures; construction materials. **Languages:** English and French.

Sixth International IAEG Conference, QLT/ CONGREX,Keizersgracht 782,1017EC Amsterdam, The Netherlands.

AUG 1990

Bucharest, Romania.

INTERNATIONAL COLLOQUIUM ON VISCOPLASTICITY OF GEOMATERIALS.

Topics: Recent Developments in theory, experiment, computational methods and applications.

Chairman Organising Committee of Colloquium on Viscoplasticity of Geomaterials, Department of Mechanics, University of Bucharest, str Academiei 14, Bucharest Cod 70109, Romania.

AUG 28-31, 1990

Beijing, China.

INTERNATIONAL SYMPOSIUM ON ADVANCES IN GEOLOGICAL ENGINEERING.

Themes: Systematic evaluation of the engineering geological conditions for site selections; Processing of the geological information and establishment of the geomechanical models; Evaluation of the geomechanical parameters; Reliability analysis of the geological engineering and the optimum design; The monitoring system of the geological engineering and the back-analysis; Case histories with emphasis on integration of geology into engineering.

Dr Yang Zhifa, International Symposium on Geological Engineering Problems, Institute of Geology, Academia Sinica, Beijing, P.O. Box 634, China. Tel: 4014031 Fax:(86)(1) 4014031.

SEP 3-5, 1990

Cranfield, UK.

INTERNATIONAL CONFERENCE ON MICROBIOLOGY IN CIVIL ENGINEERING.

Topics: include rock and groundwater environments and geotechnical engineering.

Mrs C. King/Dr P. Howsam, Silsoe College, Silsoe, Bedford, MK45 4DT. Tel:(0525) 60428. Fax:(0525) 61527.

SEP 3-7, 1990

Glasgow, UK.

3RD INT. SYMPOSIUM ON RECLAMATION TREATMENT AND UTILISATION OF COAL MINING WASTES.

Dr A.K.M. Rainbow, Head of Minestone Services, British Coal Corporation, Philadelphia, Houghton-le-Spring, Tyne and Wear, DH4 4TG, UK. Tel:091 584 3631

SEP 3-7, 1990

Chengdu, Sichuan, China.

INTERNATIONAL CONGRESS ON TUNNEL AND UNDERGROUND WORKS - TODAY AND FUTURE.

Topics: Design and construction of railway tunnels, urban tunnels and metros, hydro tunnels and rock caverns; Theoretical analysis and design of underground structures. **Languages:** English and Chinese.

Prof. Xu Wenhuan, Southwest Jiaotong University, Jiu Li Di 610031, Chengdu, Sichuan, China. Tlx: 600072 SWJU CN.

SEP 9-13, 1990

Leeds, UK

26TH ANNUAL CONFERENCE OF THE ENGINEERING GROUP OF THE GEOLOGICAL SOCIETY.

Theme: The Engineering Geology of Weak Rock.

Topics: Weak rock materials; Weak rock masses; Engineering in weak rocks.

Dr S.R. Hencher, Department of Earth Sciences, University of Leeds, LS2 9JT, UK.

SEP 10-12, 1990

Glasgow, UK.

INTERNATIONAL REINFORCED SOIL CONFERENCE.

Sessions: Theory and Design Related to the Performance of Reinforced Soil Structures; Construction Influences on the Performance; In-situ Techniques; Modelling and Laboratory Testing of Soil and Material Properties; Design, Construction and Performance of Reinforced Soil Structures over Soft and Difficult Foundations.

Conference Secretariat, Meeting Makers, 50 Richmond St., Glasgow, G1 IXP, UK. Tel:(041) 553 1930. Fax:(041)552 0511.

SEP 10-12, 1990

Royal Swazispa, Swaziland.

ISRM INTERNATIONAL SYMPOSIUM ON STATIC AND DYNAMIC CONSIDERATIONS IN ROCK ENGINEERING.

Topics: Excavation in rock, mechanical and explosive methods; Dynamic behavior of excavations in rock; Long term performance of excavations in rock susceptible to deterioration.

Mr M. Faure, ISRM Symposium, PO Box 5906 Johannesburg 2000, Republic of South Africa. Tel:(011) 399 4738. Fax:(011) 339 1281. Tlx: 9-450083 SA.

SEP 11-13, 1990

Sydney.

SEVENTH AUSTRALIAN TUNNELLING CONFERENCE AND TRADE EXHIBITION -THE UNDERGROUND DOMAIN.

Objectives: The aim of the conference is to provide a forum for discussion of the broad economic, social and technical factors that bear upon our community's use of underground space. This will be achieved by bringing together interested entrepreneurial, urban planning, technical, construction, regulatory and environmental specialists.

The Conference Manager, The Institution of Engineers, Australia, 11 Barton Circuit, BARTON,ACT 2600, Australia. Tel:(06) 270 6563 Fax:(06) 270 6530. Tlx:AA62758.

SEP 12-15, 1990

Nottingham University, UK.

BNCOLD'S 6TH CONFERENCE ON EMBANKMENT DAM.

Topics: State of the art of embankment dam; Tailings dams design and operation; Risk, hazard and safety; Research; Dams and the environment.

N.Tyler, The Institution of Civil Engineers, 1-7 Great George St, London SW1P 3AA, UK.

SEP 17-20, 1990

Toronto, Ontario

DAM SAFETY CONFERENCE OF THE CANADIAN DAM SAFETY ASSOCIATION.

Topics: Inspection and monitoring; Legislation and emergency planning; Dam safety assessment and Innovative case studies. A tour of various dams on the Niagara Peninsula is also planned.

Mr B. Hurndall, P.O. Box 4490, South Edmonton postal Station, Edmonton, Alberta, Canada T6E 4X7. Tel:(403)422-1359 or Grant Smith, Ontario Hydro, Tel:(416)590-2498.

SEP 18-20, 1990

Santander, Spain.

2ND EUROPEAN SPECIALTY CONFERENCE ON NUMERICAL METHODS IN GEOTECHNICAL ENGINEERING.

Theme: Establishment of links between the development and practical application of numerical methods in Geotechnical Engineering and the enhancement of cooperation with other existing groups within the field of numerical methods in Geotechnical Engineering.

D.C.I.T.T.Y.M., Escuela de Ingenioros de Camerinos, Universidad de Cantabria, Avda. de Los Castros, s/n, 39005 Santander, Spain. Tel: 42-201822 Fax:42-275862.

OCT 1-5, 1990

Kuala Lumpur, Malaysia.

4TH INTERNATIONAL CONFERENCE ON SMALL HYDRO 1990.

Topics: Assessing the needs of developing countries; International cooperation; Implementing technology transfer and training; Environment impact studies; site selection; using indigenous technology.

Mr L. Maskell, Water Power & Dam Construction, Quadrant House, The Quadrant, Sutton, Surrey SM2 5AS, UK. Tel:44 1 661 3622. Fax 441 661 8904. Tlx: 892084 REEDBP G.

OCT 2-5, 1990

Budapest, Hungary.

9TH DANUBE-EUROPEAN CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING.

Theme: Deep Geotechnical Works in Urban Areas.

Prof. G. Petrasovits, Geotechnical Department, Technical University, H-15212 Budapest, Muegyetem rkp.3. Tel:(36-1-)1666-42-42, Fax:(31-1-) 166 68-08. Tlx:225931 MUEGY.

OCT 10-12, 1990

Sainte-Foy, Quebec.

CONFERENCE, PREDICTION AND PERFORMANCE IN GEOTECHNIQUE.

Topics: Economic significance of landsliding in Canada; Static liquefaction in sand; Characteristics and properties of aggregates; Geotechnical aspects of earthquakes; soil and treatment.

Dr Jacques Locat, ing., President du C.O., Dept. de geologie (GREGI) Pavillon Pouliot, Universite Laval, Sainte-Foy, Quebec G1K 7P4. Tel: (418)656-2179.Fax:(418)656-2603.

OCT 11-13, 1990

Cracow, Poland.

9TH NATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING.

Topic: Theoretical and experimental bases of geotechnics; Practical problems in geotechnics; Geotechnical aspects of environmental control.

Kominet Organizacyjny IX Krajowej Konferencji, Mechanikow Gruntow i Fundamentowania, Biuro R W NOT, Dzial Org. Prezydialny, ul Straszewskiego 28, 31-113 Krakow.

OCT 11-13, 1990

Indianapolis, Indiana.

SPOTLIGHT '90. INDUSTRY, TRADE SHOW AND SYMPOSIUM SPONSORED BY THE INTERNATIONAL DRILLING FEDERATION.

International Drilling Federation, 3008 Millwood Av, Columbia, SC, USA 29205. Tel:(800)445-8629.

OCT 16-18, 1990

Lille, France.

INTERNATIONAL CONFERENCE ON UNDERGROUND CROSSINGS FOR EUROPE.

Topics: Geography; Design and work techniques; Economic and financial aspects; Operation of structures. **Languages:** English and French.

Journees d'Etudes A.F.T.E.S - Semaly, Mrs Jacqueline Boller, 25 cours Emile-Zola, 69625 VILLEURBANNE CEDEX.

Tel:78 94 86 00, Fax:78 94 86 00,

Tlx: 380801F.

OCT 17-20, 1990

Caracas, Venezuela.

3RD SOUTH AMERICAN CONGRESS ON ROCK MECHANICS.

Topics: Characterisation and properties of the rock mass; Slope stability for soft and for fractured rocks; Foundations on soft and fractures rocks; Tunnels in soft and fractured rocks; Open pit and underground mining; Rock mechanics applications to oil industry; Special topics.

Juan Francisco Lupini, Ap. Post 4074, Caracas 1010, Venezuela. Tel:(582) 7513824 / (582) 6625902. Fax:(582) 5628048. Tlx:29136 CEHVC.

OCT 30-31, 1990

Singapore.

CONFERENCE ON DEEP FOUNDATION PRACTICE.

Topics: Support and stability of deep foundations; Load bearing and settlement solutions; Strengthening of foundations of existing structures and industrial buildings; Soil-structure interaction; caisson design and construction; Piling economics, systems,design etc.; Piling installation, testing, interpretation; Stress wave measurements, load capacities; New development in micropiles; Deep raft foundations - construction method; Bored piles, tubular piles and applications; Case histories and practical examples. Keynote addresses by Prof.Bengt Broms and Mr William Loftus. **Abstracts:** 1 Jul 1990. **Papers:** 5 Sep.

CI Premier Pte Ltd, 150 Orchard Rd,#7-14 Orchard Plaza, Singapore 0923. Tel: 733 2922. Fax: 2353530.

NOV 4-7, 1990

Magnetic Island, North Queensland.

ENGINEERING IN CORAL REEF REGIONS.

A conference for Engineers and Scientists involved in the engineering activities in the Great Barrier Reef and similar regions.

Purpose: To provide engineers with an up-to-date scientific overview of the reef environment; To allow the exchange and publication of engineering experience in the Great Barrier Reef and similar regions; To identify research priorities for engineering activities in or associated with the Great Barrier Reef region.

Dr Michael Gourlay, Department of Civil Engineering, The University of Queensland, St Lucia, Qld. 4067.

Tel:(07)377 2543. Fax:(07)371 5863.

Tlx: AA40315 UNIVQLD.

NOV 7-9, 1990

Calgary, Alberta, Canada.

SECOND INTERNATIONAL CONFERENCE ON MINE PLANNING AND EQUIPMENT SELECTION.

Theme: Geotechnical Stability and Mine Equipment Selection for Surface Mines.

Topics: Pre-mining investigations; Slope stability predictions and monitoring techniques; In-situ instrumentation; Ground water; Slope stability analysis; Pit slope stabilisation techniques; Blasting and controlled blasting; Haulroad design and construction; Design and construction of waste dumps; Liquid storage and tailings ponds; Application of geotextiles; Remote sensing; Reclamation; Equipment selection procedures; Continuous mining systems; Change over to continuous mining systems; truck/shovel, draglines, conveyors,etc; In-pit crushing/conveying; Pipeline transporting; Drilling and blasting equipment; Dredging; Mine maintenance; Equipment health and performance; Monitoring systems; Case histories.

Dr Raj K. Singhal, Symposium Chairman, Faculty of Continuing Education, The University of Calgary, 2500 University Drive, N.W., Calgary, Alberta T2N 1N4. Tel:(403)220-6069, Fax:(403)282-7208.

NOV 19-21, 1990

Tehran, Iran.

FIRST INTERNATIONAL SEMINAR ON SOIL MECHANICS AND FOUNDATION ENGINEERING OF IRAN.

Themes: Exploration and Evaluation of Soil Properties; Design and Analysis in Geotechnical Engineering; Problems of Regional Soils; Underground Structures; Construction Control and Monitoring.

Organising Committee, Plan and Budget Organisation (Technical Research and Standards Bureau), Post Code: 15316, Dr Beheshti Ave., Pakistan St., 2nd Alley, No 7, 4th Floor, Tehran, Iran.

NOV 26-30, 1990

Bangkok, Thailand.

GEOTECH' 1990. 14TH ANNUAL GEOTECHNICAL SYMPOSIUM.

Topics: Model Tests in Relation to Traditional Laboratory and Field Tests.

Asian Institute of Technology, GPO Box 2754, Bangkok 10501, Thailand. Tel:(43)529004. Fax:(66-2)5290374.

FEB 25-MAR 1, 1991

Atlanta, Georgia, USA.

GEOSYNTHETICS '91. - THE BIENNIAL NAGS-IFAI CONFERENCE.

Theme: The conference will focus on current applications of geosynthetics and technical advancements.

Topics: Transportation; Waste management & other environmental applications; Commercial & industrial applications; Building and foundations; Heavy construction & earthworks projects; Geosynthetic durability; Future applications; Geosynthetic innovations; Technical advancements - research, testing, new design methods. There will also be pre and post conference courses and a special forum on failures and solutions to discuss lessons that can be learned from examples of misapplication.

NAGS Secretary, Ms Laurie Honningford, Industrial Fabrics Association International, 345 Cedar St, Suite 800, St Paul, MN, USA, 55101. Tel:(800)225-4324 or (612)222-2508. Fax:(616)222-8215.

MAR 11-14,1991

Perth, Australia

ANCOLD 1990 CONFERENCE ON DAMS

Topics: RCC and Earthfill dams; Tailings dams;Repairs and Rehabilitation; Catchment management.

C.Bradbury, 1990 ANCOLD Conference, Water Authority of Western Australia, PO Box 100, Leederville, WA 6001.

MAR 11-15, 1991

St Louis, Missouri, USA

2ND INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS.

Shamsher Prakash, Conference Chairman, Dept of Civil Engineering, University of Missouri-Rolla, Rolla, MO 65401, USA. Tel:(314)341-4489 or 341-4461. Fax:(314)341-4719.

MAR 18-20, 1991

Milano, Italy.

EUROPEAN CONFERENCE ON "SOIL AND ROCK IMPROVEMENT IN UNDERGROUND WORKS".

Topics: Ground reinforcement; Low pressure grouting; Very high pressure grouting; Drainage; Freezing; Preventative schemes.

Organising Secretariat, MGR S.r.l., Via Servio tullio 4, 20123 Milano, Italy. Tel:02/43007.1. Fax:02/4980723.

MAR 19-21, 1991

Paris, France.

INTERNATIONAL CONFERENCE ON DEEP FOUNDATIONS. **Topics:** New technologies; Behaviour and design; Control and load tests; Repetitive loads and dynamic stresses.

F.Schlosser, Tour Horizon, 52 quai de Dion Boulton, 92806 Puteaux Cedex, France. Tel:(1) 47.76.43.14. Fax:(1) 47.73.85.64. Tlx:610386F.

APR 7-12, 1991

Stresa, Italy.

4TH INTERNATIONAL CONFERENCE ON PILING AND DEEP FOUNDATIONS.

Topics: Technological developments in deep foundations with soil excavation; Pile installation equipment; Pile design; Pile testing.

J. M. Mitchell, Ove Arup and Partners, 13 Fitzroy St. London W10 6BQ, UK. Tel:01 636 1531. Fax:01 580 3924. Tlx: 295341 OVARPT G.

APR 15-19, 1991

Canary Islands, Spain.

23RD INTERNATIONAL CONGRESS OF THE INTERNATIONAL ASSOCIATION OF HYDROGEOLOGISTS. AQUIFER OVER-EXPLOITATION.

Topics: Characterisation of over-exploitation. Hydrogeological and hydrochemical aspects; Environmental effects related to over-exploitation; Protective and corrective measures in cases of over-exploitation; Legal and socio economic problems related to aquifer over-exploitation; Over-exploited aquifers in water resources management.

Dr Fermin Villarroya, Dept. de Geodinamica, Facultad de Ciencias Geologicas, Universidad Complutense, 28040 Madrid, Spain. Tel:(34-1)449-73-91. Fax:(34-1)243-91-62.

APR 15-19, 1991

Isle of Wight, UK

INTERNATIONAL CONFERENCE ON SLOPE STABILITY ENGINEERING.

Topics: Mechanics of landslides; Environmental influences on slope stability; Planning aspects; Investigation techniques; Coastal slopes (including Isle of Wight studies); Remedial works.

Conference office, Institution of Civil Engineers, 1-7 Great George St, London SW1P 3AA, UK.

MAY 6-10, 1991

Cairns, Queensland.

INTERNATIONAL CONFERENCE ON COMPUTER METHODS AND ADVANCES IN GEOMECHANICS.

Topics: Constitutive modelling of geomaterials; Methods for solving problems in rock, soil and ice mechanics; Analytical methods in geomechanics; Hardware and software; Verification by field and laboratory measurement.

IACMAG 1991, GPO Box 853, Brisbane, Qld., 4001. Australia. Tel:(07) 21 2762. Fax:(07) 220 0231. Scientific enquiries to: Conference Chairman, Dr G. Beer, CSIRO., PO Box 63, St Lucia, Qld.,4067. Tel:(07)377 7822. Fax:(07)371 7435.

MAY 12-18, 1991

Houston, Texas, USA.

FOURTH INTERNATIONAL SYMPOSIUM ON LAND SUBSIDENCE. **Objects:** Provide a forum for presenting results of research and practice in the subject; Exchange experiences related to cause, effect, control and remediation of land subsidence; Promote technology transfer between the various disciplines and countries represented; Evaluate the advance of knowledge taking place on this subject since 1983 and develop guidelines for needed future research.

Ivan Johnston, Chairman, FISOLS, A.Ivan Johnston Inc, 7474 Upham Court, Arvada, CO 80003, USA.

MAY 27-JUN 1, 1991

Florence,Italy.

10TH ISSMFE EUROPEAN REGIONAL CONFERENCE.

Topics: Experimental determination of soil properties; Modelling stress-strain behaviour of natural soils; Displacements and soil-structure interaction, Specialty sessions on a range of associated topics.

The Organising Committee,X ECSMFE, C/ - Associazione Geotecnica Italiana, Via Bormida, 2, 00198 Rome, Italy. Tel: 39 6 856129.

Fax: 39 6 8842265.

JUN 3-7, 1991

Mexico City.

6TH INTERNATIONAL CONFERENCE ON APPLICATIONS OF STATISTICS AND PROBABILITY IN CIVIL ENGINEERING.

Instituto de Ingenieria, Unam, Ciudad Universitaria, Apartado Postal 70-472, Coyoacan 04510, Mexico, d.f.

JUN 10-12, 1991

Boulder, Colorado, USA.

ASCE GEOTECHNICAL ENGINEERING CONGRESS.

Topics: Broad scope with emphasis on examples from current practice and recent innovations for future practice.

Prof. Stein Sture, Department of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, Colorado 80309-0428 USA.

Tel:(303)492-7651.

Fax:(303)492-7317.

JUN 13-14, 1991

Boulder, Colorado, USA.

INTERNATIONAL CONFERENCE - CENTRIFUGE 91.

Topics: Design and operation of new facilities; Special instrumentation problems and design of test packages; Studies of basic mechanics and gravity dependent phenomena; Application of centrifuge modelling to natural and manmade hazards reduction; Centrifuge studies related to engineering design and practice; Relations between centrifuge testing, full scale testing and numerical methods.

Prof. Hon-Yim Ko, Centrifuge 1991, Department of Civil, Environmental and Architectural Engineering,University of Colorado, Boulder, Colorado 80309-0428 USA. Tel:(303)492-6716 Fax:(303)492-7317

AUG 26-30, 1991

Vina del Mar, Chile.

IX PAN-AMERICAN CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING.

Topics: Mass movement phenomena; geotechnical properties of soils in America; Geotechnical aspects of waste disposal; Foundations; Underground excavations in urban areas. **Languages:** English and Spanish.

Mr Luis Valenzuela, Secretary General - IX PCSMFE - Vina 1991, San Martin 352, Santiago, Chile. Tel:(56-2)2516527-2257861-498330-2616149. Fax:(56-2)4127. Tlx:240501 BOOTH CL.

SEP 9-11, 1991

Oslo, Norway.

3RD INTERNATIONAL SYMPOSIUM ON FIELD MEASUREMENTS IN GEOMECHANICS.

Topics: Instrumentation and measurement techniques; Data acquisition, processing and interpretation; Use of field measurements as an aid to problem solving. **Abstracts:** Mar 15, 1990. **Language:** English.

FMGM-91, Norwegian Geomechanical Institute, PO Box 40, Taasen, N-0801 Oslo 8, Norway. Tel:+47 2 23 03 88. Fax:+47 2 23 04 48. Tlx:19787 NGIN

SEP 10-12, 1991

Beijing, China.

SIXTH INTERNATIONAL SYMPOSIUM ON GROUND FREEZING.

Topics: Heat and mass transfer; Mechanical properties; Engineering design; Case histories.

ISGF 91, Central Coal Mining Research Institute, Hepingli, Beijing 100013, PR China.

SEP 16-20, 1991

Aachen, FRG.

7TH INTERNATIONAL CONGRESS ON ROCK MECHANICS.

Topics: Rock mechanics and environmental protection; Rock mechanics based on reliable descriptions of geological conditions; Stability of rock slopes; Underground construction in rock. **Abstracts:** July 15, 1990. **Papers:** March 1, 1991. **Languages:**German, English and French.

Dr Ing. W. Wittke, ISRM-Kongressekretariat Instut fur Grunddau, Bodenmechanik, Felsmechanik und Verkehrswasserbau der RWTH Aachen, Mies-van-der-Rohe-Strasse 1, D-5100 Aachen, West Germany. Tel:02 41 80 52 47.

Fax:02 01 78 27 43.Tlx: 832704 THAC

SEP 18-21,1991

Cologne, FRG.

GEOTECHNICA - INTERNATIONAL TRADE FAIR AND CONGRESS FOR GEOSCIENCES AND TECHNOLOGY.

Theme: Interdisciplinary event for all geosciences concentrating on the fields of earth, water, air and energy.

Koln Messe, Messe-und Ausstellungs-Ges.m.b.H. Koln, Messelplatz 1, Postfach 21 07 60, D-5000, KOLN 21 FRG. Tel:02 21 821 0. Fax:02 21 821 25 74. Tlx: 8873426 MUA D.

SEP 23-27, 1991

Maseru, Lesotho.

10TH ISSMFE AFRICAN REGIONAL CONFERENCE.

Theme: Geotechnics in the African Environment. **Topics:** Geological influences; Problem soils; Testing; Foundations; Roads; Dams; Waste disposal; Environmental aspects. **Languages:** English and French.

Mr M. Lephoma, Hon Secretary 10 ARC SMFE, PO Box 7737, Maseru 100, Lesotho.

SEP 29- OCT 2, 1991

Calgary, Alberta, Canada.

44TH CANADIAN GEOTECHNICAL CONFERENCE.

Topics: Soil mechanics and foundation engineering; Rock mechanics; Engineering geology; Cold region geotechnics; Oil sands engineering; Irrigation and water resources; Environmental engineering; Case histories.

Ken Been, Golder Associates, 7017 Farrell Road SE, Calgary, AB T2H 0T3, Canada. Tel:(403)259-3413. Fax:(403)252-4884.

DEC 9-13, 1991

Bangkok, Thailand.

9TH ISSMFE ASIAN REGIONAL CONFERENCE.

Topics: Development of the theory and practice in Geotechnical Engineering (including soil sampling, laboratory and field testing, design methods and construction works); Problematic soils and their engineering behaviour (including collapsible, expansive, dispersive and erosive soils as well as other problematic soils); Soil-structure interaction and foundations (including settlement and compressibility characteristics); Embankments, excavations and buried structures; Natural hazards and environmental engineering; Ground improvement techniques. **Language:** English. Abstracts by Oct 1990, papers by Feb 1991

Prof. A.S. Balasubramaniam, Secretary South East Asia Geotechnical Society, Asian Institute of Technology, PO Box 2754, Bangkok 10501, Thailand. Fax:66-2 529 0374. Tlx:84276TH.

FEB 3-7, 1992

Christchurch, New Zealand.

6TH ANZ CONFERENCE ON GEOMECHANICS.

Dr D.McG.Elder, Convenor Organising Committee, C/- Soils and Foundations Ltd, PO Box 451, Christchurch, New Zealand. Tel:(3)798 432. Fax:(3)667 780.

FEB 10-14, 1992

Christchurch, New Zealand.

6TH INT.SYMPOSIUM ON LANDSLIDES.

Dr D.Bell, Geology Department, University of Canterbury, Christchurch, New Zealand.

FEB 25-28, 1992

New Orleans, USA.

SPECIALTY CONFERENCE ON GROUTING, SOIL IMPROVEMENT AND GEOSYNTHETICS.

Topics: Soil Reinforcement, densification and stabilisation; Application of geosynthetics; Grouting and grout materials; Environmental technology. **Abstracts:**Sep 1, 1990.

Dr Ilan Juran, Department of Civil Engineering, Louisiana State University, Baton Rouge, LA 70803, USA. Tel:(504)388 8699. Fax:(504)388 5990.

MAY 10-16, 1992

Rostock, GDR.

3RD BALTIC CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING.

Prof. Dr Rattay, Kammer der Technik, Anschrift, Clara-Zetkin-Str. 115/117, PSF 1315, Berlin DDR. Tel:2 20 25 31. Tlx:011 4841 TECH KAMMER.

MAY 28-31, 1992

Aalborg, Denmark.

XI Nordic Geotechnical Meeting.

Prof. Jorgen Steenfelt, Danish Geotechnical Society, Maglebjergvej 1, DK-2800 Lyngby, Denmark.

AUG 23-26, 1993

Kingston, Canada.

INTERNATIONAL CONGRESS ON MINE DESIGN.

Peter Scott, Department of Mining and Engineering, Goodwin Hall. Tel:(613)545-2212.

Fax:(613)545-6597.

JAN 5-10, 1994

New Delhi, India.

XIII INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING.

Prof.S.K. Gulhati, Organising Secretary General 13th ICSMFE, Indian Geotechnical Society, C/o Indian Institute of Technology, New Delhi, 110016, India.

Introduction

Although Australia has a population of only 17 million people, it is a continent which is as large as Europe and the USA. The majority of the population live in large coastal cities around the east and south east edges of the continent. The major tunnelling works associated with water supply, sewerage and hydro-electric power are therefore of a scope greater than might be expected for a country with so small a population as a direct result of the concentration of habitation. Australia is also major mining country, specialising in hard-rock metalliferous and coal mining.

Over the last 20 years there has been a revolution in underground construction through the increasing use of mechanical excavation methods and developments in Australia have often played a leading role. This has affected investigation, design and ground support procedures and has demonstrated the ability to dramatically increase productivity. Nevertheless, efficient conventional methods such as drill and blast methods are still applicable and widely used.

Australian Underground Construction and Tunnelling Association (AUCTA)

The Australian Underground Construction and Tunnelling Association was formed in 1972 and is jointly sponsored by The Institution of Engineers, Australia and the Australasian Institute of Mining and Metallurgy. It is a non profit organisation uniting engineers, tunnellers and miners in its membership with the aims of:

* encouraging the use of subsurface space to meet social, environmental and economic demands for facilities.
* promoting advances in research, investigation, design, construction and maintenance of tunnels and subsurface openings and structures.
* acting as a focal point for the collection and dissemination of information on all aspects of tunnelling and underground construction.

The Association membership includes engineering and mining fraternities and extends to contractors, consultants, owners, public authorities, town planners, equipment and material suppliers and all individuals interested in tunnelling, underground construction and the utilisation of undergrouind space.

International Tunnelling Association (ITA)

AUCTA is one of the 35 member nations of the International Tunneling Association, which has its secretariat in France. ITA has a number of working groups: Maintenance and Repair of Underground Structures, Contractural Sharing of Risk, Subsurface Planning, Catalogue of Tunnels, Health and Safety in Work, Research, Immersed Tube Tunnels, Shotcrete Use and General Approaches to the Design of Tunnels. AUCTA has nominated corresponding representatives on all these groups.

ITA has an Annual General Assembly which is organised in one of the member nations countries. It also publishes the journal Tunnelling and Underground Space Technology (TUST); AUCTA membership qualifies for a heavily discounted subscription.

AUCTA Activities

The Association encourages member participation in activities which include:

* organised talks on topics of interest to the members, usually in the form of lunchtime meetings
* tunnelling and underground excavation conferences, which are held triennially
* seminars on particular aspects of tunnelling and underground construction
* site visits to underground works
* presentation of the "Tunnelling Achievement Award" usually in conjunction with the triennial conference to and individual for outstanding achievements in the tunnelling industry.
* assisting with seminars and conference sessions organised by other mining and engineering groups.

AUCTA also provides members with:

* a newsletter/journal "Australia Underground Construction and Tunnelling"
* copies of the United States National Committee on Tunnelling Technology newsletter (quarterly)
* contents abstracts (in English) from the Japan Tunneling Association newsletter "Tunnels and Underground"
* a price reduction on conferences and AUCTA publications
* reduced subscription rates to the International Society for trenchless Technology (ISTT)
* a regular list of references for recently published articles on tunnelling
* information on international underground construction
* a direct link to those interested in underground construction both nationally and internationally

Membership

Four types of membership are available, Full (company or organisation), Affiliate (individual), Student (undertaking full time study at a tertiary institution) and Retired (wholly retired from a professional occupation)

*	Full Member	-	$130 perannum	Student Member	-	$10 per annum
*	Affiliate Member	-	$25 per annum	Retired Member	-	$10 per annum

Full members receive a free copy of the AUCTA publication "Australasian Tunnelling Construction Index"

Any interested party can join and it is not necessary to be a member of either The Institution of Engineers, Australia or the Australasian Institute of Mining and Metallurgy>

Committee

The Association is administered by a Committee which is elected and supported by members. Elections are held every odd numbered year and Committee Members usually hold office for a period of two years. The Chairman of the Association is elected in even numbered years by the Committee from its members and hold office for a period of two years. He is also Chairman of the Committee.

Any member of the Association is eligible for nomination for the Committee.

Publications Available

From:
The Australasian Institute of Mining and Metallurgy
P.O. Box 122
Parkville, Victoria 3052
Tel: (03) 347 3166 Fax: (03) 347 8525

Second Australian Tunnelling Conference
Melbourne, 1976, Design and Construction of Tunnels and Shafts. 234 pages

Fourth Australian Tunnelling Conference,
Melbourne, 1981. 389 pages
$10.00 each or $15.00 for both

Sixth Australian Tunnelling Conference
Bore or Blast? 2 Vols, 358, 225 pages
Melbourne 1987, $80.00

From:
E.A. Books, Engineers Australia
P.O. Box 588
Crows Nest, New South Wales 1065
Tel: (02) 438 1533 Fax: (02) 438 5934

Australasian Tunnelling Construction Index
A summary of tunnels constructed in Australia
$20.00

Fifth Australian Tunnelling Conference
State of the Art in Underground Development and Construction, Sydney 1984, 212 pages - $36.00

Seventh Australian Tunnelling Conference
The Underground Domain, 316 pages
Sydney 1990 - $39

Regulations

Status

The Australian Underground Construction and Tunnelling Association is a non-profit making organisation with activities directed towards the learned society role. It is administered by the Australian Underground Construction and Tunnelling Committee.

The Association is affiliated with the Australian Geomechanics Society, which is a joint venture of The Institution of Engineers, Australia and The Australasian Institute of Mining and Metallurgy.

Objects

The objects of the Association are to:

- advance knowledge of all the factors affecting the use of tunnels
- stimulate a better understanding of these factors by governments, planners and the general public, as well as by those directly concerned with the science and practice of tunnelling
- act as a focus for the collection and dissemination of information on all aspects of tunnelling, including social, economic, aesthetic, medical, industrial and legal, as well as technical aspects
- promote and review research and development and generally to advance knowledge of the socioeconomic, including environmental aspects, of tunnelling
- encourage the writing, presentation and publication of papers on all aspects of the field
- encourage the preparation of specialist technical manuals, data books, codes of practice, forms of contract, and other publications affecting tunnelling
- arrange conferences, symposia and meetings at suitable intervals and to collaborate with other agencies in arranging such functions
- correspond and act as a link with appropriate international bodies, and to participate in international activities concerned with the application, planning and practice of tunnelling
- review from time to time the adequacy of education and training of technologists in the field of tunnelling.

Membership

Members include Commonwealth and State government departments and authorities, companies and other bodies active in the field.

Affiliates are individuals who support the objects of the Association.

Acceptance of a Member or of an Affiliate is subject to approval by the Committee and payment of the appropriate subscription.

Australian Underground Construction and Tunnelling Committee

The Committee shall consist of a Chairman elected by the Committee who shall hold office for two years, and the following members:

- 1 person nominated by each of the I.E.Aust and the Aus.I.M.M.
- 4 persons elected by the Members, each candidate for election being the nominee of one of the Members, and on the staff of that Member. Each Member shall be entitled to make one nomination and cast one vote in the election for Members' representatives on the Committee
- 3 persons nominated from among Affiliates. Each Affiliate shall be entitled to cast one vote in the election for the Affiliates' representatives on the Committee
- Other members shall be
  the immediate past-Chairman of the Association;
  the Chairman of the Australian Geomechanics Society or his nominee; and
  not more than 3 persons nominated by the Committee, to represent areas or disciplines considered to warrant increased representation.

In the event of there being fewer than 7 committee members elected, the Chairman may nominate persons to fill the vacancies, subject to confirmation by the Committee at its next meeting.

The Chairman may nominate replacements for elective members as appropriate to fill vacancies occurring between elections.

Officers of the Association

The officers of the Association shall be a Chairman, a Deputy Chairman and a Secretary. The Chairman shall be elected every second year by the Committee from its own members at the second meeting in each even-numbered year and shall also be Chairman of the Committee.

The Deputy Chairman shall be elected at the first meeting of a newly elected Committee and shall also be Deputy Chairman of the Committee.

The Secretary shall be a joint nominee of the Chairman, the Chief Executive of The Institution of Engineers, Australia, and the Chief Executive Officer of the Australasian Institute of Mining and Metallurgy.

The venue of the Secretariat shall be agreed by the Chairman of the Association, the Chief Executive of The Institution of Engineers, Australia, and the Chief Executive Officer of the Australasian Institute of Mining and Metallurgy, and shall normally be the Organisation employing the Secretary.

Tenure of office of the Committee

Elections shall be held in August of every odd-numbered year.

The new Committee shall take office on 30 September following an election.

Meeting

The Committee shall normally meet twice in each calendar year at the discretion of the Chairman.

Finances

Funds of the Association shall be handled by the Secretariat.

A financial statement shall be circulated to Members and Affiliates of the Association each year.

Funds may be used to pay for or contribute towards the activities of the Association including:

- Subscriptions to other appropriate bodies
- Secretarial and administrative expenses incurred by the Association
- Travelling and other expenses incurred in meetings of the Committee
- Publications approved by the Committee.

Subscriptions

Subscriptions shall be fixed by the Committee from time to time.

AUSTRALIAN UNDERGROUND CONSTRUCTION AND TUNNELLING ASSOCIATION

MEMBERSHIP APPLICATION FORM

1.	FULL MEMBER (Organisation name and Address)	....
		....
		....
		....
2.	AFFILIATE MEMBER STUDENT MEMBER* RETIRED MEMBER# (delete as appropriate)	Mr/Mrs/Ms/Dr/etc..
		....
		....

*Student members please enter details of your course and the tertiary institution in Section 3 below.
#Retired members please enter brief details of your previous professional occupation in section 3 below.

Member of: I.E.Aust YES/No Aus.I.M.M. YES/NO

3.	STUDENT/RETIRED MEMBER further details	....
		....
		....
		....
4.	Name and address for mail, subscription renewal etc.if different from 1 or 2 above	....
		....
		....
		....

I/we apply to join the Australian Underground Construction and Tunnelling Association as a full/affiliate/student/retired member and I/we agree to be bound by the Association Regulations.

Cheque/money order for $..is enclosed, OR
BANKCARD/MASTERCARD authorisation overleaf completed.

Signature..

Date...

Secretariat: 11 National Circuit, BARTON, ACT 2600 Tel: (062) 70 6555 Fax: (062) 73 1488 Telex: AA 72 6548

Jointly sponsored by The Institution of Engineers, Australia and The Australasian Institute of Mining and Metallurgy

AUSTRALIAN
UNDERGROUND
CONSTRUCTION AND
TUNNELLING
ASSOCIATION

METHODS OF PAYMENT

Cheques should be made payable to "The Institution of Engineers, Australia (AUCTA 732016)" Overseas applicants please ensure that your remittance complies with the requirements listed below. Applicants in Australia may also use the Bankcard/Mastercard facility by completing the authorisation form below.

Charge my MASTERCARD/BANKCARD account with $ Aust. ______________________
(Delete not applicable)

Account No. | | | | | | | | | | | | | | | | |

Expiry Date / / Signature ______________________

I.E.Aust Register No. ______________________ (if applicable)

FORWARDING OF APPLICATIONS

Please complete the membership application form, detach it from the brochure, attach your remittance or bankcard authorisation and forward to:

The Secretary,
AUCTA
11 National Circuit,
BARTON, A.C.T. 2600

Telephone: (062) 70 6555
Facsimile: (062) 73 1488
Telex: AA726548

ARRANGEMENTS FOR OVERSEAS REMITTANCES

The following remittances can no longer be accepted for negotiation in Australia unless the special handling charges, recently introduced by the Australian banks, are included in the equivalent value on conversion. They are cheques and drafts:

* drawn in a currency other than Australian dollars; or
* drawn in Australian dollars on a bank outside Australia.

The special handling charges are of the order of $50 Australian each.

The following methods of payment are available:

* a draft or cheque in Australian dollars on an office in Australia of a bank operating in Australia; a list of banks operating in Australia at the present time is set out opposite.
* a direct deposit in Australian dollars (for the value required after deduction of all bank handling charges) to the Institution's account No. 062910 90 1587 with the Commonwealth Bank of Australia, Barton, A.C.T., Australia; *full details of the payment must be forwarded by airmail to the Accounts Manager, The Institution of Engineers, Australia, National Headquarters, 11 National Circuit, Barton, A.C.T. 2600 Australia.*
* an international Money Order in Australian dollars.
* a Mastercard or Bankcard authorisation in Australian dollars.

Advance Bank Australia Limited
Australian and New Zealand Banking Group Limited
Australian Bank Limited
Australian Resources Development Bank Limited
Bank of America Australia Limited
Bank of China
Bank of New Zealand
Bank of Queensland
Bank of Singapore (Australia) Limited
Bank of Tokyo Australia
Bankers Trust Australia Limited
Banque Nationale de Paris
Barclays Bank Australia Limited
Challenge Bank Limited
Chase AMP Bank Limited
Citibank Australia Limited
Commonwealth Bank of Australia
Deutsche Bank Australia Limited
Hongkong Bank of Australia Limited
IBJ Australia Bank Limited
Launceston Bank for Savings
Lloyds Bank NZA Limited
Macquarie Bank Limited
Mitsubishi Bank of Australia
National Mutual Royal Bank Limited
National Australia Bank Limited
Natwest Australia Bank Limited
Rural and Industries Bank of Western Australia
Savings Bank of Tasmania
Standard Chartered Bank Australia Limited
State Bank of New South Wales
State Bank of South Australia
State Bank of Victoria
Westpac Banking Corporation

Australian Geomechanics Society

APPLICATION FOR SUPPORTING MEMBERSHIP
(1991 Subscription $300)

Name of Organisation ....

Address ....

....

.... Postcode ....

Telephone .... Facsimile .... Telex ....

Is your organisation
☐ A Commonwealth Government body or Department
☐ A State Government body or Department
☐ A private or public company or corporation
☐ Other (please specify) ....

....

Supporting members may nominate two staff members to receive individual membership privileges, copies of all publications, etc.

Please provide details:

Title .... Name .... Brief Designation ....
(Dr/Mr/Mrs/Ms/etc.)

Address for mail ....

.... Postcode ....

Telephone .... Facsimile .... Telex ....

Title .... Name .... Brief Designation ....
(Dr/Mr/Mrs/Ms/etc.)

Address for mail ....

.... Postcode ....

Telephone .... Facsimile .... Telex ....

Occasionally, the Australian Geomechanics Society secretariat is asked by Australian and foreign organisations to supply lists of contractors, consultants and equipment suppliers in the geotechnical area. The Society provides the list of Supporting Members to meet such requests. Please indicate below the particular areas of interest of your organisation and state whether you have any objection to having your name included on such a list.

We do/do not object to having our name included in the list.

We apply for Supporting Membership of the Australian Geomechanics Society and agree to be bound by the Statutes of the Society as now in force or as amended from time to time.

Date .... Signature ....

for ....
(organisation)

General areas of interest:

☐ Soil mechanics
☐ Foundation engineering
☐ Rock mechanics
☐ Engineering geology

Specialised areas of interest:

☐ Foundations
☐ Slope stability
☐ Roads and pavements
☐ Earth dams
☐ Field testing
☐ Laboratory testing
☐ Site inspection
☐ Resource evaluation
☐ Underground openings
☐ Mining applications
☐ Other (please specify)

Australian Geomechanics Society

APPLICATION FOR MEMBERSHIP

Title .... Surname .... Other Names ....
(Mr/Mrs/Prof etc)

Address ....

.... Postcode ....

Telephone .... Facsimile .... Telex .... Brief Designation ....

Membership qualifications

1. I am/am not a member of The IEAust (Grade) .... or The AusIMM (Grade) ....
2. This section to be completed by those who are not members of AusIMM or IEAust.

Academic and Professional Qualifications ....

....
(Use recognised abbreviations. Documentary evidence may be required)

Employer ....

Position ....

Activities in Geomechanics ....

....

....

3. This section to be completed by persons applying for student membership

Are you a full time student at an Australian tertiary institution? Yes / No.

When are you due to finish your studies? ....

Name the Institution ....

If you are not a student member of The IEAust or The AusIMM, this application must be accompanied by a letter signed by the Head of Department of your tertiary institution stating that you are a full time student.

International Societies and Subscription 1991

IEAust or AusIMM members .... $37.50 (Student and retired members $19.00) $ ....

Others .... $75.00 (Student and retired members $38.00) $ ....

The above annual fee entitles you to membership of one of the three International Societies listed below. If you wish to be a member of more than one society, you may do so by paying a fee of $8 for each additional Society (maximum $14).
Please circle the Societies you wish to join:

I.A.E.G. I.S.R.M. I.S.S.M.F.E. Additional charge $ ....

If you elected to join the IAEG and wish to receive the IAEG Bulletin, please add $8.00 to cover postage as the Bulletin is posted direct to individual members from France. You cannot subscribe to the Bulletin unless you elected to be a member of the IAEG Bulletin $8 $ ....

TOTAL $

I declare that the information given above is correct. I hereby apply for membership of the Australian Geomechanics Society and I agree to be bound by the Statutes of the Society as now in force or as amended from time to time.

Date .... Signature ....

So that we may provide a better service, please indicate your areas of interest by ticking the appropriate box(es).

GENERAL		SPECIALISED					
Soil mechanics	☐	Foundations	☐	Field testing	☐	Underground openings	☐
Foundation engineering	☐	Slope stability	☐	Laboratory testing	☐	Mining applications	☐
Rock mechanics	☐	Roads & pavements	☐	Site inspection	☐	Other (please specify)	☐
Engineering geology	☐	Earth dams	☐	Resource evaluation	☐	....	

OBITUARY

Keith Grant, previously of the CSIRO Division of Applied Geomechanics in Melbourne, passed away this year in Canberra.

He will be remembered for his work on terrain evaluation and his contributions to pedogenic materials, (laterites, silcretes etc).

Our sympathies are extended to his family.