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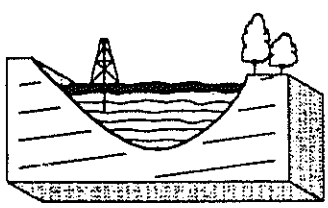
Proceedings of the Symposium on

GEOTECHNICAL ASPECTS OF WASTE MANAGEMENT

WELLINGTON MAY 1994



GEOTECHNICAL ASPECTS OF WASTE MANAGEMENT

VICTORIA UNIVERSITY OF WELLINGTON 13 - 14 MAY 1994

ORGANISING COMMITTEE

Dick Beetham
Stuart Palmer
Fred Smits
Shona Page
Elizabeth Smits

Administered by Janet Simes, Absolutely Organised, Wellington

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IPENZ PROCEEDINGS OF TECHNICAL GROUPS

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PROCEEDINGS OF THE SYMPOSIUM ON:

GEOTECHNICAL ASPECTS OF WASTE MANAGEMENT

ABSTRACT:

This publication presents the proceedings of a symposium on 'Geotechnical Aspects of Waste Management' held in Wellington in May 1994. Authors presented papers on selected topics under the general headings of: Planning and Legislative Requirements, Waste Management, Rehabilitation of Contaminated Sites and Mining. Legislation, investigation design and construction aspects are addressed in the papers.

KEY WORDS:

Geotechnical, landfills, impermeable liners, mining, contamination

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FOREWORD

The Symposium on Geotechnical Aspects of Waste Management was held at Victoria University of Wellington, 13 - 14 May 1994. The Symposium was another in the continuing series which the New Zealand Geomechanics Society arrange on an approximately three year cycle.

1990	Groundwater and Seepage	Auckland
1986	Pile Foundations for Engineering Structures	Hamilton
1983	Engineering for Dams and Canals	Alexandra
1981	Geomechanics in Urban Planning	Palmerston North
1977	Tunnelling in New Zealand	Hamilton
1974	Stability of Slopes in Natural Ground	Nelson
1974	Lateral Earth Pressures and Retaining Wall Design (Workshop)	Wellington
1972	Using Geomechanics in Foundation Engineering	Wanganui
1969	N Z Practices in Site Investigation for Building Foundations	Christchurch

The subject of this Symposium was selected by the Management Committee as an area of current interest and development as a consequence of the requirements of the Resource Management Act and increasing public awareness of environmental issues. Subsequently authors were invited to prepare papers in various topic areas.

The Organising Committee would like to take this opportunity to thank all the authors for the papers prepared for this Symposium.

ORGANISING COMMITTEE

May 1994

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☒ Papers not available at time of printing. These will be published in a special issue of "Geomechanics News"

LANDFILLS UNDER THE RESOURCE MANAGEMENT ACT

J J M Wiltshire, LL.B

SYNOPSIS

The Resource Management Act 1991 has demanded a re-evaluation of whether previous legal provisions allowing the operation of landfills are adequate. Landfill owners or operators need to be particularly aware of the requirements which will apply from 1 October 1994. They also need to be aware of the requirement to obtain all necessary consents and will need to ensure that the consents held cover all elements of the activities they propose to carry out or continue.

The requirements for landfill sites relate to proposed, existing and closed sites and can be provided for by district and regional plans, or by appropriate resource consents. New planning provisions or applications will require consultation with the tangata whenua and local residents. Stringent penalties and enforcement proceedings are applicable under the Act.

INTRODUCTION

Earlier this year, Wellington's Dominion newspaper reported on its front page:

> *"Wellington City Council has applied for six resource consents for the $800,000.00 northern landfill it opened last August but has not been able to use as intended because it did not obtain the consents at the time."*

This report highlights how critical it is to pay careful regard to the requirements of the resource management regime and to get it right the first time. Landfills are inherently expensive, and legal fees and time delays may prove costly.

The Resource Management Act 1991 has demanded a re-evaluation of whether previous legal provisions allowing the operation of landfills are now adequate, not only in respect of proposed landfills but also regarding those which are existing and even those which are currently closed. Moreover, Regional Councils have new found jurisdiction over discharges to land, which is likely to result in a closer scrutiny of landfill and tip operations.

Landfill owners or operators, or intending landfill operators, will need to be particularly aware of the new requirements for discharge permits. In particular, section 15 of the Resource Management Act is a stringent provision relating to the discharge of contaminants into water or air and onto land. Section 15(1) provides:

"15 Discharge of contaminants into environment -

(1) No person may discharge any -

(a) Contaminant or water into water; or

(b) Contaminant onto or into land in circumstances which may result in that contaminant (or any other contaminant emanating as a result of natural processes from that contaminant) entering water; or

(c) Contaminant from any industrial or trade premises into air; or

(d) Contaminant from any industrial or trade premises onto or into land -

unless the discharge is expressly allowed by a rule in a regional plan and in any relevant proposed regional plan, a resource consent, or regulations."

(2) No person may discharge any contaminant into the air, or into or onto land, from -

(a) Any place; or

(b) Any other source, whether moveable or not, -

in a manner that contravenes a rule in a regional plan or proposed regional plan unless the discharge is expressly allowed by a resource consent or allowed by section 20 (certain existing lawful activities allowed)."

Section 15(1)(c) and (d) refer to "industrial or trade premises". Under the definition in section 2 of the Act, which includes the storage, transfer, treatment and disposal of waste material, this term would encompass landfills.

Section 418 of the Resource Management Act (as amended by the RM Amendment Act 1993) is a transitional provision which states:

"(1) *For the purposes of this Act, **section 15(1)(c)*** [discharges into air] *shall not apply in respect of - ...*

(c) Any use of premises for the storage, transfer, treatment, or disposal of waste materials or other waste-management purposes, or composting organic materials; ...

within a region, which was lawfully being carried out before the 1st day of October 1991 and was not subject to any licence or other authorisation under the Clean Air Act 1972 and any regulations under that Act repealed by this Act relating to the emission of air pollutants (within the meaning of that Act), until the third anniversary of the date of commencement of this Act, unless a regional plan sooner provides otherwise.

(1A) For the purposes of this Act, section 15(1)(c) shall not apply in respect of any of the activities relating to the discharge of contaminants specified in subsection (1)(a), (b), and (d) [crematorium, process specified or described in Clean Air Act, factory farm] *that would lawfully have been carried out if they had commenced before the 1st day of October 1991 and would not have required any licence or authorisation under the Clean Air Act 1972 and any other regulations under that Act repealed by this Act; and this shall apply until the third anniversary of the date of commencement of this Act, unless a regional plan sooner provides otherwise.*

*(1B) For the purposes of this Act, **section 15(1)(d)*** [discharge onto or into land] *shall not apply in respect of any activity discharging contaminants on to or into land within a region, **which was lawfully being carried out** before the 1st day of October 1991 and which **did not require any licence** or other authorisation to discharge contaminants on to or into land under any of the Acts, regulations, or bylaws, or parts thereof, amended, repealed, or revoked by this Act, until the third anniversary of the date of commencement of this Act, unless a regional plan sooner provides otherwise."*

Thus owners and operators of existing landfill sites, have had a respite from the requirements of section 15(1)(c) and (d) in respect of discharges into air and onto land, and new landfills have not required consents for discharges into air. However the holiday is nearly over. Unless these factors are clearly provided for in regional plans or regulations and unless procedures are already in place to obtain the necessary consents, now is the time to take a hard look at what may be necessary in legal terms to operate a landfill after 1 October this year.

In this morning's seminar I will focus briefly on the legal requirements of first, closed landfill sites, secondly existing sites and finally new sites. I will also consider the requirements for resource consent applications, particularly the need for consultation and the importance of the assessment of effects. To conclude I will mention the enforcement and penalty sections which give sharp teeth to the provisions of the Act.

CLOSED LANDFILL SITES

With respect to landfill sites which are now closed, I simply wish to draw to your attention the fact that if they continue to discharge contaminants into or onto land, or methane or other significant discharges to air, discharge permits will be required under section 15 after 1 October. Consent to discharge contaminants or water into water are already required.

Existing use rights cannot be relied on as section 20(1) makes no allowance for discharges under section 15(1), and section 15(1) is not subject to section 20. Section 15(2), however is subject to section 20. Therefore existing use rights may be relied on in circumstances limited to discharges into air or into or onto land contrary to a new rule or proposed rule.

I will later discuss the penalties and enforcement provisions for carrying out such activities, in the absence of existing use rights, without consent.

EXISTING LANDFILL SITES

Although the Resource Management Act provides for the continuation of existing designations, town planning consents and water rights, the adequacy, extent and term of these rights ought to be carefully scrutinised to ensure that they provide adequately for all current or proposed activities. Do they cover all activities carried out or proposed on the site, for example recycling depots, transfer stations, compost production, or hazardous substance disposal?

Consideration should also be given when preparing or changing district plans, to the question of whether it is appropriate simply to renew existing designations or to replace them with site specific planning controls. It may also be necessary to widen the designation or consent to encompass existing or future activities on the site which may not be adequately covered at present.

In any event, and although the site may already be designated or have existing use rights for waste disposal, all existing landfills still in use will require additional consents under section 15 from 1 October. I will shortly discuss the requirements for applying for these consents.

NEW LANDFILL SITES

Proposers of new landfill sites will first need to seek land use authorisation. This may be achieved by way of designation, resource consent or plan provision.

Although landfills have traditionally been operated by local authorities pursuant to a designation, this course may be unsuitable if there is a possibility that the landfill will be privately owned and operated in the future. This is because designations cannot be transferred to a private operator, as a private operator does not currently fall under the section 166 definition of a "network utility operator". That definition provides a number of specified roles which will be considered network utility operators, including persons undertaking or proposing to undertake drainage or sewerage systems. There is no express provision for landfill operators, and one may conclude from the other specific inclusions, that the exclusion of landfill operators is deliberate.

Unless there is a provision in a district plan making a landfill a controlled or discretionary activity, or at least providing appropriate policies and objectives, there may well be some difficulty in obtaining consent to a non-complying activity for the purpose because of a potential difficulty in overcoming either of the disjunctive tests of section 105(2)(b) of the Resource Management Act .

An alternative course to obtaining land use authorisation could be for the local authority to alter the district plan and allow the landfill as a permitted or controlled activity (either generally (which is somewhat unlikely), or in a special zone). This could be achieved by either the local authority, which itself controls a landfill, initiating such an alteration to the plan, or by a private operator lobbying the authority for the same outcome. Alternatively, there is now provision in the Resource Management Act for a proponent to specifically apply for a change to a district plan or a regional plan. The advantage of implementing a plan provision in this way is that it will be available to future private operators.

All consents, designations, or plan provisions should be wide enough to cover existing and proposed activities on the site and must clearly define all land use activities and interferences with water and discharges. The consent authority cannot grant more than has been applied for. Nor can the operator do more than is permitted by the plan or consent.

As well as seeking land use authorisation, proposers of new landfill sites will need to obtain discharge permits from the Regional Council for disposal of stormwater and contaminants to land, air and water; and water permits for damming and diversion of stormwater and leachate.

JOINT HEARINGS

The above requirements for new landfills mean that authorisations will need to be sought from both territorial and regional authorities. Section 102 of the Act provides for joint hearings by two or more consent authorities. Subsection (1) of that section states:

"(1) *Where applications for resource consents in relation to the same proposal have been made to 2 or more consent authorities, and those consent authorities have decided to hear the applications, the consent authorities shall jointly hear and consider those applications unless -*

(a) *All the consent authorities agree that the applications are sufficiently unrelated that a joint hearing is unnecessary; and*

(b) *The applicant agrees that a joint hearing need not be held."*

Moreover the Resource Management Act now provides for joint hearing of resource consents along with other consents and/or plan change or designation hearings. Section 103 provides:

"(1) *Where 2 or more applications for resource consents in relation to the same proposal have been made to a consent authority, and that consent authority has decided to hear the applications, the consent authority shall hear and decide those applications together unless -*

(a) *The consent authority is of the opinion that the applications are sufficiently unrelated so that it is unnecessary to hear and decide the applications together; and*

(b) *The applicant agrees that a combined hearing need not be held.*

(2) *This section shall also apply to any other matter the consent authority is empowered to decide or recommend on under this Act in relation to the same proposal."*

Therefore the issues of land use authorisation could be heard together with applications for discharge permits and water permits.

RESOURCE CONSENT APPLICATIONS

Where an activity on a closed, existing or new landfill site contravenes section 15 of the Act, a resource consent - in this case called a "discharge permit" - will need to be applied for. Applications for resource consents are governed by section 88 of the Act. Section 88(4) provides as follows:

"(4) *... an application for a resource consent shall be in the prescribed form and shall include -*

(a) *A description of the activity for which consent is sought, and its location; and*

(b) *An assessment of any actual or potential effects that the activity may have on the environment, and the ways in which any adverse effects may be mitigated; and*

(c) *Any information required to be included in the application by a plan or regulations; and*

(d) *A statement specifying all other resource consents that the applicant may require from any consent authority in respect of the activity to which the application relates, and whether or not the applicant has applied for such consents; ..."*

This subsection is subject to the qualifications in subsections (5) and (6) of section 88, which are discussed below.

Importantly, section 104(3) requires an authority considering an application to discharge to;

" ... have regard to -

(a) The nature of the discharge and the sensitivity of the proposed receiving environment to adverse effects and the applicant's reasons for making the proposed choice; and

(b) Any possible alternative methods of discharge, including discharge into any other receiving environment."

ASSESSMENT OF EFFECTS

Amongst other things, every application for a resource consent must include an assessment of effects on the environment. That assessment shall be in accordance with section 88(5) (if the consent relates to a controlled or discretionary activity) -

"The assessment ... shall only address those matters specified in a plan or proposed plan over which the local authority has retained control, or to which the local authority has restricted the right to exercise its discretion, as the case may be."

The assessment shall also be in accordance with section 88(6) -

"Any assessment ...

(a) Shall be in such detail as corresponds with the scale and significance of the actual or potential effects that the activity may have on the environment; and

(b) Shall be prepared in accordance with the Fourth Schedule."

The Fourth Schedule sets out matters to be included in an assessment of effects. It states:

"1. Matters that should be included in an assessment of effects on the environment -

Subject to the provisions of any policy statement or plan, an assessment of effects on the environment for the purposes of section 88(6)(b) should include -

(a) A description of the proposal:

(b) Where it is likely that an activity will result in any significant adverse effect on the environment, a description of any possible alternative locations or methods for undertaking the activity;

[(c) Repealed by section 225 of the RMAA 1993;]

(d) An assessment of the actual or potential effect on the environment of the proposed activity:

(e) Where the activity includes the use of hazardous substances and installations, an assessment of any risks to the environment which are likely to arise from such use;

(f) Where the activity includes the discharge of any contaminant, a description of -

(i) The nature of the discharge and the sensitivity of the proposed receiving environment to adverse effects; and

(ii) Any possible alternative methods of discharge, including discharge into any other receiving environment:

(g) A description of the mitigation measures (safeguards and contingency plans where relevant) to be undertaken to help prevent or reduce the actual or potential effect:

(h) An identification of those persons interested in or affected by the proposal, the consultation undertaken, and any response to the views of those consulted:

(i) Where the scale or significance of the activity's effect are such that monitoring is required, a description of how, once

the proposal is approved, effects will be monitored and by whom.

2. Matters that should be considered when preparing an assessment of effects on the environment *- Subject to the provisions of any policy statement or plan, any person preparing an assessment of the effects on the environment should consider the following matters:*

(a) Any effect on those in the neighbourhood and, where relevant, the wider community including any socio-economic and cultural effects:

(b) Any physical effect on the locality, including any landscape and visual effects:

(c) Any effect on ecosystems, including effects on plants or animals and any physical disturbance of habitats in the vicinity:

(d) Any effect on natural and physical resources having aesthetic, recreational, scientific, historical, spiritual, or cultural, or other special value for present or future generations:

(e) Any discharge of contaminants into the environment, including any unreasonable emission of noise and options for the treatment and disposal of contaminants:

(f) Any risk to the neighbourhood, the wider community, or the environment through natural hazards or the use of hazardous substances or hazardous installations."

The assessment of effects is important not only as a requirement under section 88, but also as a foundation for evidence before the consent authority, or at the later Planning Tribunal stage. Therefore it is essential that the assessment provides sufficient detail to avoid the possibility of being refused consent or of more detailed and onerous conditions being imposed than might otherwise be the case.

PLAN CHANGES

If a proposal is to be advanced by plan provisions or a plan change, a rigorous examination is required in terms of Part II of the Act, and there is a specific duty on the district or regional council to consider alternatives, assess benefits and costs etc. under section 32 of the Act.

The timing of a Council's section 32 duty was discussed by the High Court in Countdown Properties (Northlands) Ltd and Countdown Foodmarket NZ Ltd v Dunedin City Council (AP 214/93). Except where a privately initiated plan change is not *"adopted"* by the council as its own, a section 32 analysis must be prepared prior to public notification of the requested change. Difficult issues are still to be resolved as to the nature of the analysis required, and the extent to which a report is required.

CONSULTATION

Consultation with local residents is strongly advised. Not only can it alleviate many of their concerns but it could also potentially minimise the number of objections.

Moreover, consultation should also be undertaken by the applicant or a plan making body, with the appropriate iwi. Although the law in this area continues to develop, a recent case to which I will refer, has clarified the position to some extent. While consultation by the consent authority with relevant Maori groups is not expressly required in resource consent applications, I draw your attention to the Planning Tribunal decision in Gill v Rotorua District Council (unreported Planning Tribunal 3/6/93 (W29/93)) where the Tribunal overturned a resource consent on the basis that (inter alia) adequate consultation with iwi had not taken place.

The Tribunal in that case held that the Treaty of Waitangi principle of active protection extended to a duty to actively consult. The duty was not merely incumbent on the applicant (who had in fact gone to extensive lengths to co-ordinate the views of the relevant iwi) but was held to be a duty on the part of the Council to actively consult with the relevant tangata whenua.

It has been more recently held by the Planning Tribunal in J J Hanton & Ors v The Auckland City Council and BP Oil New Zealand Limited (Decision A 10/94), that the Council as a consent authority does not have a duty to consult (following the reasoning in Ngatiwai Trust Board v Whangarei District Council & Ors - Decision A 7/94).

These cases do not however diminish the requirement of an applicant or proponent to consult (and that would include the Council itself when acting as a proponent). Nor do they derogate from the desirability of an adviser to a consent authority investigating and reporting on the extent to which a proposal would affect natural or physical resources which are the object of a valued relationship with Maori people.

CONDITIONS AND MANAGEMENT PLANS

Designations, plan provisions and resource consents will in the current climate almost inevitably contain conditions as to the design, operation and management of the landfill. The consent authority may also wish to incorporate a management plan as part of the conditions of consent. It might therefore be useful to include, as part of an application, possible conditions and/or a proposed management plan, as this will make the authority's task easier and make the authority aware of the type of conditions and/or plan that the applicant wants.

Alternatively, it could be made a condition of the consent that a management plan be prepared and approved by the consent authority or some form of peer review panel. This was the case in Waste Management New Zealand Limited (and others) v Auckland Regional Water Board (Decision W71/92).

The detailed and rather rigorous conditions resulting from the Waste Management case which concerned a metropolitan major landfill, will not necessarily be applicable in a different context, for example to a small rural landfill, or to other urban landfills in different areas. A cautionary note is also expressed in the minute to the parties appendixed to Macraes Mining Co Ltd v Waitaki District Council and Otago Regional Council (C14/94), where Planning Judge Skelton stated that a council has no authority to approve a management plan outside of the normal statutory consent procedures. He held that a resource consent condition requiring preparation and approval of a management plan would be ultra vires. Judge Skelton reaffirmed this approach more recently in Bird v Timaru District Council (C27/94).

In considering conditions, regard should also be given to the recent guidelines prepared by the Centre for Advanced Engineering, although their relevance to different contexts may need careful assessment by technical advisers.

ENFORCEMENT

Part XII of the Resource Management Act provides for enforcement orders and abatement notices.

Both interim and permanent enforcement orders may be made by the Planning Tribunal. Section 314(1) provides:

"(1) An enforcement order is an order made under section 319 by the Planning Tribunal that may do any one or more of the following:

(a) Require a person to cease, or prohibit a person from commencing, anything done or to be done by or on behalf of that person, that, in the opinion of the Tribunal, -

(i) Contravenes or is likely to contravene this Act, any regulations, a rule in a plan, a rule in a proposed plan, a requirement for a designation or for a heritage order, or a resource consent ...

(b) Require a person to do something that, in the opinion of the Tribunal, is necessary in order to -

(i) Ensure compliance by or on behalf of that person with this Act, any regulations, a rule in a plan, a rule in a proposed plan, a requirement for a designation or for a heritage order, or a resource consent; or

(ii) Avoid, remedy, or mitigate any actual or likely adverse effect on the environment caused by or on behalf of that person."

Such orders will inevitably involve expense, delay and disruption.

The Planning Tribunal also has the power to change or cancel a resource consent if the information made available to the consent authority contained inaccuracies relevant to the enforcement order sought, and those inaccuracies materially influenced the decision to grant consent. This function is set out in section 314(1)(e). It again emphasises the importance of careful preparation of the assessment of effects so as to avoid such inaccuracies and the possibility of cancellation of the consent.

Similarly, a local authority can authorise an "enforcement officer" (as defined in section 2 of the Act) to serve abatement notices pursuant to section 322. Under section 322(1)(b) such notices may require an owner or occupier to cease to act or to act so as to avoid, remedy or mitigate the adverse effect identified on the environment.

Failure to obtain consent for unauthorised discharges or other activities may not only result in having the offending activities curtailed at some expense, it may also result in prosecution. Section 338 of the Act provides that every person commits an offence who contravenes or permits a contravention of (inter alia) section 9 (which imposes duties and restrictions in relation to land) and section 15 which restricts the discharge of contaminants. It is also an offence to contravene an enforcement order or any abatement notice. Under section 339 such offences are punishable on summary conviction by a maximum of two years imprisonment or a fine not exceeding $200,000. If the offence is a continuing one, a further $10,000 fine for every day the offence continues, can be imposed.

In the recent High Court decision of Machinery Movers Limited v Auckland Regional Council AP21/93, which was an appeal against a sentence for contravention of section 15 of the Resource Management Act, the Court cited and agreed with the approach of the Ontario Court in R v Bata Industries Ltd (1992) 9 OR (3d) 329; (1992) 7 CELR (NS) 293, calling the case the "most comprehensive and instructive consideration of environmental sentencing criteria which we have found ...". In the Bata decision the Judge said on page 293 of the sentencing report:

"Breaches of these regulations and laws must be dealt with in such a fashion as to prevent their repetition and to foster the principle of environmentally responsible corporate citizenship ...

The purpose of sentencing an offender is to protect the public, to deter and rehabilitate the offenders, to promote compliance with the law, and to express public disapproval of the act ...

There are unique sentencing considerations to bear in mind in public welfare offences, but there can be no doubt that the protection of the public is the primary consideration in sentencing in this field."

This approach confirms that the Resource Management Act places far greater emphasis on environmental protection and introduces a more stringent regime of penalties and punishment than previously applied.

CONCLUSION

Clearly the Resource Management Act leaves no room for complacency when it comes to the operation of landfills, particularly as the 1 October deadline for resource consents for certain discharges is fast approaching. Careful consideration needs to be given not only to the legal requirements in respect of new and existing landfill operations, but also those which are no longer operative. I cannot stress enough the importance of thorough and detailed preparation in embarking on the consent application or plan provision process, which in the long run will ensure the smooth legal operation of landfill sites and will avoid the potential of long term penalties, and in the short term may reduce time delays and unnecessary expense which we have already seen in Wellington.

Although every effort has been made to ensure the accuracy of the information and opinions expressed in this paper, it should not be treated as the basis for forming a decision.

ASPECTS OF EVIDENCE PRESENTATION AT R.M.A. CONSENT HEARINGS

L D Wesley
University of Auckland
Auckland

SYNOPSIS

The presentation of evidence at legal proceedings is an important part of the work of many engineers. It is important that such evidence be presented in a manner which makes it comprehensible to non technical people. Comments are made on evidence preparation and presentation, especially in relation to Resource Management Act consent hearings.

INTRODUCTION

The background to this paper is the writers' experience as a witness at legal proceedings and as a member of tribunals or commissions hearing water right applications for landfills in the Auckland area. The purpose of the paper is to comment on the manner in which evidence is presented, as seen from the tribunal side, in the hope that this may be of some assistance to colleagues who are involved in preparing and presenting such evidence. It is a matter of some concern to the writer that at the end of some hearings non-technical members of tribunals may still be struggling to come to grips with the essential features of the application. This seems to reflect shortcomings in the way evidence is presented rather than inability on the part tribunal members to understand technical matters.

GENERAL COMMENTS

The purpose of presenting evidence is clearly to communicate certain facts and opinions to the tribunal members, as well as to any other parties who have an interest in the application. The presenter is therefore required to communicate with both technical and non-technical people, and is successful in as much as he or she achieves this aim.The witness has not been successful if only technical people understand the material being presented.

Evidence is prepared in written form and may be made available to the tribunal prior to the hearing or be produced at the hearing itself after it has started. The way in which it is presented at the hearing is at the discretion of the applicant (or submitter). Most lawyers acting for an applicant require the evidence to be read verbatim, and this is generally desirable as there is no certainty that all tribunal members will have read all of the evidence prior to the hearing.

Reading technical evidence verbatim is not a good way of communicating its essential features to tribunal members, especially those of a non-techical background. The heavy concentration of technical material may well render it incomprehensible to lay people. Other measures are needed to get evidence across. In particular, verbatim reading needs to be interspersed with breaks where brief "ad lib" explanations are offered to make clear what is being said. The use of clear simplified diagrams (as OHPs) can often be very beneficial as an aid to the explanation of technical matters. These take time and skill (not to mention cost) to prepare but if properly done are well worth the effort.

ADVOCATES OR EXPERT WITNESSES

Technical witnesses are presumably supposed to present unbiased independent evidence, especially witnesses who claim to be professionals. This is not easy to do; technical experts are engaged to assist their clients in supporting one side of a case, and their neutrality is jeopardised as soon as they accept their brief. There are always strong pressures to turn independent witnesses into advocates. Some appear to make no attempt to remain independent, others succumb to the pressures acting on them, and others manage to retain their independence and neutrality very well. Tribunal members tend to be rather bemused by successions of technical witnesses standing up and presenting diametrically opposing viewpoints, in some cases with great conviction and no reservations at all. The writers' view is that some "expert' witnesses do adopt the role of advocates too enthusiastically, and in doing this they are unlikely to be serving the best interests of their client, their profession, or the public. (Some people may well argue that they do serve the best interests of the public, for by expounding opposite viewpoints technical witnesses demonstrate their own fallibility and discourage the public from putting too much faith in so called experts).

THE USE OF LANGUAGE AND DIAGRAMS

Specialised language is an integral part of most if not all specialist disciplines. In some disciplines, the language used is so obscure as to render the discipline incomprehensible to anyone outisde it. This may be because the discipline itself is genuinely obscure or esoteric, or it may be because those who practice it wish to portray it as such. In the case of civil engineering generally, and geotechnical engineering in particular, the ideas and concepts involved are not particularly obscure or esoteric; it can be well argued that most of them are closely related to the everyday experiences of ordinary members of the community. As such it ought to be possible to describe them in language which makes them comprehensible to non-technical people. To a considerable extent this is the case at water right hearings. Much of the evidence is presented in reasonably clear English without undue use of technical terms. However, there is always room for improvement. A few examples may be of value.

The statement "coarse grained highly transmissive materials will be emplaced up the slopes---" appeared at one hearing. The term "highly transmissive" presumably means the same as "highly permeable"; the former would sound like jargon to lay people while the latter would be easily understood. The word "emplace" is something of a mystery to the writer. Looking for it in the civil department office dictionary and in my home dictionary proved in vain;it was not to be found. The nearest to it was emplacement, which was a "platform, usually for guns". The word "emplace" may be an obscure word, a recent addition to the language, or the invention of the user. In any case, why use "emplace" when place is the obvious word to use.

The phrase "zones of elevated hydraulic conductivity" appeared several times at a recent hearing. It presumably has the same meaning as "zones of high permeability". The term hydraulic conductivity may have something to commend it from a technical point of view but it certainly has nothing to commend it if those using it wish to be understood by the general public. Some witnesses have the good sense to always use the term permeability in preference to hydraulic conductivity. If witnesses must use complex words in their evidence, they should at least learn to pronounce them fluently and correctly prior to hearings; some witnesses have great difficulty on occasion pronouncing their own terms (methanogenic comes to mind as a recent example).

The issue of diagrams is just as important as the question of language. Drawings prepared for technical purposes are often rather cluttered with technical information and may not be very helpful in getting across the basic concepts which are of most significance to an application. Conceptual or schematic drawings prepared specifically for a hearing can greatly assist non-technical people to grasp the essential features of a scheme. Fig 1 shows a conceptual drawing presented at the Mt Wellington hearing to illustrate simply the concept of a groundwater divide.

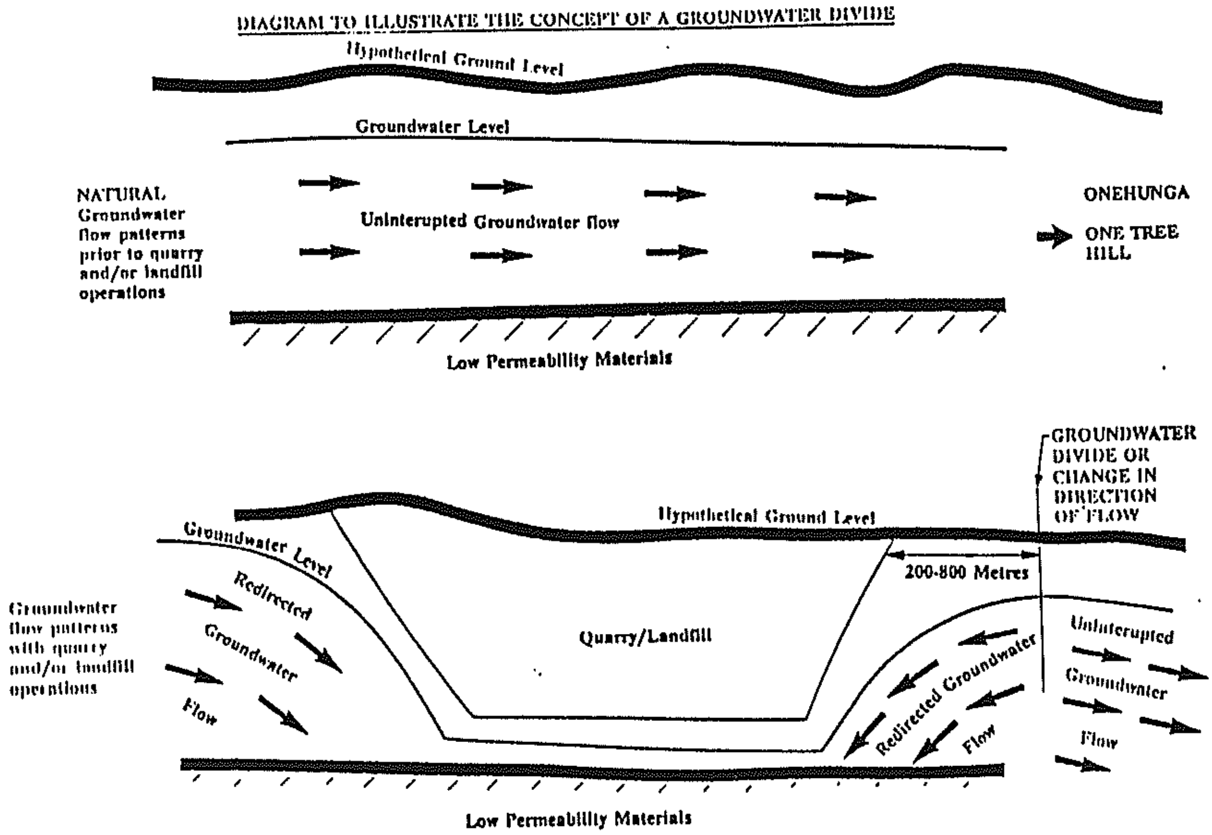

Fig.1 Conceptual Cross-section of Groundwater Flow at Mt Wellington Quarry.

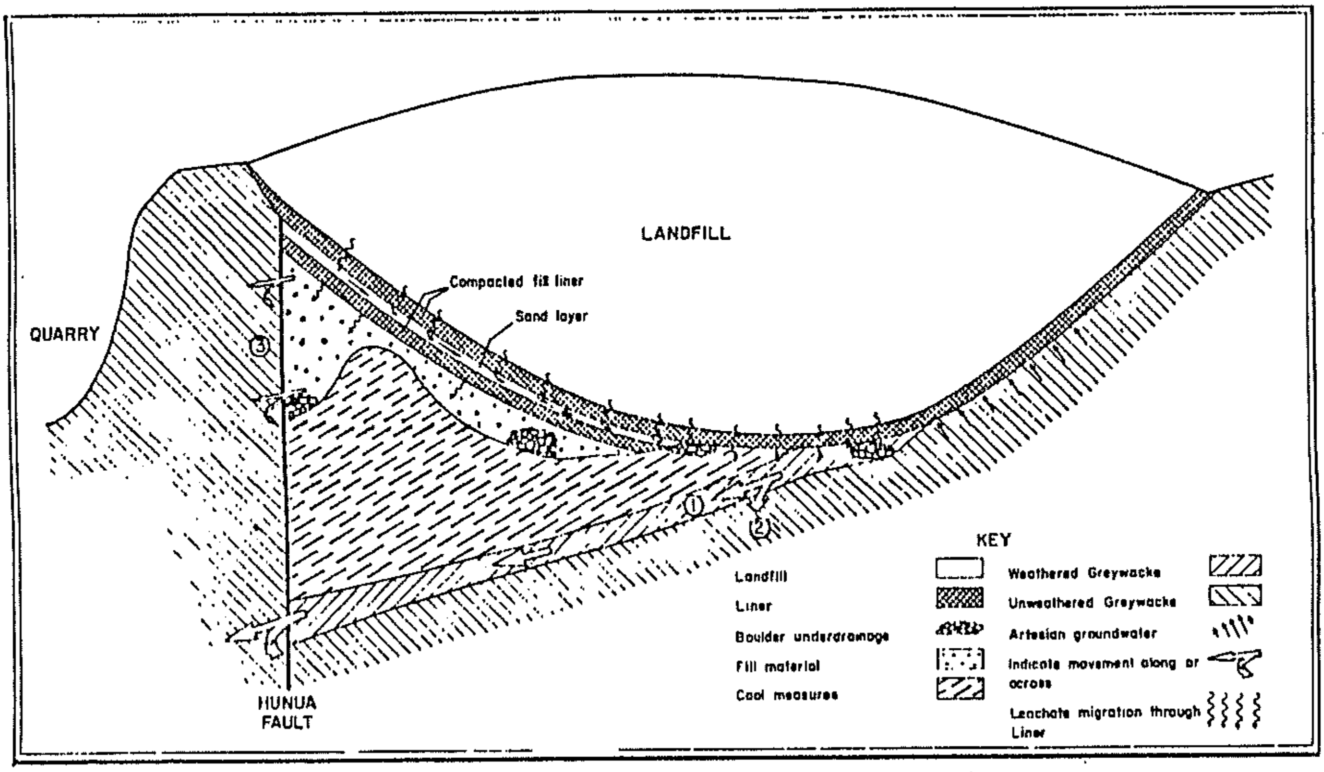

Fig. 2 Conceptual Cross-section of Proposed Peach Hill Landfill.

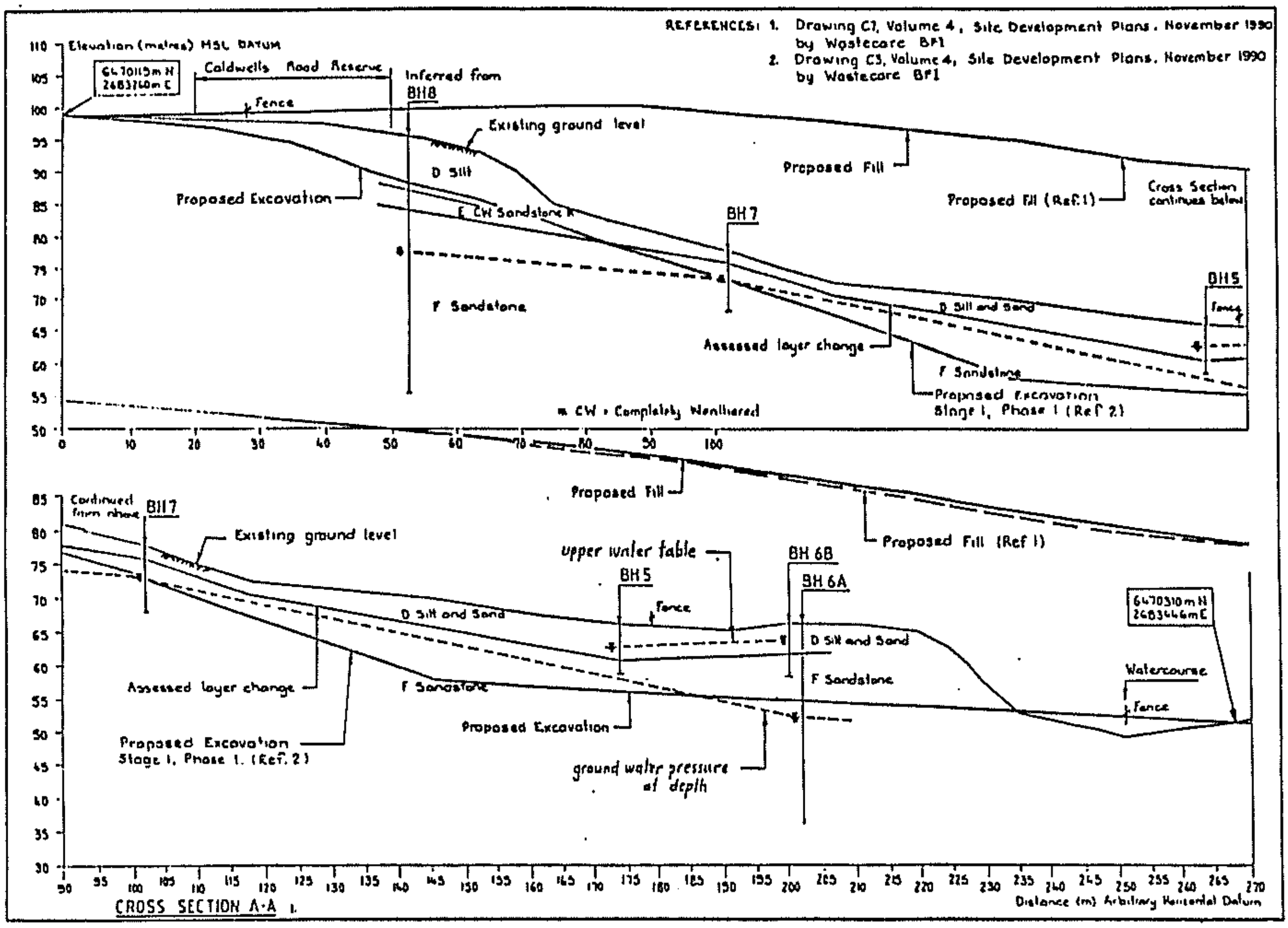

Fig.3 Cross-sections of Proposed Landfill Site at Whitford.

Fig 2 shows a conceptual drawing of the Peach Hill site, illustrating the essential aspects of ground water flow.Fig 3 shows a cross section through a proposed site at Whitford; this is an engineering drawing which has not been simplified for the purposes of the hearing. Figs 1 and 2 are clearly much easier to follow than Fig 3.

CONTENT OF EVIDENCE

Accuracy

It hardly needs to be said that evidence should be reliable, especially evidence of an analytical or numerical nature. However,in the haste to meet tight deadlines it is easy for errors and shortcomings to creep into evidence. Factual mistakes may go undetected, or a systematic sequence of presentation may be lost, with the result that the evidence appears disjointed and uncoordinated. Ideally, evidence needs to be thoroughly checked by someone not directly involved in putting the material together in the first place.

Imaginative or Extravagent Evidence

When no sound technical basis exists by which a scheme can be opposed there is an obvious temptation for technical witnesses acting for opposition parties to come up with quite unrealistic or totally speculative suggestions in order to try to discredit an applicants case. It is possible that the relaxed nature of consent applications tends to encourage the production of some highly imaginative and extravagent evidence. In consent applications evidence is not given under oath and no cross examination is permitted, so the constraints on witnesses with fertile imaginations are not great. This is not intended as a criticism of the present format of consent hearings; the writer believes that the current informal atmosphere of hearings is preferable to the strictly adversarial nature of conventional courtroom proceedings. Also, the writers' experience is that biased and highly speculative evidence is usually seen for what it is, by both non-technical and technical members of tribunals.

CONCLUSION

Effective presentation of evidence at legal proceedings requires considerable care and skill, both in the written preparation of the evidence and in its presentation. Expert witnesses should try to put themselves in the position of those who will hear the evidence, who are likely to be made up of a range of people from those of their own background to those with no technical knowledge at all. Evidence needs to be tailored to meet the needs of both.

REGIONAL COUNCIL'S EXPERIENCE IN THE MACRAES PROJECT

K J Currie
Wellington Regional Council
Wellington

Synopsis

In 1992, Macraes Mining Company Ltd applied for resource consents in connection with a significant expansion of its gold mining venture at Macraes, Eastern Otago. This paper reviews aspects of the processing of those applications from the perspective of the consent authority.

Introduction

The Macraes Mining Company Ltd operates a gold mine at Macraes, Eastern Otago. Mining Licences and Water Rights were issued in 1989 for the project which commenced mining in 1990. In November 1992 the Company lodged with the Otago Regional Council and Waitaki District Council applications for resource consents in connection with a substantial extension to the project. These applications were heard by the Councils' Joint Hearing Committee in March 1993.

This paper reviews aspects of the consent process from the point of view of the consent agency, and concentrates on the consents relating to the extension.

Project description

The initial project which was granted water rights in 1989 was based on an ore resource of 6 million tonnes and an initial production rate of 500,000 tonnes per annum rising to 750,000 to 1,000,000 tonnes per annum. Two adjacent open pits (25 and 7 hectares) would be excavated. Gold extraction would be by carbon-in-pulp leach, with an initial concentration of ore by floatation for the sulphide ore fraction. Floatation tailings (5.2 million tonnes) would be contained in a 95m high dam in adjacent Maori Tommy Gully, with concentrate and oxidised ore tailings (0.9 million tonnes) being contained in a 51 m high dam a little further up the same gully. Some 35 million tonnes of waste rock would be used in the dam structures, and placed in waste rock stacks about the site. Water supply and storage, and silt control structures were included in the consented project.

The extension proposal entailed 35 million tonnes of ore from a series of open cut mines along strike in a north-west/south-east direction from the original Round Hill pit. The processing plant, which had in the interim been increased to process 2.1 million tonnes of ore per annum would increase further to 3 million tpa capacity. The significantly increased volume of tailings would no longer be separated, but would be mixed and the capacity of the tailings dams would be increased top accommodate this. The upper tailings dam would increase to 56 m height, and the lower tailings dam would increase in height to 120 m. Additional waste rock stack would be required, as well as some backfilling of open pits.

In addition to the new consents and variations to existing consents relating to this extension, the Company also sought variations to conditions of existing consents to address operational and management issues which had arisen since the mine commenced operations. One key such variation was sought to change the primary contaminant controls from receiving-water standards to controls on seepage characteristics.

Joint consents

Macraes Mining Co sought resource consents and variations from both the Otago Regional Council and the Waitaki District Council. It was agreed that the process would be managed jointly by the two agencies. A joint team was established involving staff of the two Councils and their advisors. One legal firm serviced both Councils and the practice manager of this firm administered and co-ordinated the joint processing.

The Resource Management Act provides for joint hearings of consents. In this instance, the joint processing was initiated in 1991 as soon as the extension proposal was announced, well before the applications were lodged in November 1992.

Environmental assessment and audit process

Following initial discussions between the Macraes Mining Co and the two consent agencies the Company, in May 1991, prepared a scoping report describing what studies and information it proposed to incorporate in the assessment of environmental effects to accompany the applications. At this stage, the consent agencies assembled an audit team whose role was to review the scoping report, to review the assessment of environmental effects, and to provide advice to the consent agencies.

The audit team comprised a range of relevant expertise, primarily from Royds Consulting, Davie Lovell-Smith, Robertson Ryder and Associates, and the Otago Regional Council. The team and project administration was co-ordinated for both Councils by their common legal advisors, Cook Allan Gibson.

Thus the audit team were able to review the scope and nature of the information to be provided at an early stage. This was useful in focusing attention on the key issues, and served to identify areas where further information was not required.

Over the following 18 months the Company and its advisors prepared the mining proposals and the environmental impact information which would accompany the applications. During this time the Company significantly modified the proposal on several occasions. The magnitude of the proposal was increased; mixed tailings were proposed instead of separating concentrate and floatation tailings; the configuration of tailings dams was modified; and an alternative approach to establishing and monitoring environmental standards was devised.

Members of the Company team and the consent agencies audit team liaised extensively during this period, and a number of technical issues were able to be addressed as they arose which greatly facilitated the review of the final information, and the addressing of issues in negotiations and the hearing. By the time the applications and accompanying information were formally lodged in November 1992, it had already been substantially audited and most issues largely addressed. This was especially valuable in light of the limited time frames allowed by the Resource Management Act for processing applications.

Issue resolution process

The Company consulted with potentially affected people and organisations before and during the preparation of the proposal so was able to address many concerns directly.

Twenty submissions were received when the applications were publicly notified. Five were in support. Eight of the submissions in opposition related solely to the proposal to mine Golden Point, citing the historical value of the old workings which would be affected or removed, and the visual impact from an adjacent historical reserve.

The remaining submissions which opposed applications generally focused on specific issues. Two however (Minewatch Aotearoa and Friends of the Earth, Auckland) included more general opposition to mining *per se*. These submissions proved somewhat more difficult to deal with, particularly as these submittors did not attend any of the informal discussions, pre-hearing meetings, or the hearing. As a result the Company had to approach the hearing on the basis that all issues were potentially open, so had to justify all aspects in evidence, despite many having been resolved with the other parties.

Following the pre-hearing meetings and liaison the Consent agency team put forward a suite of suggested conditions which were substantially agreed between the parties (other than the two referred to above).

The impact of the proposal to mine Golden Point on visual and historical values was not resolved at the time of the hearing. The Company proposed a modified proposition, and withdrew its original application to mine Golden Point with a view to submitting a fresh application for a modified proposal.

While the need for a bond was agreed, the nature and quantum of the bond was not resolved prior to the hearing, and was the subject of subsequent negotiations and consideration by the hearing committee.

Issues

Where to set environmental standards?

In the consents issued in 1989 for the original project, the primary water quality standards were set in Deepdell Creek which flows past the site. No surface discharge of contaminants from the tailings area was allowed, but any seepage from the gully would find its way into Deepdell Creek (after a number of years). Primary water quality standards were set in the creek downstream of the site.

Further, "trigger" levels were defined at groundwater monitoring sites in the gully downstream of the major tailings impoundments. These levels were based on assumed seepage and groundwater flows and characteristics and, if exceeded, required the Company to take appropriate action to ensure that the receiving water standards were not compromised. Because of the imprecision of the assumed seepage model, conservative trigger levels were used, and there was provision to review the trigger levels in the light of operational experience.

Receiving water monitoring showed apparent exceedences of the standards which could not be explained by seepage of discharges from the minesite. Probable explanations for these variations include: natural variability in background water quality (which exceeded initial survey ranges); influence of other activities in the catchment; difficulties in obtaining a representative sample; and analytical variations.

Though the standards set were within method detection limits (using ultraclean techniques), standards for several species were less than the practical quantitation level (PQL) for that species.[1] The PQL's for some chemicals as established by the State of New Jersey Dept of Environment and Energy are set out below in comparison to the Deepdell Creek Water Right limits.

Constituent	PQL ($g.m^{-3}$)	W Right Limit ($g.m^{-3}$)
Cyanide	0.04	0.0054
Arsenic	0.008	0.18
Cadmium	0.002	0.0007
Copper	0.01	0.0065
Iron	0.10	1.0
Lead	0.01	0.0015
Mercury	0.0005	0.0001

It became clear that the receiving water standards as established were unlikely to be able to be enforced unless exceedences were particularly gross, and that the continued use of standards which do not recognise PQLs would continue to cause difficulties in interpretation.

After extensive discussions between the Company and Councils' team, the Company included in its proposal a change from receiving water standards to standards set on the quality of seepage at a compliance site in Maori Tommy Gully prior to seepage reaching Deepdell Creek. A complementary monitoring programme was established, including monitoring groundwater seepage further up the gully to detect seepage contamination sufficiently before it reached the compliance site to allow remedial measures to be implemented. The possibility of seepage taking a preferential flowpath which bypassed the monitoring bores was precluded by the construction of a grout curtain across the valley immediately upstream of the monitoring bores to dissipate groundwater flow.

This approach was endorsed by the Councils' audit team, and was incorporated in the final decision.

Post-mining management

Site management must continue well after mining ceases. This management includes:

* backfilling of some pits and making all pits safe
* covering tailings
* re-contouring waste stacks, tailings dams, site works etc, re-instating soil profile and re-vegetation
* ensuring long term seepage quality and quantity is controlled
* maintenance of impoundments, stormwater diversion channels etc
* monitoring

The consents need to provide for this longer term management, either at the end of planned mining operations, or in the event of some earlier cessation of operations and/or default by the Company.

Principal elements of the above site management measures were defined in the proposal. In view of the evolving nature of the mining project, and uncertainties over what the communities' aspirations for the area might be in 15 - 25 years time, it was decided to defer details of post-mining management until nearer the time. Conditions on many consents require a closure plan (and related consents if required). Such a condition is:

> *"Prior to the expiry or surrender of this consent, the Grantee shall prepare a management, monitoring and contingency plan for the future management of diversions and discharges to the satisfaction of the Council and shall seek appropriate consents for any ongoing activity identified by the Resource Management Act 1991 as requiring a consent.*
>
> *The objectives to be met at all stages of this management plan are to ensure the effective long term containment of the waste and to protect the Shag River, Deepdell Creek and its tributaries and uses and values associated with these waters.*
> *The plan shall make provision for implementation of the plan."*

Post-operational seepage management is intended to be a continuation of the monitoring and (if required)

[1] A PQL is the level at which a parameter can be quantified, irrespective of the laboratory which performs the analysis or, in general, the methods used for sampling or analysis. Variations between laboratories and errors in sampling and analysis are accounted for by using a PQL.

interception of contaminated seepage by pumping. This would continue until monitoring indicates that seepage has attenuated to the point where monitoring and management systems can be decommissioned. Alternative seepage treatment and dilution options are also available.

To ensure that rehabilitation and continued environmental protection will be effected after mining or in the event of a default or operational unavailability of the mining Company, security in the form of bonds is included in the consents as follows:

Existing Ministry of Commerce bond

$1.5 million - for site rehabilitation at the main pit area

Resource consent bonds

Bonds for rehabilitation of other areas to be fixed in relation to annual work programme , with a minimum of $145,000 specified for the Roundhill East area

$4.0 million - term of 51 yrs - contingency for waste management, tailings treatment/relocation and related matters

$285,000 p.a. - term of 7 years after rehabilitation - for contaminant and site monitoring

Flexibility

This project (not un-typically) has been subject to significant changes. This is the result of further ore bodies being mined, changes in mining strategies and techniques, and as a result of operational experience.

Major changes to the mine plan and environmental management systems occurred while the extension project was being prepared, and added almost 12 months to the initially projected time for the lodging of consent applications. These changes included changes to extent of mining and mining sequences, technological changes allowing mixing of tailings, deletion of proposal to deposit tailings in second gully system, and changes to seepage control and monitoring systems. The prospect of such changes should be recognised when establishing project management systems. Clearly changes which occur late in the process will incur greater delays and costs than those which occur prior to significant studies, audits and negotiations taking place.

Flexibility needs to be also built into the consents to provide for ongoing mine management. There is a potential conflict between the desire for certainty and specificity in the consent conditions on one hand, and the desire to allow for modifications to mining operations and techniques on the other.

In this instance the issue was addressed through management plans. The consents prescribe environmental standards and outcomes to be achieved, and require that a management plan be prepared to deal with the more detailed matters of how the resources are to be used and managed to achieve the stated objectives. The management plans are subject to approval and are regularly reviewed.

Costs

The process is costly. In addition to the cost incurred by the Company in preparing and advancing its case, the consent agency incurred significant costs which were recovered from the Company.

Financial management systems were instituted at the outset. The consent agencies prepared estimated budgets, broken down into component parts of the process. These required periodic revision, especially following changes which the Company made to the proposal and the timetable. Monthly accounts from the consent authorities and audit team enabled a better management of cash-flow by the Company and agencies, and meant there were no (or fewer!) surprises.

The total consent agency costs, including the audit team, the hearing, and all the preliminary liaison and dealings was $319,456.78, of which all but $46,700 had been paid by way of progressive payments made over the two years which preceded the Councils' decision.

Conclusions

In my view the process was particularly effective, in that a complex suite of issues were effectively resolved in a manner which was technically credible, and which provided for substantial resolution of points of view in a timely manner which minimised confrontation.

The success of the process was substantially due to close liaison which started early, and particularly to the use of an audit team which was established as the Company was commencing the preparation of its proposal and the assessment of environmental effects. This enabled the Company's and Councils' experts to resolve issues in the preparation stage which greatly assisted the audit of the application documentation, and the negotiation of consent terms. It is important in projects of this size and complexity to allow sufficient lead time, and to involve the consent agencies early.

From the other perspective, consent authorities need to recognise that these processes involve a large and sustained workload, and need to arrange and manage resources appropriate to the job. It is unlikely that Councils will have all appropriate skills in house. This needs to be recognised and appropriate skills engaged so that effective technical liaison can occur early in the process. The engaging of consultants still leaves the Councils with a substantial workload in giving direction, policy and process decisions, and management of the process, and adequate resources need to be provided for this.

The risk of this early collaboration is that changes to the proposal in its formative stages can result in additional costs by the consent agency team where it invests time in proposals which do not in fact proceed. Clearly, these costs are minimised if proposal changes occur early in the project, and strategic judgements need to be made as to when the proposal has reached a degree of definition where the consent agency needs to be involved. Good communication between the applicant and the consent agency is important.

The process needs to be well managed. Where more than one consent agency is involved, they should co-ordinate their processes from the outset. In this instance the full range of consents were jointly managed as a single project, and I recommend this approach. For a project of this size, financial management is important and consent agencies should establish estimated budgets, which are periodically reviewed, and should establish with the applicants an agreed invoicing regime.

While the relationship between the Company and consent agencies was well organised, some actions of submittors were more difficult to manage. Where submittors focused on specific issues, meaningful (but not always successful) negotiations could take place. By and large, submittors were not able to assess in detail a lot of the technical information, and relied heavily on the credibility of the consent agencies' audit team and process to derive a level of comfort in these matters.

Submission which present an "in principle" opposition to the proposal are more difficult to deal with by negotiation and discussion, and this is virtually impossible in cases such as this where such submittors do not participate in discussions, pre-hearing meetings, or even the hearing.

In summary, the process was successful because:

* Good information was provided early.
* There was good liaison between the Company and Councils (including early involvement of audit team).
* Joint processing went well - instituted early.
* pre-application liaison & consultation plus post-application negotiations largely successful (with those who participated).
* It was our second time around - we learned from 88/89 process and operational experience, and knew each other better.
* There was a substantial degree of professionalism by those involved.
* Non-technical submittors could take substantial comfort from the thorough technical audit.

Acknowledgement
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LEGISLATION AND MINING PROJECTS

Chris Mitchell
Phillips Fox
WELLINGTON

SYNOPSIS

The key statutes for mining projects are the Resource Management Act 1991 and the Crown Minerals Act 1991. This paper summarises the essential elements of both statutes and looks at the interrelation between them and questions of process and timing. The paper concludes with a discussion of the main issues raised by the legislation on significant mining projects and the general responses to those issues by consent authorities.

INTRODUCTION

The significant changes in environmental law which came into force in late 1991 were accompanied by equally significant changes to the minerals regime. The new environmental law (The Resource Management Act 1991 and its major amendment in 1993) built on the reorganisation of central and local government in the late 1980s, and replaced the long standing patchwork of legislation on planning, water and air. At the same time, the new minerals law replaced a similarly outdated and insufficient collection of laws on mining and exploration for various minerals.

Both new sets of laws are to be implemented through detailed plans and programmes which will provide the policies and criteria against which applications will be assessed. Generally speaking, those detailed plans and programmes will be issued in 1994/1995, and many of them will not be in force until 1995/1996. In the meantime, both laws contain lengthy and complex transitional provisions, and in some instances previous laws, policies and plans still apply.

In very general terms, the changes in these laws have achieved (as designed) a significant shift towards environmental protection values in the assessment of applications involving the use of natural and physical resources. The effect of these changes on mining, and development generally has been twofold. A pronounced short term effect has been created by the *"transitional"* phase in which the timing of new controls and the impact of their application has been so uncertain as to create a strong incentive to defer new applications. The second, more permanent effect, is the imposition of substantially greater costs in environmental protection programmes, compliance/monitoring and impact assessments. One of the intriguing omissions from these radical legislative reforms is the absence of any mechanism to assess (even in a general way) their impact on the economy. Given the clear policy of the legislation to add value to or retain value in the environment, the absence of some means of identifying the national impact of these new laws is a mystery.

There has been no major litigation involving proposed mining projects under the new legislation. A number of cases have involved transitional provisions which necessarily require reference back to old legislation and transitional planning controls. It is therefore not entirely helpful to try to illustrate the breadth of the changes of the law and their likely impact by reference to recent projects. Good case studies, unfortunately, will not really become available until the plans and programmes are in place, and then, no doubt, only at some significant cost to the participants.

THE LEGISLATION

In a mining context, our two new principal statutes address key issues as follows:

* RMA - Environmental effects: costs and benefits. Policies and controls: resource consent(s) maybe needed.

* CMA - Ownership of minerals: rights to explore for and take minerals. Access on to land.

RMA applies to New Zealand and to the territorial sea (i.e. the 12 mile limit, but not the 200 mile *"exclusive economic zone"*). The controls extend to uses of land, water and air. The controls rely for their implementation on plans which are to be prepared by relevant local authorities (or those which are deemed to exist as *"transitional plans"*) and these in turn are subject to any national policy statements issued by the Minister. Essentially, the basic statutory controls take the form of prohibitions as follows:

* Land may not be used in contravention of a (proposed) district plan unless the activity is expressly allowed by resource consent or is an *"existing"* use.

* Subdivision of land is not allowed unless it is expressly authorised by a district plan or a resource consent.

* Use of the *"coastal marine area"* is not allowed unless expressly allowed by a regional coastal plan or a resource consent.

* Use of beds of lakes and rivers is not allowed unless expressly allowed by a regional plan or a resource consent.

* Use of water (including taking, damming or diversion) is not allowed in contravention of a regional plan unless expressly allowed by a resource consent, and is subject to restrictions even where a regional plan does not control it.

* Discharges of contaminants into the environment (air, ground or water) are prohibited unless expressly allowed by a regional plan or a resource consent.

Responsibility for different resource consents is divided amongst a hierarchy of consent authorities. At the top of the hierarchy the Minister for the Environment can (as will be noted below) call in any application which in his/her opinion is of *"national significance."* In those events the Minister is the consent authority. For consents which are *"restricted coastal activities"* (these may include some mining operations) the Minister of Conservation is the consent authority. For consents involving water, air and discharges of contaminants the consent authority will be a Regional Council. For consents involving land use and subdivisions the consent authority will be the district or City Council. Because a typical mining project will involve more than one consent, it is quite often necessary for more than one consent authority to consider the applications, and in that event RMA provides a mechanism for them to do it jointly.

RMA clearly has a significant impact on any mining project. Obviously, the number of different planning instruments which will be relevant, and the number of different consents which will be required will vary according to location and the type and duration of the proposed mining activity. As might be expected from the list of prohibitions set out above, a typical mining project might well involve a number of different consents and different consent authorities. It will invariably also require extensive (and probably expensive) assessments of likely effects on the environment and consultation with interested parties before the applications are even lodged.

The *"front end"* cost of resource consent applications (which, anecdotally at least, is significantly higher than under older legislation) is mitigated to some extent by the streamlining of the process itself, so that the applications will be determined within a relatively short and predictable period.

The effect of CMA on a mining project depends immediately on whether the project involves a Crown mineral. CMA defines *"mineral"* as:

> *"A naturally occurring inorganic substance beneath or at the surface of the earth, whether or not under water; and includes all metallic minerals, non-metallic minerals, fuel minerals, precious stones, industrial rocks and building stones, and a prescribed substance within the meaning of the Atomic Energy Act 1945."*

The Act re-states the absolute ownership by the Crown of petroleum, gold, silver and uranium minerals. In addition to these, the Crown owns all minerals situated on its own land, or situated on any land which it *"alienates"* after 1 October 1991, and in any case where the mineral was specifically reserved to the Crown by enactment or contract. In practice, the Crown retained mineral rights on any land alienated by it after the Land Act 1948, but they will undoubtedly also have reserved rights in many other cases of alienation from the last century onwards.

If the mining project involves a mineral which is not owned by the Crown, CMA does not apply.

Where CMA does apply, the project will require some kind of permit. Permits may be sought variously for prospecting, exploring and mining.

The policies and criteria against which applications for such permits are to be determined will be found in a relevant minerals programme, but none have so far been issued.

The Minister of Energy must issue draft minerals programmes by 1 October 1994, and the process of preparing these programmes is naturally well under way. It is anticipated that there will be five minerals programmes dealing with:

* petroleum;

* coal;

* metallic metals;

* non-metallic metals;

* building stones and aggregates.

In one sense, these programmes are conceptually similar to the plans which are to be issued under RMA. There are also some similarities in the process of notification and adoption which will be dealt with in the next section.

However, the minerals programmes also highlight the key difference, purpose and approach between the two laws (and thus the reason they were *"separated at birth"*). Where RMA establishes a consent regime for activities which make some use of or have some impact on the natural and physical environment, CMA is concerned with the allocation, often in a highly competitive situation, of the rights to the resources themselves. So minerals programmes under CMA will typically be concerned with methods of allocation, royalties, establishing the size of exploration and mining areas and generally ensuring the *"efficient allocation of rights"* and a *"fair financial return"* (s12).

One of the salient differences between CMA and the old minerals regime which it replaced, is that RMA consents must also be obtained. Whilst the former Mining Act certainly required regard to environmental effects, RMA and CMA now achieve the separate purposes outlined above. Questions of efficiency and price on the resource allocation are dealt with by the Minister (on behalf of the Crown as owner) under CMA, and the environmental *"balance sheet"* is assessed under RMA.

In this respect, the key difference between an exploration/mining resource consent and other applications under RMA is the non-application of the cornerstone principle of *"sustainable management"* to minerals. Shortly before the Bills were enacted, it was decided that the concept of sustainability was too fraught with difficulties to apply to minerals, and so the exception was made. However, this does not mean that *"sustainable management"* is not relevant to exploration/mining resource consent applications - far from it. Indeed, the contrary arguments may often involve the sustainability of other natural and physical resources which will be affected by the mining operation.

The other important aspect of CMA is the changes it has made to the miner's right of access to private land. The holder of a permit under CMA may now enter private land (after ten working days notice) to carry out a *"minimum impact activity"*. Minimum impact activities include the following:

* Geological, geochemical and geophysical surveying.

* Taking samples by hand.

* Aerial surveying.

* Land surveying.

Where access is required for other activities, CMA sets out a comprehensive procedure. The first point to note is that, in default of agreement, there is a wide range of land for which no access will be obtainable for non-petroleum exploration or mining. For other kinds of land, and for all land involving petroleum permits, CMA provides a step by step process to arbitration over access and compensation.

PROCESS

RMA and CMA do not specify the order in which their respective consents are to be obtained, and as neither a resource nor a mining permit is of any immediate use without the other, it follows that the consents and permits may often be sought in tandem. However, for reasons which will be discussed below, it is also often likely that RMA consents will need to be obtained first.

The RMA processes are often perceived to be exceedingly time consuming, but the reality is that delays are still often the result of inadequate preparation and applications. On any activities involving significant environmental effects (and recall the breadth of the definitions of both *"environment"* and *"effect"*) a consent authority cannot accept an application unless it is accompanied by an assessment of effects of the environment *"AEE"*. This must include not only all the matters suggested by the AEE's name, but also identify the people who might be interested in or affected by the proposal and detail the consultation which has been undertaken with them.

Once sufficient work has been done to lodge the resource consent applications for the mining project, the applications will be launched into a timetable involving public notification, submissions, formal hearing and decision. It is possible that relatively low level activities, particularly in remote areas, may have non-notified applications - these are more likely to occur with prospecting or exploration activities. Of course, it is also entirely conceivable that *"minimum impact activities"* will not require resource consents at all.

The timetable for the determination of applications is relatively swift. However, further delays can arise if (as almost invariably occurs with contentious applications) the decision of the consent authority is appealed to the Planning Tribunal. The Court like procedure of the Planning Tribunal and the traditional conduct of proceedings before it (not the least by the parties themselves) tends to create a waiting list, a relatively long hearing, and then finally a delay of some weeks before decision.

There was some anticipation, as yet largely unrealised, at the time of RMA's enactment that its *"additional dispute resolution"* powers might offer a less confrontational approach to environmental disputes. It is certainly an option which should never be overlooked, particularly in cases where a mining proponent is dealing substantially with opposition from affected land owners which will otherwise inevitably spill over into the CMA procedures. Unfortunately, these alternative dispute resolution mechanisms remain largely unused.

Mining operators will need to be particularly conscious, both of the raft of new district, regional and coastal plans which will be appearing during 1994 and 1995 and of the hierarchy established by RMA. The hierarchy is somewhat simplified by the likelihood that no relevant national policy statements will be issued in the foreseeable future. The New Zealand Coastal Policy Statement has moved a step closer with the recommendation of a revised NZCPS by the Board of Inquiry appointed to hear submissions on it. The revised NZCPS suggests some policies which will be relevant to the mining of sands and to petroleum installations, but otherwise it does not deal in specific terms with mining operations.

Regional coastal plans (which are required to be issued by 1 October 1994) are required to give effect to the NZCPS. These plans may be of some local significance to mining operations given the relatively high importance of sands and rocks mined from coastal marine areas. However, many regional and district plans will simply not deal with mining operations unless they are already established within the regional district. In the absence of national policies, there may be very wide differences among regions and districts throughout the country in the approach, if any, taken by their plans to mining operations. As with resource consent applications, the ultimate authority for the resolution of any disputes regarding the content of plans is the Planning Tribunal.

The processes under CMA are rather less complex. Draft minerals programmes will, as outlined above, be notified during 1994, the draft programmes will be publicly notified and open to submission within 40 working days of that notification. At the end of the submission period the Secretary of Commerce is required to make a report on the submissions and recommendations on the draft programme to the Minister who will then make recommendations to Cabinet as to the final form of the minerals programme.

The process of allocating prospecting, exploration and mining permits will be established by the various mineral programmes. There does not need to be a relevant minerals programme for a permit to be sought, but once the programmes are in force such applications will be rare.

An application for a permit under CMA is made either in the relatively simple form provided, or by tender if the Minister should choose to use that form of allocation.

TIMING

For present purposes, the critical aspects of timing are the duration of consents/permits under RMA and CMA and any provisions relating to the lapsing of rights which are not exercised.

The position under RMA is naturally more complex, given the wide range of resource consents. All resource consents may have fixed terms which will generally reflect the likely duration of the activity and/or the ability to foresee environmental effects. Generally, land use consents may be unlimited, but other consents have a maximum duration of 35 years.

Consents under RMA will generally lapse within two years unless they are given effect to within that period, but the consent itself may stipulate a shorter or longer lapsing period. The *"banking"* of resource consents for a project which will involve significant planning, design or development phases can be difficult, and the potential for the lapsing of consents and the changing of rules during a lengthy period, needs to be recognised.

Permits under CMA are relatively simpler:

* Prospecting permits last for two years.

* Exploration permits last for five years.

* Mining permits last for 40 years.

though the permit in each case can specify an earlier expiration date.

Under CMA permits may be extended (unlike RMA where a new application is always required) and there is no comparable lapsing provision.

Again, the differences in approach reflect the differences inherent in controlling environmental effects and allocating mineral resources on a commercial basis.

REVIEWABILITY AND REVOCATION

RMA allows a consent authority to build conditions into consents which allow for the review of all or any of the other conditions (except for the condition on the duration of the consent) to be reviewed at the instigation of either or both the consent holder and the consent authority. These powers are now used, more and more frequently on large applications, particularly to review the adequacy of mitigation measures and monitoring requirements.

In addition to these built in review provisions, a consent authority may review the conditions of the consent where a regional plan has come into effect setting rules relating to water or air qualities, flows or rates of use of water etc, and the consent authority believes that the conditions should be changed to meet the new requirements. The consent authority will always have the ability to review conditions if the information given to it by the applicant contained *"inaccuracies which materially influenced the decision."*

In applications of *"national significance"* the Minister for the Environment has a *"call-in"* power. By this power the Minister assumes the role of consent authority on the application. The power may be used where the Minister *"considers that a proposal is of national significance."* There is a lengthy list of factors which the Minister is to have regard to in any such consideration, and they will quite conceivably include large scale mining and energy projects.

The only power under RMA to revoke a resource consent is granted to the Planning Tribunal. The Planning Tribunal may *"change or cancel"* a resource consent if it is satisfied that the information given to the consent authority by the applicant contained significant accuracies which influenced its decision. Whilst this power will clearly deal with cases of outright deceptive conduct, it may also cover cases where relevant and material expert evidence is shown subsequently to have been wrong. It is a rarely used provision which gives an applicant every encouragement to ensure that the information it provides on significant environmental effects is accurate. If, for example, an expert erroneously anticipated the absence or successful mitigation of an environmental effect, the occurrence of that effect could jeopardise the consent.

The position under CMA is again simpler. The Minister may make changes to a permit with a consent of or on the application of the permit holder, or where the permit itself provides for such changes. The changes can cover extensions to minerals or land covered by the permit and can extend (within CMA limits) a situation of a permit.

The Minister has power to revoke a permit for contravention or non-compliance. The permit holder must first be given 20 working days to remedy the alleged breach, but if the Minister is satisfied that the permit should then be revoked he/she can issue a notice either revoking it or taking ownership of the permit, which may then be allocated to another person. Any notice of revocation issued by the Minister may be appealed by the permit holder to the High Court, and the permit continues in force until that appeal is determined.

ISSUES

Significant mining projects appear to be invariably controversial, and the new RMA/CMA regime has undoubtedly shifted the significant power back to potential objectors and significant burdens onto the prospective operator. There are diverse reasons for the invariable controversies, but whether they be characterised as political, social, scientific or economic they can often be expressed forcefully and relevantly under RMA. Sometimes the basis for objection lies in a perception of direct environmental effects - that the proposed mining activity is incompatible with other uses or that it will degrade some intrinsically valuable part of the environment. Sometimes the objection is less direct - such as the association of a mineral with an obsolete or wasteful technology which is of itself environmentally damaging - but it can be nevertheless powerful.

So the AEE for a mining project should be prepared with not only a view to the immediate direct aspects of environmental impact, but also with an eye on the potential *"enemy."* A significant change introduced by RMA is that any one at all may make a submission on any resource consent application which is notified. This potential underscores not only the importance of but the potential benefits of adequate consultation.

Under both RMA and CMA, persons exercising functions are to have regard to the principles of the Treaty of Waitangi. What does this mean? The question has vexed not just decision makers (including the Planning Tribunal) but applicants and Maori alike. Does it mean that the consent authority or the Minister is to have regard to a relevant claim for the mineral before the Waitangi Tribunal? Does it mean that resource consents or access involving sites of importance to Maori could be limited when that importance was hitherto unknown? These sorts of questions will never be definitively answered. However as the processes of consultation improve and confidence on both sides increases, the provisions may well become less daunting than they appear now.

In the context of a mining proposal which involves clear and unavoidable effects on the environment, the proponent will have no alternative but to look for opportunities of mitigating those effects. Perhaps just as importantly the proponent needs to identify counterbalancing environmental effects. These may include direct benefits to the social environment, such as the creation of wealth and employment or indirect benefits to the physical environment, such as the consequent closure of an environmentally inferior mining operation, or the substitution of a more efficient, or plentiful resource for another.

Specific problems relating to mining projects vary enormously in character depending on the mineral to be mined and the kinds of operation and process. The range of possible environmental effects is of course equalled by the range of possibly affected or interested parties.

Almost all mining involves land clearance and the consequent destruction of vegetation, the creation of noise, interference with surface and ground water and the generation of dust. Almost all mining involves some kind of visual impact. A number of mining operations involve the contamination of the sites they occupy by chemicals used in the mining processes. In the same way as these generic environmental effects are associated with alluvial, or open cast, or underground mining, or quarrying, so generic remedial measures will be expected. These remediation measures are now well known in the mining industry.

A common concern of both consent authorities and other interested parties is the mining operators performance of remediation and restoration work during and after the mining activity. The consent authorities have been tending, under RMA, to a three faceted insurance approach. The first is a monetary security, which will be discussed below. The second is a restraint on the length of the consent. It may be disconcerting to the holder of a mining permit for say 20 or 30 years to receive a resource consent for only half (or even less) of that term. At expiry of the resource consent the activity will require a further consent, and its proponent will need to show not only close compliance with the terms of the original consent but also the adoption, where appropriate, of even higher environmental safeguards. The third and final mechanism is the review power which was discussed above. Under an appropriately worded review condition, a consent authority can effectively take control of any area of perceived weakness or problem in terms of environmental effect. So for example, in a mining operation where there is some concern over the control of silting in a stream, a Regional Council could require frequent monitoring and amendments to mitigation measures to fine tune the response to the problem. The industry will no doubt hope that increasing confidence with the review mechanism will reduce the need to set short consent terms which always tend to place a higher element of uncertainty over the economic liability of a project.

The interesting feature of both RMA and CMA is that they both provide for security for compliance with conditions of resource consents and permits. The security may either be a monetary deposit or a bond. In the case of RMA, the bond will almost invariably be required for ongoing and post closure remedial work, and in default it can be called up and applied towards such work. A bond under CMA principally secures payments to the Crown under the permit, but it may also be forfeited for non-compliance with conditions of the permit. However, CMA does not provide that in the event that the bond is defaulted to the Crown it can be used towards restoration.

As noted above it is not realistic or helpful to look for specific environmental problems which are generic to all mining categories because these are such significant differences in scale, remediability and impact. Nevertheless the most serious problems alleged to be created by mining operations will generally be found on all contentious proposals and will often prove fatal to the proposal:

* Significant off-site effects.

* Permanent effects/contamination of the site.

* Permanent damage to ecosystems.

Under RMA the types of resource consent sought may be quite different but they are ultimately all assessed against the same unweighed criteria. So whether the proposal involves quarrying in an urban environment, or open cast mining in a rural environment, or sand mining offshore from a remote beach, the statutory criteria are much the same. The significant variables are introduced by the district and regional planning documents and of course by the community of affected people. The most consistently effective solution involves a major commitment of money and resources to the preparation of the applications and the consultative processes.

CONCLUSION

The advent of RMA and CMA brought profound changes to the mining industry which will not be fully apparent until the plans and programmes under which their policies will be implemented come into force.

With the exception of some minimum impact activities, it is likely that any mining activity which is not expressly authorised by a regional or district plan will need resource consents under RMA. Whether some of the large scale mining activities which were approved under the old regime would receive consents under RMA and still remain viable is uncertain. What is certain is that any significant mining project which is litigated before the Planning Tribunal be finally approved only if it has been meticulously prepared and environmentally designed.

The changes to the law on access under CMA are well known, and in tandem with the new obligations under RMA underscore the need for mining proponents to carry out effective consultation and negotiation with the community of interested parties.

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CONSENT FOR SLUDGE DISPOSAL, CAREY'S GULLY, WELLINGTON (A Case Study)

R G Stroud
Beca Carter Hollings and Ferner Ltd
Wellington

SYNOPSIS

Substantial works, especially those which have the potential for producing contaminants, will require resource consents from Regional and possibly District Councils, unless the Regional and/or District Plans have rules specially permitting the proposed activity. The resource consent process, as set out in the Resource Management Act, follows formal steps of application hearing and decision, but requires a consultative approach which can if followed properly avoid confrontation and accommodate legitimate concerns for environmental issues.

This paper summarises the successfully complete consent process for establishing a sludge treatment plant, and the disposal of treated sludge in Wellington landfill at Carey's Gully.

BACKGROUND

The Wellington City Council has changed its District Plan to provide for the establishment of a sewage treatment plant on a site south of the Miramar Golf Course near Moa Point. It is proposed that most of Wellington's sewage be treated in this plant and that the treated effluent be disposed of through a long outfall to the sea.

Liquid sludge will be produced as a byproduct of the treatment of sewage. This sludge is a mixture of around 98% water and 2% solids. The solids derive from the settlement of milliscreened sewage (primary solids) and the settlement of the treated effluent (secondary solids).

The liquid sludge contains organic matter and may also contain pathogens and viruses, and a limited amount of heavy metals. Although not highly toxic, the sludge can pose a potential health risk. For this reason safe and adequately controlled disposal is necessary.

There are no opportunities in the Moa Pt area for the disposal of sludge. It is proposed therefore to pump the liquid sludge through an underground pipeline over a distance of some 7 km to a dewatering plant at Carey's Gully. At this dewatering plant a large proportion of the water will be removed from the sludge so that the dewatered sludge can be disposed of to the landfill nearby. It is proposed that the water which is removed will be discharged back into the sewerage system and the dewatered sludge will be placed in the landfill along with ordinary domestic and commercial refuse.

Resource consents were required for these activities. Before dealing with this process it is pertinent to consider the Resource Management Act which governs this.

RESOURCE MANAGEMENT ACT REQUIREMENTS

Section 5 of the Resource Management Act states that:

> "(1) The purpose of this Act is to promote the sustainable management of natural and physical resources.
>
> (2) In this Act, "sustainable management" means managing the use, development, and protection of natural and physical resources in a way, or at a rate, which enables people and communities to provide for their social, economic, and cultural wellbeing and for their health and safety while -
>
> (a) Sustaining the potential of natural and physical resources (excluding minerals) to meet the reasonably foreseeable needs of future generations; and
>
> (b) Safeguarding the life-supporting capacity of air, water, soil, and ecosystems; and

(c) Avoiding, remedying, or mitigating any adverse effects of activities on the environment".

That sets the scene for any action taken under the Act.

Those parts of the Act which are pertinent to this matter are Section 9 and 15 which impose a restriction on the use of land and on discharges of contaminants into the environment.

Section 9 states:

"(1) No person may use any land in a manner that contravenes a rule in a district plan or proposed district plan unless the activity is -

(a) Expressly allowed by a resource consent granted by the territorial authority responsible for the plan; and

(b) An existing use allowed by [section 10 or section 10A].

(2) No person may contravene [section 176 or section 178 or section 193 or section 194 (which relate to designations and heritage orders)] unless the prior written consent of the requiring authority concerned is obtained.

(3) No person may use any land in a manner that contravenes a rule in a regional plan or a proposed regional plan unless that activity is -

(a) Expressly allowed by a resource consent granted by Wellington Regional Council responsible for the plan; or

(b) Allowed by section 20 (certain existing lawful uses allowed).

(4) In this section, the word "use" is in relation to any land means -

(a) Any use, erection, reconstruction, placement, alteration, extension, removal, or demolition of any structure or part of any structure in, on, under, or cover the land; or

(b) Any excavation, drilling, tunnelling, or other disturbance of the land; or

(c) Any destruction of, damage to, or disturbance of, the habitats of plants or animals in, on, or under the land; or

(d) Any deposit of any substance in, on, or under the land;".

Section 15 states that:

"(1) No person may discharge -

(a) Contaminant or water into water; or

(b) Contaminant onto or into land in circumstances which may result in that contaminant (or any other contaminant emanating as a result of natural processes from that contaminant) entering water; or

(c) Contaminant from any industrial or trade premises into air; or

(d) Contaminant from any industrial or trade premises onto or into land - unless the discharge is expressly allowed by a rule [in a regional plan and in any relevant proposed regional plan], a resource consent, or regulations".

Where there are no rules, or the activity is contrary to a rule, then a consent or consents are required, and a specified process to obtain those consents must be followed.

The Act sets down procedures to be followed which are:

- The preparation of an application which includes an assessment of effects on the environment and involves public consultation;
- Public notification of the application;
- Receipt of submissions;
- Hearing of the application and submissions, and issue of decision.

The Act assists in this process by providing a standard application form in which the activity applied for must be described, and requires that an Assessment of Effects on the Environment be attached. The fourth schedule of the Act details the matters which should be included in an AEE and the matters which should be considered in its preparation.

The AEE must describe the proposal, examine alternative sites, routes and methods, assess actual or potential effects of the undertaking, describe mitigation measures, identify persons who may be effected and detail consultation carried out. It also should include monitoring measures proposed.

Matters that should be considered are stated as being:

"(a) Any effect on those in the neighbourhood, and where relevant, the wider community including any socio-economic and cultural effects:

(b) Any physical effect on the locality, including any landscape and visual effects:

(c) Any effect on ecosystems, including effects on plants or animals and any physical disturbance of habitats in the vicinity:

(d) Any effect on natural and physical resources having aesthetic; recreational, scientific, historical, spiritual, or cultural, or other special value for present or future generations:

(e) Any discharge of contaminants into the environment, including any unreasonable emission of noise and options for the treatment and disposal of contaminants:

(f) Any risk to the neighbourhood, the wider community, or the environment through natural hazards, or the use of hazardous substances or hazardous installations."

These then are the provisions which effectively drive an application for a consent to do something which is not specifically provided for in a District or Regional Plan.

For activities such as disposal of waste, which potentially have substantial effects on the environment, a significant amount of work is required in exploring alternatives, in undertaking public consultation and in the examination of means to prevent or at least minimise detrimental effects on the environment.

THE PROPOSAL

The proposal is for the treatment and disposal of liquid sludge which will be produced as a by-product from a sewage treatment plant on a site south of the Miramar Golf Course. Liquid sludge will be produced as a by-product of the treatment process at two stages of the process. Primary sludge accounts for approximately 55% of the sludge volume and will be produced by settlement of the milliscreened sewage. Secondary solids account for approximately 45% of the total sludge volume and are produced by the biological treatment of sewage and removed from the treated effluent by secondary settlement. The sludge will be stored in holding tanks which blend the sludge and provide for some storage at the treatment plant before being discharged into two mixing tanks which provide up to four hours' storage each.

The liquid sludge will be pumped via an underground pipeline to a dewatering plant at Carey's Gully an operational tip, or as currently described a sanitary landfill, at the rate of about 1,000 m^3 per day. This will produce up to 80 m^3 of dewatered sludge per day. The sludge will arrive at the site as a liquid containing 2% solids. It will be dewatered to produce a 25% dry solids cake. The dewatering plant is proposed to be located on a site being formed by the removal of a ridge which is being excavated to supply covering material at the tip face.

The size of the structure is dependent upon the plant details. These will be established through the process of competitive tendering. However, in order to be able to assess the effects on the environment of the building, a concept design was developed based on the maximum sized plant which could be established. The site plan (Fig 1) shows the maximum extent of the building footprint and this has a total area of 2,700 m^2. An elevation was produced to show the maximum height and width of building that could be expected, its general form and likely appearance.

The dewatering process consists of deposition of solids from the liquid and further extraction from the solids by centrifuge on press. This process is shown in schematic form in Figure 2.

The liquid sludge is delivered at a relatively constant rate, but the dewatered sludge is transferred to the landfill area only during daytime operating hours. There is therefore a need for built-in holding capacity in the system. The main holding capacity is provided by two large tanks.

In the process of dewatering, sludge odours may be released.

To prevent emission of odour, the building will be maintained under negative pressure. In addition, the equipment and tanks will have separate covers. Air extracted from the building and from beneath the covers will be removed and passed through a soil filter for removal of odour prior to discharge to the atmosphere. The flow rate of foul air to be treated is estimated to be up to 15 m^3/sec and the retention time in the soil filter will be up to 90 seconds.

The soil filter will comprise a 600 mm layer of active bark/soil mixture overlying a 100 mm layer of bark. That in turn will overlie a layer of pea gravel over rounded river gravel. The total depth of this soil filter is some 1,000 mm from the diffuser pipes to the ground surface. Air extracted from odorous areas is discharged through diffuser pipes into this medium. Odour producing chemicals are removed by physical, chemical and biological reactions which occur as the air filters through to the ground surface.

The filter will be up to 2,200 m^2 in area at ground level and the air will discharge to atmosphere over the entire surface. This is assessed to be of sufficient size and capacity to ensure that the odour producing chemicals are removed from the air. It is expected that there will be no discernable odour from the air discharged.

Drainage from the soil filter will be directed to the leachate collection system.

The water that is removed from the sludge will be directed into the existing sewerage system and conveyed back to the treatment plant at Moa Point. The semi solid sludge will be transferred to the fill area and deposited in landfill.

Around 80 - 100 tonnes of dewatered sludge will be deposited into the landfill each day. This sludge will have been mechanically dewatered to provide a solids concentration of between 20 and 25% and with lime added may be up to 26 or 27% dry solids concentration. This is spadeable consistency and has the appearance of damp topsoil. The addition of a small proportion of lime temporarily stabilises the dewatered sludge. This stabilisation subdues microbial reactions that may otherwise result in the production of odour. Therefore,with this addition of lime, the potential for emission of odour from the dewatered sludge is reduced. Trucks will transport the sludge to the working face of the landfill at a rate of some 10 return trips per day.

A receiving station has been established at the landfill and some 75% of private vehicles visiting the tip use this facility. However, the balance, those with trailers and small trucks, go directly to the tip face as do commercial rubbish collectors. In the management of the landfill operation the current sludge disposal area and rubbish disposal areas are to be kept separate to avoid risk of contamination.

When each load of sludge is deposited it will be mixed with refuse then covered with refuse, with the refuse acting as a bulking agent. International experience shows that a bulking ratio of 4:1, that is four tonnes of dry solids to one tonne of sludge is desired to most effectively incorporate sludge into landfill. At the end of the each day the filled area will be covered with a layer of soil recovered from the valley sides.

The volume of sludge forms a significant portion of the total volume of fill material but because it is moist it softens some components and assists in the compaction of the other waste material. It is also plastic and fills voids and will not therefore significantly reduce the expected life of the landfill.

Covering will ensure that there will be no future odour production. Any leachate from the sludge which will be minimal, will percolate through the landfill and be collected together with other leachate, by the leachate collection system and directed into the sewerage system for treatment at Moa Point.

SITE AND ENVIRONS

Carey's Gully contains Wellington's major landfill operation. It is located approximately 5 km south west of the Wellington Central Business District. The landfill site is approximately 2.5 km north of the coast at Owhiro Bay. See Fig 3.

The landfill is located in the upper section of a gully system which drains down the valley and occupied by Landfill Road. The topography of the gully system comprises narrow gullies, with slopes of typically 40° extending up to spurs. The main ridges are typically 200 m above the base of the gully. The landfill is therefore contained within a steep high sided "basin" from which run-off is funnelled through the single narrow valley.

Access to the landfill is provided by Landfill Road which connects with Happy Valley to the west. Happy Valley Road passes the suburb of Happy Valley, from Brooklyn in the north to Owhiro Bay in the south.

Carey's Gully Landfill was established 15 years ago and has a present life expectancy of over 70 years. Daily average compacted refuse at the landfill is approximately 350 m^3. The landfilling operations fill the bottom portion of the valley. Stage I of the landfill, known as Demolition Gully, has been completed and is in the valley to the south of the current filling operations. In the future, as filling continues, the landfill will proceed up the valley in stages. Stage II, where landfilling is currently in operation, is the most downstream portion of the landfill in question. Stage III, immediately upstream of Stage II, has been prepared for landfilling operations. Drainage works will be extended as necessary in the future in order to conduct the stream and ephemeral streams around the landfilling operations. All activities associated with the compacting and covering of compacted refuse are undertaken at the site. A receiving station has recently been established which minimises the need for public access to the tip face.

The leachate from the landfill is collected by a series of subsoil drains installed at the base of the landfill and discharged to the sewer. In addition, surface run-off from the hillsides adjacent to the landfill is intercepted by cut-off drains above the landfill and diverted via settling ponds to the stream.

Evidence given on subsurface hydrology at the hearing of the application stated that:

"The geological conditions in the area are homogeneous. They do not included ground types such as limestone, volcanic deposits, alluvial materials, internally erodible or collapsible solids or other materials which are either inherently permeable or may contain voids, tunnels, solution channels or other features that may provided a ready passage of groundwater out of the area."

It was also stated that no known:

"Geohydrological assessments or investigations have been undertaken in the landfill area. The nature of the topography and rock conditions would severely complicate carrying out investigations of the regional groundwater patterns in the bedrock and the results would be very difficult to interpret. To investigate geohydrological conditions deep boreholes in hard rock, costing tens of thousands of dollars each, would be required. Even with a number of holes there would be difficulty in reaching other than very generalised conclusions. Such an exercise must therefore be considered of academic rather than political value."

And further that:

"Within Carey's Gully, there are local areas of hard slightly weathered tightly jointed rock outcrops. These would be expected to cause any subsurface flow containing leachate to merge with the surface water in the stream. Thus monitoring of the stream water lower down the valley should provide an indication of the level of contamination in both surface and subsurface flow."

And finally:

"A number of lineaments have been observed on aerial photographs. These could be zones of more highly shattered rock and thus conceivably provide a preferential zone of infiltration of the form. However weathering may have rendered the surface exposures of these lineaments relatively impermeable. The orientation of these features north-south would mean that any infiltration would not discharge into Happy Valley but be held within the zone of more permeable rock mass. Discharge into other surface water (apart from eventually the sea) appears to be unlikely and would only occur after a very considerable time."

Land to the west of Carey's Gully is largely rural in nature. In the immediate vicinity the land is owned by Wellington City Council. Land on the southern side of the landfill is undeveloped. To the north the landfill can be viewed from Ashton Fitchett Drive in the Panorama Heights subdivision. The closest residential dwellings to the site are in Happy Valley Road, approximately 800 m west/south-west of the site.

CONSENTS

The site for the proposed dewatering plant and for the disposal of the dewatered sludge is the main landfill for Wellington in Carey's Gully. This is located in a large deep gully system on the south west edge of Wellington but topographically isolated from the urban area. The landfill operation was established some 15 years ago and is authorised by a designation over the land. The designation is for "Refuse Disposal and Associated Works" and for "WCC Purposes". It is arguable that dewatered sludge is refuse and that a dewatering plant is an associated work and that therefore these activities are provided for in the District Plan and do not require consent. However, the process of dewatering will produce contaminants such as air emissions and liquid effluent, and the depositing of sludge in the landfill may cause a discharge of contaminants into the air and onto land which may enter water.

These discharges do require consent from Wellington Regional Council as there are no rules in a regional plan at this stage, and the designation does not constitute a resource consent for this purpose.

Because application for consents were required to be made for discharges of contaminants, and because of the uncertainty relating to the designation was decided that the application package should address all activities. In this way the Council would avoid criticism of lack of transparency and ensure that it could not be attacked in future, for not having the necessary consents.

Therefore the applications that were made addressed the following:

- Construction, use and operation of sludge dewatering building and equipment.
- Construction and use of land for a soil filter.
- Use of land for disposal of sludge.
- Discharge of sludge onto and into land where it may, through natural processes, enter water.
- Discharge of air from soil filter into air.
- Discharge of contaminated air from sludge in landfill.

The first three of these applications would be submitted to Wellington City Council and the last three to Wellington Regional Council.

ASSESSMENT OF EFFECTS ON THE ENVIRONMENT

Applications for resource consents are required, as described earlier, to be accompanied by an assessment of effects on the environment. Such an assessment was prepared describing the activity, assessing the different effects or potential such as noise, odour, water contamination, traffic, cultural, visual, social, heritage and so forth. The different site options were discussed, the treatment option and disposal options considered. Options such as incineration, electricity production, gas recovery and composting were all addressed and the reasons for non adoption explained.

Details on how nuisances or hazards would be avoided or mitigated were included.

PUBLIC CONSULTATION

An inherent part of the process of project development, assessment of effects, and formulation of measures to address any affects, is public consultation.

The choice of disposal site, methods of sludge treatment, methods of transportation and transport routes were all examined at earlier stages in the consideration of Wellington sewerage treatment options. These issues were all exposed to full public consultation and debate.

In the examination of issues relating specifically to Carey's Gully and the activities of sludge dewatering and disposal proposed thereon, a fresh round of consultation was undertaken.

The process involved initial contact with parties considered to have a special interest in the project and the distribution of a summary hand out which described the nature of the proposal including location, construction and operation. In a covering letter the commercial operators on Landfill Road, interest groups and organisations were encouraged to respond with any queries or concerns about the proposal.

Consultation with the residents in the general area adjacent to the proposed site focused on a public meeting held in Happy Valley. Prior to the meeting copies of the summary handout were hand-delivered to all residential properties in Happy Valley Road with a covering letter advising of the meeting. The meeting was attended by 45 residents as well as officers from Wellington City Council and consultants.

The proposal was explained in detail, including how the sludge was proposed to transported to the landfill area, how it would be dewatered and deposited in the landfill, the method proposed to be used for containing odour and how extracted water and leachate would be collected and returned to the sewerage reticulation system. This was followed by a general question and answer session.

Participants were divided into small groups to identify issues and potential mitigation measures. Responses covered aspects of the sludge proposal as well as existing operations at the landfill and the current sewerage reticulation system. A number of concerns were raised including those with the existing contamination of the Owhiro stream.

Consultation was also undertaken with the Tangata Whenua.

The Council's response to concerns was to prioritise the programme for pollution elimination (sewage infiltration in stormwater) in the Brooklyn area and a given commitment to commence a sampling programme to measure water quality in the Owhiro Stream.

PUBLIC NOTIFICATION SUBMISSIONS AND HEARINGS

The application was made on the and the Wellington Regional Council and Wellington City Council duly notified these. A twenty working day period is allowed for submissions, and before this period had expired a total of 11 submissions were received on the various applications that were made. The submissions came from several individuals, a number of resident associations, two special interest groups, the Public Health Service and Waste Management NZ Ltd.

A number of submissions that were made against the proposals and were clearly motivated by the desire to reopen consideration of other site options for sewerage treatment plant. The Public Health Service submission was supportive but sought that conditions be applied to protect public health.

The concerns of Waste Management Ltd were with the adequacy of information presented and the terms and conditions of the consent sought. The motivation was evidently to ensure that all waste disposal projects, whether pursued by local authorities or by the private sector, were all dealt with in the same manner, that is that in the resource management process there should be no privileged position for local government.

The hearing of the application and of submissions was preceded by a prehearing which clarified some issues but did not effectively resolve any.

A hearing was convened and the Hearings Committee comprised a Joint Hearing Committee appointed by Wellington Regional Council and Wellington City Council, consisting of three councillors of Wellington Regional Council and a commissioner appointed by the Wellington City Council.

On completion of the hearing and before making a decision, Wellington Regional Council requested all submittors and other parties to submit proposals for conditions. Most responded including both Wellington Regional Council and WCC with vast numbers of conditions.

THE DECISION

The committee in granted the consents. In doing so stated that it had "referred to the definition of *"Sustainable Management" in the Resource Management Act 1991 as including managing the use of and protection of natural and physical resources*

> *in a way which enables people' and communities to provide for their social, economic and cultural wellbeing and for their health and safety whilst sustaining the resources, safeguarding its life supporting capacity and avoiding or mitigating adverse effects.*

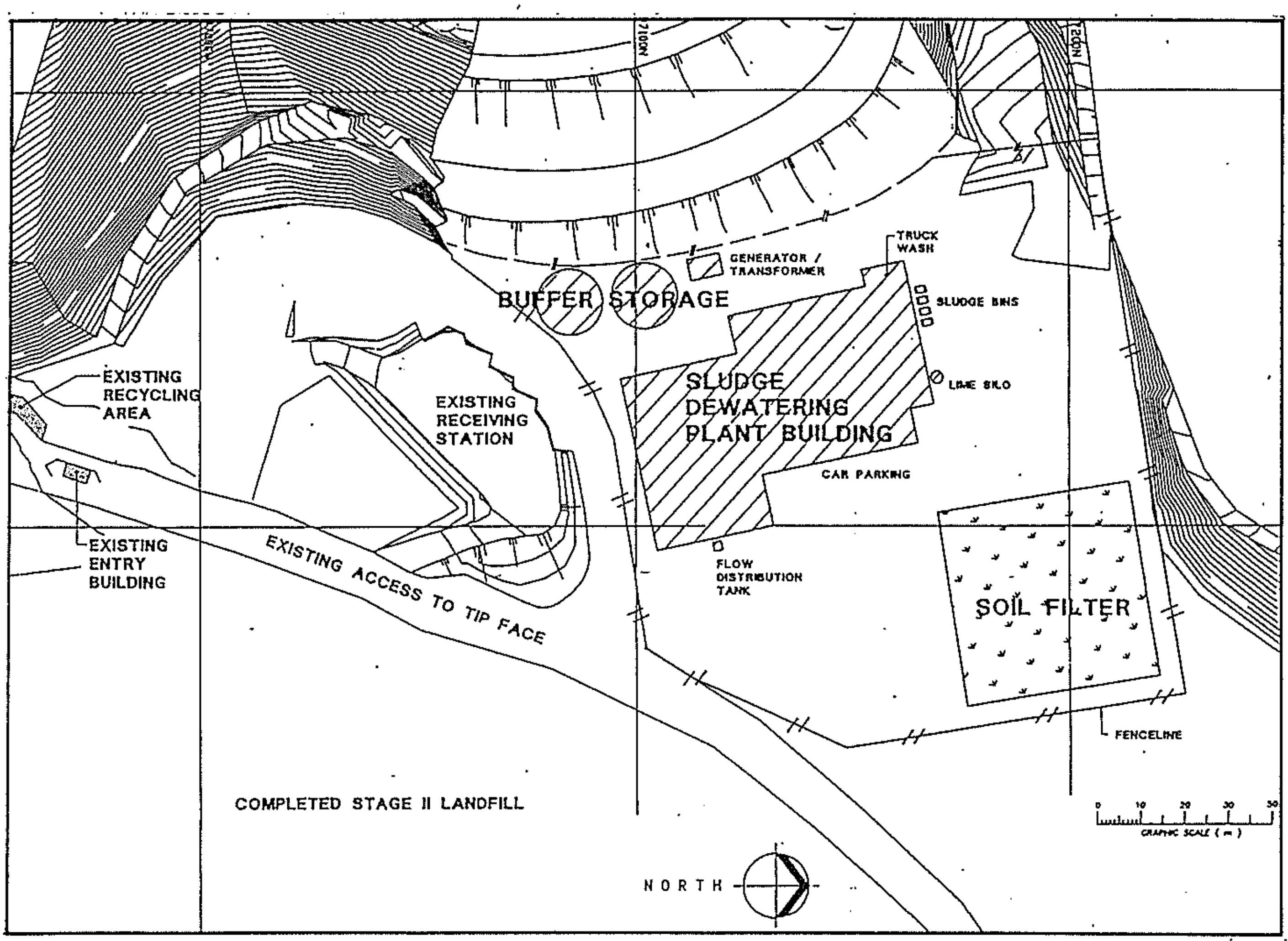
The committee considered that the proposal for the disposal of sewage and waste came within this definition."

It went on to say amongst other things that it had "considered the submissions on the applications and believes that all legitimate issues and concerns have been covered by conditions on the consents".

And further "the committee overall considered that based on expert evidence it had heard from the various witnesses called on behalf of the applicant, the consents sought should be granted, subject to the specified conditions on the consents.

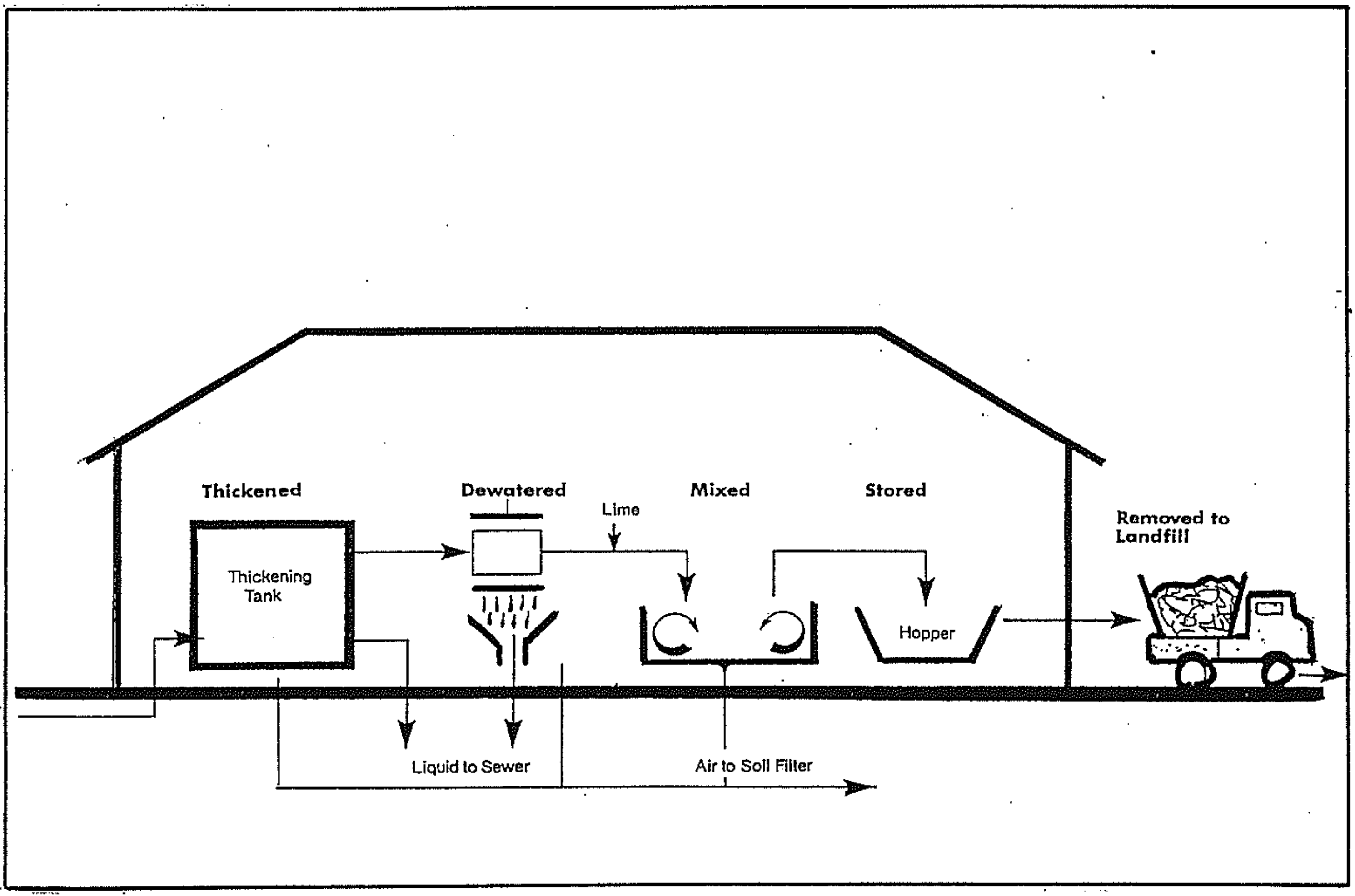
Most of the conditions simply reinforced particulars of the application. These included conditions requiring that the dewatered sludge be not less than 20%, that there by no offensive odour beyond the boundary, that the refuse be covered at the end of each day and that a leachate collection system be provided and maintained.

A number of the conditions were been appealed against by the Wellington City Council. Most of these were not of a critical nature but the appeals sought clarity. These issues have now been resolved by agreement and the appeals withdrawn. The project has now all the required consents.



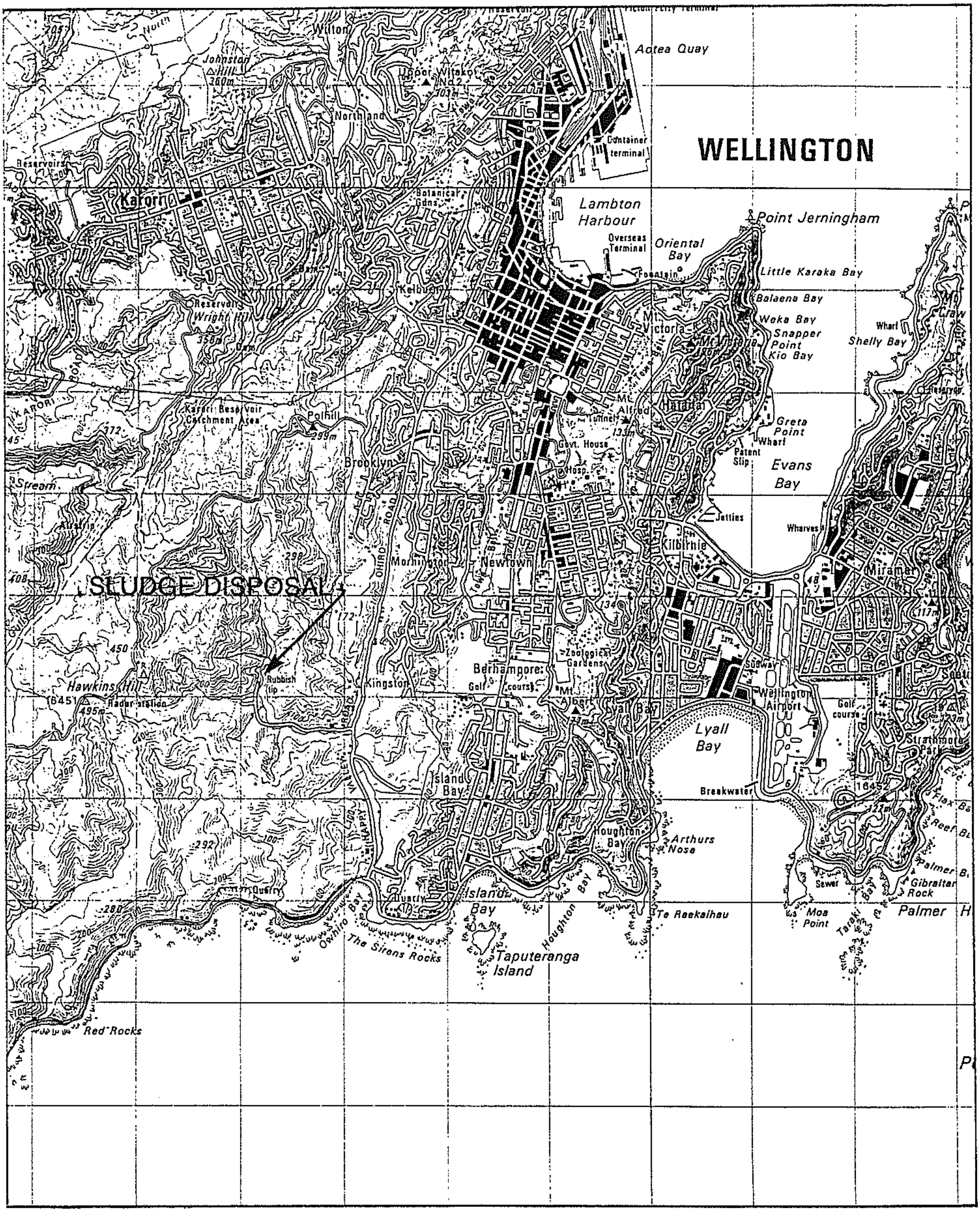
SITE PLAN

FIGURE 1



SLUDGE DEWATERING PLANT SCHEMATIC

FIGURE 2



LOCATION PLAN

FIGURE 3

GEOTECHNICAL FACTORS ASSOCIATED WITH REFUSE LANDFILL SITE SUITABILITY - ARE WE ON THE RIGHT TRACK ?

P.I. Kelsey
A.H. Nelson
Earthtech Consulting Ltd
Auckland

SYNOPSIS

Current assessments for site suitability for municipal refuse disposal is focused on natural ground conditions and engineered liners which provide security in terms of potential contamination of surface and groundwaters. Sites which provide "perfect" geotechnical conditions are very rare.

Current practice with respect to new sites is reviewed in light of site performance from landfills of significantly lower engineering standards. Impacts from these sites are at a lower level than expected according to current practice.

INTRODUCTION

Municipal refuse disposal methods have undergone major changes in the past ten years. In many areas in New Zealand past common practice involved numerous relatively small scale uncontrolled refuse tips. These were often located adjacent to or near harbours or rivers in relatively close contact with natural surface and groundwaters.

Current refuse disposal practice, particularly for new sites is focused on containment issues where engineered landfills are designed to provide a low risk of contamination of surface and groundwaters.

Geotechnical factors play a major role in providing site containment in addition to ease of construction, ease of permitting and cost effectiveness.

This paper addresses current practice regarding geotechnical issues and the landfill site selection process. Examples are provided from regional studies. Performance monitoring from existing refuse landfills constructed to significantly lower standards is also discussed with a view to assessing the appropriateness of the new standards.

GEOTECHNICAL FACTORS

Geotechnical factors taken into consideration during site selection are:

i) <u>Containment of Leachate and Landfill Gas</u>

- Site containment is probably the major geotechnical issue in landfill site selection.

  Favourable hydrogeological conditions include soil and rock materials characterised by low to very low hydraulic conductivity (10^{-7} to 10^{-10} ms^{-1}) with corresponding low groundwater flows (less than $100m^3d^{-1}$).

- <u>Upper Catchment Sites - Groundwater Recharge Areas</u>

To assist in the control of surface water, proposed landfill sites are commonly located in the upper-most reaches of surface water catchments. Such areas are normally zones of groundwater recharge and thus groundwater protection is required.

A 'closed' single catchment groundwater system is preferred to minimise contingency options should groundwater contamination occur. In such a situation extraction bores or a trench constructed immediately below the landfill toe would be able to intercept any contaminant plume. With low groundwater flows from low hydraulic conductivity conditions, volumes of groundwater requiring treatment in a worst case scenario would be relatively small (say less than $100m^3$ d^{-1}).

Hydrogeological security is also provided by groundwater divides under boundary ridges associated with the site. Where located in the upper reaches of the surface water catchment, the landfill footprint will be relatively close to groundwater divide positions. Groundwater divide continuity needs to be assessed in terms of variable geology or rock mass features (such as open jointing or fault zones) providing moderate to high hydraulic conditions drawing down the groundwater table in this area. Landfill construction (liner and capping) will also significantly modify catchment hydrology with a reduced area available for groundwater recharge. Reduced groundwater recharge would in turn result in the lowering of groundwater divide positions and this would need to be assessed.

If deeply incised catchments are located adjacent to the site, topographical effects can cause the drawdown of "cross-catchment" groundwater. This topographical effect has been encountered in site evaluations.

- Lower Catchment Sites - Groundwater Discharge Areas

  Usually less common than upper catchment sites due to surface water control and other constraints, sites located in groundwater discharge areas can provide favourable containment due to upward groundwater flow.

  Groundwater discharge would be collected by groundwater diversion drains. Liner leakage would also be intercepted by the diversion drains. Sites characterised by low hydraulic conductivity would be favoured in such conditions due to low groundwater discharge flows requiring treatment if contaminated.

- Attenuation

  Attenuation reduces the effect of landfill contaminant discharge. Natural attenuation occurs both in the unsaturated zone (including the liner if present) and within groundwater as contact is made with saturated soil and rock materials. Attenuation involves the following processes; adsorption, biological uptake, cation and anion exchange reactions, dilution, filtration and precipitation reactions (Bagchi, 1983).

  Attenuation has been relied upon in the past to 'treat' leachate discharge from unlined sites. Under current practice, attenuation is usually assessed in terms of mitigating the effects of very low volume leakages from lined landfills.

- Landfill Gas

  Discharge to the atmosphere of landfill gas via the working face and through the intermediate and final capping layers is becoming a major permitting issue. Of relevance to groundwater quality is the leakage of landfill gas into the ground through the side walls and base of the landfill.

  Landfill gas can also migrate through liner systems. In situations where natural ground adjacent to the landfill is unsaturated and contains open joints or similar features, off-site gas migration is a real concern in urban areas.

  The initial migration of landfill gas through the volcanic soils and rocks surrounding the Greenmount Landfill in Auckland has been well documented. Control of the gas has been achieved by installing a perimeter gas well extraction system. The bulk of the gas is collected by internal wells and used to generate electricity.

  Of interest is the effect of landfill gas on the groundwater quality. Careful monitoring of the leachate and groundwater quality has identified a small but detectable effect on the groundwaters due to a phenomenon known as the "gas transfer effect". Landfill gas consists largely of methane and carbon dioxide with trace levels of other gases and contaminants. The effect of gas migration is principally due to the CO_2, which lowers the pH and results in a slightly more acidic and harder groundwater. Very low levels of ammonia and VOC's (Volatile Organic Compounds) have been detected - these are believed to be a result of gas migration.

  Water quality in the volcanic aquifer directly beneath the Greenmount Landfill meets drinking water standards and the gas transfer effect is not considered significant. However, recognition of the gas transfer effect has allowed a distinction to be drawn between the effects of gas migration and the effects of leachate leakage. After 14 years operation there is no detectable leakage of leachate through the clay liner at the Greenmount site.

- Engineered Containment Systems

  During the site selection process, a principal objective is to identify a site with the most favourable natural containment. Absolute security usually cannot be relied upon from natural containment and thus engineered lining systems are required. The detail of such systems and their particular applications are beyond the scope of this paper. In summary they range from a single clay liner to systems consisting of double clay/synthetic composite liners. In general, where redundancy cannot be provided by natural site containment, more sophisticated liner designs are adopted.

  Stability issues need to be carefully addressed with synthetic liners in particular, as very low site interface friction angles can occur between synthetics and soils.

ii) Slope Stability

Where an engineered liner system is to be used, the stability of natural sideslopes needs to be ensured in order to avoid liner rupture.

Small scale slope instability, of the order of 5m in depth can usually be stabilised with regrading or drainage. An example of a large landfill site which required extensive regrading which was visited by the author is the Bee Canyon Refuse Landfill in Orange County, California. The 110 million cubic metre air space (30 year life for 3 million people) site extended over 150 hectares. Approximately 20% of the valley area was covered by landslides up to 20 metres depth.

Large scale landslides which are deep seated (generally greater than 10 metres depth), and cover a large portion of the site are generally considered a fatal flaw in terms of the site selection process. Large landslides provide potential problems with short term instability during construction and overall instability following completion. Apart from the stability issue, ground and groundwater conditions can be very complex and difficult to understand and thus permit.

In hilly terrain large landslides can provide what appears to be favourable topography for landfill development. During a site selection study in West Auckland, a large scale landslide was identified from geomorphological evidence with approximate dimensions of 1 km x 1.2 km. This site was not taken past the initial desk study phase of investigation due to this stability constraint.

iii) Foundation Conditions

Landfill depths can reach 30m to 40m resulting in floor loads in the vicinity of 400kPa. Any resulting settlement needs to be within limits which can be tolerated by the liner and leachate collection system. Compressible soils usually need to be removed prior to landfill construction.

iv) Construction Materials

A favourable site is one from which landfill construction materials can be sourced on-site. Natural soil liner suitability is primarily dependent on hydraulic conductivity with 1×10^{-9} ms^{-1} being adopted as the recommended minimum value in New Zealand (CAE, 1992). Suitable liner materials generally have a fines content (clay and silt) greater than 30%, PI greater than 10% and rock fragments no larger than 50mm (Daniel, 1990).

Values higher than 1×10^{-9} ms^{-1} may be considered if used in conjunction with a synthetic liner or if the design has redundancy elsewhere. Clay liner thicknesses generally range between 600mm and 900mm. Site specific liner trials should be undertaken prior to full scale construction to demonstrate that the design criteria can be achieved under field conditions.

Erosion and dispersion behaviour of liner materials also needs to be checked.

The primary requirement for intermediate cover is for trafficability. Intermediate cover may need to be supplemented with low grade aggregates for winter conditions.

Final capping layers generally consist of at least:

- 100mm topsoil
- 500mm clay rich soil
- 200mm intermediate cover to final lift

In a number of New Zealand landfills there is a trend favouring the development of "wet" landfills via leachate recirculation and other means. This removes the need for a highly impermeable cap as some ingress of moisture is desirable. This approach is very different from the USA "dry tomb" philosophy where major efforts are made to exclude as much moisture as possible from the landfill.

v) Seismic Hazards

Potential disruption of a liner system from active faulting is considered to be a fatal flaw for site selection purposes. Particularly in seismically active areas potential fault rupture needs to be specifically addressed. Liquefaction can also preclude a site for landfill development due to loss of foundation support affecting liner and leachate collection systems. The surcharge effects of a landfill can mitigate potential liquefaction in certain situations.

SCOPE OF NEW LANDFILL INVESTIGATIONS

Geotechnical investigations are staged. At each major decision making point, care should be taken so that all sites are to the same level of understanding to ensure the process is technically unbiased. The stages are generally as follows:

i) Constraints Map Preparation

Desk study review and an initial assessment of geology from most favourable to least favourable conditions. Areas of "poor" geology defined on constraints map.

ii) Site Rating Assessment

Sites selected on topographical suitability are inspected from the air and public vantage points. Inspection of roadside exposures are undertaken. Usually between 50 and 100 sites are rated for a major new landfill.

iii) Preliminary Field Investigations

Preliminary field investigations of generally three sites involving mapping, hand-auger bores, test pits and drilling if required. Development of site design concepts.

iv) Detailed Field Investigations

Comprehensive site investigations for Assessment of Environmental Effects for permit applications of the preferred site. Detailed mapping, borehole investigations, geophysical and CPT investigations if required.

For the above staged approach geotechnical factors are incorporated with other social, cultural and planning issues as part of the overall site selection process.

REGIONAL STUDIES

Earthtech Consulting Ltd has been involved in landfill site selection studies in the Auckland, Bay of Plenty and Manawatu Districts. An outline of the favourable areas in each of the Districts in terms of geotechnical factors is summarised as follows:

i) Auckland Region

Large areas of Auckland are underlain by Tertiary soft rocks of the Waitemata Group and Northland Allocthon. Large currently operating landfills are located in this geology are at Rosedale, Redvale and Whitford.

Waitemata Group materials are generally characterised by clay-rich residual soils which provide reasonable landfill construction materials. Waitemata bedrock overall is characterised by low hydraulic conductivity (10^{-6} to 10^{-8} ms^{-1}) with corresponding low groundwater flows.

Sites with this geology are however rare due to urbanisation extending over topographically favourable terrain. Large scale instability can also be a potential constraint with Waitemata Group materials. Prebble (1992) defined the Southern Landslide Zone, an extensive area of deep seated land movement in the South Auckland area. The Manukau Landfill site proposed in 1991 was located within this zone and required very extensive investigations to ensure stability. This site is presently unpermitted.

A number of landfills in the Auckland area have been located on the Auckland Volcanics. Such sites provide a lesser degree of hydrological security compared to Waitemata Group sites due to moderate hydraulic conductivity conditions. Landfill construction materials are also very limited and would normally have to be imported as is the case for the currently operating Greenmount Landfill.

ii) Bay of Plenty Region

Geology in the region is dominated by ignimbrites and tephra derived soils from the

Taupo Volcanic Zone. These materials provide serious constraints to landfill development in terms of; highly erodible and dispersive soils, difficult workability of allophanic soils, lack of suitable materials for liner and capping construction and poor hydrogeological security with complex ground and groundwater conditions which can be characterised by high hydraulic conductivity.

Suitable sites in the Bay of Plenty Region are restricted to isolated areas of basement greywacke, Pleistocene soft rocks and Pliocene volcanics.

Basement greywacke is generally limited to the eastern margins of the Bay of Plenty. Sites in such areas are not however common primarily due to unfavourable topography. Suitable landfill construction materials, particularly for a clay liner are limited due to a poorly developed and fines deficient residual soil profile. Weathered greywacke can also provide moderate hydraulic conductivity conditions.

Within the eastern Bay of Plenty area Pleistocene soft rocks have been targeted for landfill sites. Preliminary drilling has indicated that these sites have potentially favourable natural containment and on-site construction materials.

For the western Bay of Plenty area a site underlain by Pliocene andesitic volcanics is currently under detailed investigation following a regional site selection study. The site is weathered to 25m depth providing low hydraulic conductivity conditions. Reasonable landfill construction materials are present on-site although depending upon liner option adopted, some materials may need to be imported.

Seismicity is particularly an issue in the eastern Bay of Plenty area with the presence of the Taupo Volcanic Zone. Potential fault rupture was specifically addressed for all short-listed sites in this region.

iii) Manawatu Region

Large areas of the Manawatu District are underlain by Holocene alluvium which is characterised by moderate to high hydraulic conductivity offering poor natural containment.

Also on the extensive lowlands are Pleistocene terraces capped by 2m to 10m of loess with low hydraulic conductivity. The loess could provide suitable natural containment and liner materials and investigations are currently in process in such areas.

The remaining geological units in the district are the basement greywacke and the Pliocene soft rocks. Sites of this geology are not common due to unfavourable topography. Of the two units the Pliocene soft rocks generally provide more favourable geotechnical conditions due to potentially good containment and construction materials.

iv) Summary

For site selection studies undertaken, geotechnically "perfect" sites are rare. Unlined sites are generally not an option due to hydrogeological uncertainties. Site suitability is considerably more restricted for lined sites due to the demands made by the engineered containment systems.

Many older landfills are present in the regions studied. A number of these are located in areas considered to be of unfavourable geology. Due to the limited number of favourable sites available and high cost of investigations of new sites, performance monitoring of old sites should be evaluated in terms of current site selection criteria.

EXISTING SITE PERFORMANCE - ARE OUR SITE SELECTION CRITERIA VALID?

Table 1 provides a very basic comparative evaluation of a number of existing sites in the North Island. Leachate containment, gas controls and monitoring systems have been rated on a scale of 1 to 3 where 1 meets the CAE guidelines and 3 indicates no system at all. None of these sites achieves a total score of 4, which would indicate full compliance with the CAE guidelines. The older sites had no engineered controls and contaminants were simply discharged into the natural environment. While there is no doubt that these older sites degraded the receiving environment, 10 or so years after closure there is little evidence of a "toxic time-bomb" threatening to engulf suburbia as we know it.

A good example of this is Craigs Quarry, where relatively low levels of contaminants were identified only after specific attempts were made to place a borehole into the landfill itself.

In some instances, contamination downstream of a landfill site has been attributed to road run-off and general atmosphere contamination, with no direct evidence of a landfill related source.

The current trend towards total management of the waste stream from source to final disposal should effectively reduce the levels of difficult, controlled and hazardous wastes entering the landfills. A significant reduction in leachate strength in terms of hazardous elements should come about as a result of the widespread use of the "pretreatment" option to comply with leachability criteria.

In essence, the new landfills will receive a less toxic mix and will be better managed with full containment systems and effective treatment or disposal methods for leachate and gas. Is this standard of engineering an overeaction to poor management practices in the past? Can we justify and afford the level of investigation, containment and monitoring currently being imposed under the Resource Management Act and reinforced by the CAE guidelines? Should we not perhaps be looking more closely at the effects of the existing landfills and perhaps extending the life of these landfills with tighter controls over the nuisance factors?

Are we simply adopting hazardous waste containment technology from overseas and applying this to municipal solid waste in New Zealand which contains a very small fraction of industrial waste?

As more monitoring data becomes available, we will be able to define more accurately the real effects of landfills. These effects on the immediate site environment should be the driving force for the consent process, not "guideline" standards promulgated by the USEPA, CAE and others.

FUTURE TRENDS

What will the landfills of the future be like?

- Landfills will still be a necessary commodity but there will be fierce competition for landfills to be sited in urban areas to clean up old landscape scars and provide a desirable recreational area on completion.

- Nuisance factors will no longer be on issue as a result of effective screening, dust and odour controls and new management practices.

- Old sites will be reworked and compacted, using the degraded refuse as cover, thus increasing the air space of closed sites.

- New sites will be biodegraded quickly and efficiently and the degraded refuse reworked again before final closure.

- Research into clogging, diffusion and gas transfer effects will have redefined containment practices. The new sites will incorporate an attenuation layer, self clogging liner and a diffusion barrier.

- Monitoring will be limited to nuisances (dust, noise, odour) and key contaminants only.

CONCLUSIONS

Complying with all aspects of the CAE guidelines is proving to be cumbersome. A limited amount of high quality monitoring data from existing sites does not support the common belief of major contamination and leakage from these sites. A rational approach based on an evaluation of site specific effects from existing sites in a region may show that full compliance is not necessary and that effects can still be well managed without a "Rolls-Royce" system.

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TABLE 1

COMPARATIVE EVALUATION OF EXISTING LANDFILL SITES

Site	Volume Placed m^3	Years of Operation	Natural Containment	Liner	Leachate System	Gas Control	Monitoring	Total Score	Comment on Effects
Kay Road Waitakere City	500,000	1984 to 1993	Good, Waitemata Group groundwater discharge zone	3	2	2	1	8	Minor leachate seeps in stormwater
Barrys Point North Shore	600,000	1930 to 1980	Poor, volcanics and tidal flats	3	3	3	2	11	Passive gas system installed 1990's to protect buildings on site
Craigs Quarry Auckland	?	1960's and 1970's?	Poor, volcanic aquifer	3	3	3	3	12	Can detect leachate contamination in groundwater within 200m of site
Greenmount Auckland City	3 million	1979 to 1994	Poor, volcanic aquifer	2	2	1	1	6	Gas transfer effect is only detectable contamination
Pikes Point Auckland	1.5 million	1974 to 1984	Poor, volcanic aquifer and tidal flats	3	3	3	2	11	Leachate system reinstated 10 years after closure to prevent small seeps along foreshore
Papakura	70,000	1982-1994	Average, Waitemata Group	3	3	3	1	10	No significant groundwater contamination
Gisborne Paokahu	600,000	1971 - 1994	Poor, estuarine silts overlying sands	3	3	3	2	11	No detectable leachate 50m clear of site in brackish, flood plain environment
Awapuni	2 million	1950's - 1994	Poor, river gravels	3	3	3	2	11	Contamination of shallow aquifer under landfill but no contamination detected in waters of river adjacent to landfill

1 - System general meets CAE guidelines
2. - System has been engineered and is well controlled
3. - No system installed or very basic system

REDVALE LANDFILL - GENERAL DESIGN AND CONSTRUCTION CONSIDERATIONS

A. P. Kortegast
Director
Tonkin and Taylor Ltd
Auckland

SYNOPSIS

The Redvale Landfill is the first privately developed landfill in New Zealand to meet accepted USEPA design standards for operation as a Municipal Solid Waste Landfill. At the time the design was undertaken USEPA Subtitle D criteria were being developed in the USA and the landfill design concept was based on expected Subtitle D performance criteria. The final design meets the Subtitle D criteria promulgated in October 1991. The design maximises the advantage of a site geology of low permeability mudstones and limestones by utilising these materials to achieve a very low liner permeability. However, these highly sheared Lower Tertiary age rocks demonstrate poor strength characteristics and require careful engineering and materials control in developing the site. The landfill represents "state-of-the-art" in New Zealand and the design basis forms an appropriate yardstick for assessing future landfill proposals.

1. BACKGROUND

In June 1990 when Waste Management N.Z. Ltd (WMNZ) advanced consent applications for the Redvale Landfill, it was the first privately owned company in New Zealand to seek water right and planning permission for a landfill. In approaching the project WMNZ looked to the experience of parent company Waste Management International (WMI), in order to develop an appropriate design given the absence of a clear regulatory direction or set of design standards in New Zealand.

Tonkin & Taylor Ltd was engaged to provide local design expertise and consent process support and in particular, specialist geotechnical and environmental advice. This paper summarises the key design and construction issues associated with development of the Redvale Landfill and focuses in particular on the issue of appropriate design standards for Municipal Solid Waste Landfills (MSWL's), resulting from experience with the Redvale project.

2. SETTING

The site location is shown on Figure 1. The Redvale site had been considered as potentially suitable for a landfill for a number of years, with earlier investigation of the site by the then ARA, in 1984. WMNZ purchased the property in 1988 and following assessment of overall design concepts by WMI, further land was purchased (the East Property), to result in an overall site area of some 85 ha. Inclusion of the East Property was necessary in order to be able to develop a suitable final landfill form and to provide the total airspace required to ensure economic viability. General site features as shown on Figure 2.

It was recognised from the outset that the site had a number of positive attributes with regard to its development as a landfill site, namely:

- relative isolation from housing, but reasonable proximity to the waste source
- a geology which consists almost entirely of low permeability mudstones and muddy (marly) limestones of the Northland Allochthon (Lower Tertiary age)
- minimal groundwater circulation and no local groundwater utilisation
- an inward gradient to any excavation as a result of low soil and rock permeabilities and high groundwater levels
- good access directly off State Highway 1
- a history of quarrying and heavy vehicle operations on the site

On the negative side, the geotechnical properties of the Northland Allochthon materials were known to be poor, with shear strengths characterised by low friction angles, and poor slaking resistance being typical. The Pleistocene clays and regolith soils overlying the mudstones were also known to be highly plastic and generally well wet of optimum water content.

As with siting for all such locally undesirable land uses (LULU's), the proposal met opposition from local residents. This was eventually resolved through the consent process, and is not discussed further in this paper.

The site geology is complex and its tectonic history has resulted in significant transportation resulting in severe stresses and a great amount of deformation in the Tertiary age rocks. In short, overturning movements have resulted in emplacement of a pod of highly disturbed Northland Allochthon rocks amongst and over younger Waitemata Group Sandstones and Siltstones. The eroded Northland Allochthon surface was later blanketed by Pleistocene and Quaternary alluvium. General site geology is shown on Figures 3 and 4. It is notable how the eroded nappe structure results in a 'pod' of low permeability, Lower Tertiary age rocks surrounding the landfill cell and at least 100 m deep in most places.

3. DESIGN BASIS

3.1 Regulatory Background

At the time design was commenced there was no regulatory basis or accepted standard for landfill design in New Zealand.

That situation still exists as the now much referenced CAE guidelines stop well short of forming a definitive engineering design basis.

Critical factors assessed by the Redvale design team in terms of the design basis for the facility were:

- the waste types the facility was to be designed to accept
- overseas precedents and in particular the regulatory framework which had been developing in the USA and Europe during the late 1980's
- the issue of engineering the site in a cost-effective way, taking advantage of the low permeability materials present to form a disposal "cell".

At the time the design was developed, hazardous waste facility engineering requirements had become explicitly defined in the USA under the RCRA laws (Subtitle C), which were promulgated in 1984. These regulations provide the basis for design of Hazardous Waste landfills and by experience of current practice, facilities accepting significant quantities of hazardous waste (i.e. the newer co-disposal sites). In the late 1980's/early 1990's a further set of regulations known as Subtitle D was being developed in the USA for MSWL's. Subsequent trends in Europe have also generally followed the Subtitle D path, which essentially set out to:

- define separate (and less rigorous) standards for MSWL's as against hazardous waste facilities using a performance-based, rather than necessarily prescriptive approach
- promote the discontinuation of co-disposal as a waste management practice
- provide for a combination of engineering and site management standards to control the full range of potential environmental impacts which can arise from landfill activities

The Subtitle D regulations grew out of the perceived success of the more stringent Subtitle C regulations. Part of the difficulty faced in the USA was the sheer number of landfills dealing predominantly with Municipal Solid Waste and their wide variability in terms of engineering design standards and environmental controls. Subtitle D was thus intended to 'bridge-the-gap' between regulators and owners, with requirements for progressive improvement of existing facilities and performance criteria for new ones. This resulted in inclusion of a timetable for effective dates of compliance, shown as Figure 5.

The regulations had a lead time of 24 months as can be seen from Fig. 5, with full power originally to be given to the legislation after October 9, 1993. A further interesting feature of the regulations is that States can seek to "customise" the regulations to meet the local situations. This could, for example, result in a blanket ban on the performance assessment approach and simply require the double liner for all landfills in areas where groundwater usage from shallow aquifers is high.

The Subtitle D Regulations have now been passed into law in the USA. These regulations are also appropriate to New Zealand. In the opinion of the author they, or something similar, should be adopted for use in a regulatory way in this country to ensure a consistency of design approach.

The Subtitle D regulations are summarised as follows (after Carson; 1993):

Subpart A - General Provisions

The intention of the regulation:
"... is to establish minimum national criteria for municipal solid waste landfills (MSWL's), including

MSWLF's used for sludge disposal and disposal of non-hazardous [municipal solid waste] combustion (MWC) ash (whether the ash is co-disposed or disposed of in an ash monofill). Part 258 sets forth minimum national criteria for the location, design, operation, cleanup, and closure of MSWLF units. (Federal Register, 1991)."

Subpart B - Location Restrictions

Location restrictions are as summarised below:

- new, expanding and existing landfills must be $\geq$ 10,000 feet (3,000 m) from airport runways that are utilised by jet aircraft, or $\geq$ 5,000 feet (1,500 m) from airport runways used by piston/propeller aircraft unless it can be proven that a hazard from birds does not exist. (Any new MSWLF or expansion within 5 miles (8 km) from any airport must notify Federal Aviation Administration (FAA).)
- new, expanding and existing MSWLF's must not be located in 100 year floodplains, nor can they interfere with the 100 year flood event or pose a threat to human health and the environment.
- new units and expansions must not be placed in wetlands unless proven (acceptable) via demonstration (to the state).
- new units and expansions must not be sited within 200 feet (60 m) of a fault that has experienced displacement within the Holocene Epoch, unless it can be demonstrated (acceptable) (to the state).
- new units and expansions must not be placed in seismic impact zones which are defined as lithified areas having $\geq$ 10% possibility of a maximum horizontal acceleration of 0.10g [where g refers to the natural gravitational acceleration of the earth] in 250 years, unless otherwise demonstrated (acceptable) (to the state).
- new, expanding and existing units must not be placed in unstable areas, which are defined as areas of potential landslide. All components of the facility must remain intact and be protective of human health and the environment.

Subpart C - Operating Criteria

Existing, new MSWLFs and expansions must:

- exclude the receipt of regulated hazardous waste
- provide daily cover soil or equivalent
- control potential on-site disease vectors
- control explosive gases such as methane
- eliminate open burning of waste (with some exceptions)
- control public access to the site
- provide run-on and run-off controls
- control discharges of surface waters
- cease disposal of liquid wastes with some exceptions
- demonstrate compliance through record keeping

Subpart D - Design Criteria

Design criteria are applicable only to new MSWLF's and expansions. There are two basic design options and the design is a function of (the state's) approval status:

1) (in approved states), a site-specific design standard utilising a point-of-compliance pollutant criteria, or
2) (in unapproved states), a uniform design must be used

The options are indicated graphically in Figure 6. In site-specific designs (in approved states), the liner and leachate collection system must ensure that a list of 24 organic and inorganic constituents, as described in the Federal Register (1991) in the uppermost aquifer do not exceed maximum contaminant levels (MCLs) as designated in The Safe Drinking Water Act at a designated point of compliance that is to be located 0-492 feet (0-150 m) from the facility boundary.

In unapproved states, a standardised composite landfill liner consisting of a geomembrane and compacted clay soil of at least 2 feet, overlain by a leachate collection system.

Subpart E - Groundwater Monitoring and Corrective Action

Monitoring systems must be designed to obtain samples from the uppermost aquifer to characterise background and must be outside the point of compliance. Sampling and analysis standards are included in the rule.

Subpart F - Closure and Post-Closure Care

Closure plans were to be in place by the effective date of the regulation, October 9, 1993. The plans must contain methods to achieve specific closure and post-closure maintenance objectives. Basic landfill closure includes a cover with hydraulic conductivity of no greater than that of the bottom liner, or, no less than 1 x 10^{-5} cm/sec, overlain by topsoil to sustain vegetative plant growth where appropriate. Post-closure monitoring time is 30 years, but this can be altered in approved states.

Subpart G - Financial Assurance Criteria

Owner/operators must demonstrate financial responsibility for closure, post-closure care, and potential corrective action, based on a worst-case scenario. Potential methods to assure financial responsibility are surety bonds, trust funds, guaranteed performance and other credit instruments.

Effective Dates

The USEPA recognised the difficulty in preparing state solid waste plans and delayed the effective date for existing smaller facilities (less than 100 tonnes per day), to April 9, 1994. The effective date for facilities receieving over 100 tonnes per day is now 9 october 1994. For some groundwater monitoring and minimum separation issues, the effective date has not yet been set.

3.3 Redvale Design Basis

Although the Subtitle D regulations were not finalised at the time the Redvale landfill was designed, it was considered that the principles of Subtitle D would be unlikely to change and that fundamentally their logic was sound and applicable to the New Zealand situation. It was therefore decided to design the landfill based on (the likely) Subtitle D regulations and to operate it as MSWL. That is, it would be engineered with a clay liner on a performance basis (given the lack of any local aquifers), and it would not accept 'hazardous waste'.

Out of this background the following design basis was developed for Redvale:

- a liner design based on likely Subtitle D criteria and formed from natural materials
- no nett loss of wetland areas from the site
- application of daily cover
- no burning of wastes at the site
- control of birds and other disease vectors (by minimising open area, applying daily cover etc)
- no public access
- use of effective silt control measures
- acceptance only of MSW and approved special wastes (no hazardous wastes)
- incorporation of a leachate collection and treatment system
- incorporation of a gas control system
- application of final cover along with cover drainage, maintenance and re-vegetation

This design basis has, by a process of attrition/adhesion, now become more accepted (and expected) in New Zealand and is the sort of basis on which most environmental engineering consultants would now approach such a design assignment. However, the requirement for this level of engineering is certainly not consistent across all regulatory bodies in New Zealand as yet.

4.0 SPECIFIC DESIGN ELEMENTS

4.1 Site Configuration

The Redvale landfill is designed for a nett airspace of approximately 14.5 Mm^3.

The design configuration was aimed at developing an overall earthworks balance along with target basegrade slopes of 2% and sideslope liner slopes of 1 vertical : 3 horizontal.

The site is designed to follow a pre-determined phasing plan, developed based on waste input or other (e.g. geotechnical factors).

Final slope designs were carefully assessed by landscape architects during the design phase to ensure a final form which marries with the surrounding landscape. Final cap design was assessed using water balance models, along with experience gained elsewhere and top slopes of from 1:10 to 1:5 were selected, with some areas as flat as 1:20. An 'average' top slope of about 1:6 was targeted.

The final grade and basegrade plans for the site are shown as Figures 7 and 8.

4.2 Liner Design

The liner design is based on testing of site materials including:

- laboratory testing of cores
- laboratory testing of re-moulded Northland Allochthon and Waitemata Group materials

- in-situ testing in boreholes
- in-situ testing in large diameter shafts

The results of the permeability testing programme are shown on Figure 9. From this work and following review by the then Auckland Regional Water Board and the consent process itself, the following liner specification was developed:

(i) A liner constructed of re-compacted clay (or mudstone) with a hydraulic conductivity of $\leq 1 \times 10^{-9}$ m/s and a minimum thickness of 900 mm to be placed in all areas where materials other than Northland Allochthon are exposed in the basegrade.

(ii) In all other areas the Northland Allochthon is to be inspected following excavation. Any suspect areas (i.e. where surface defects exist or the material is loose) are to be sub-excavated to 900 mm depth and replaced as for (i) above.

(iii) In all other basegrade areas the upper 200 mm - 300 mm of material is to be scarified, and replaced with compaction in a minimum of 2 lifts to achieve the permeability standard set out in (i).

(iv) In all areas the permeability of the upper 900 mm of the basegrade/liner shall be $\leq 1 \times 10^{-9}$ m/s.

The Phase 2 cell sideslope showing general liner construction is shown on Figure 10. As can be seen from Fig. 10, maximum benefit is taken from the low permeability of the Northland Allochthon materials, with the liner meeting USEPA Subtitle D criteria on a performance basis.

4.3 Final Cover Layer

The design objectives of the final cover are to:

- exclude excess rainwater from the landfill (i.e. minimise long term leachate generation)
- provide a stable base for a revegetation layer, hence minimising erosion
- control landfill gas migration

At Redvale the final cap configuration will be determined following initial trials, but will generally be of minimum permeability (approx. 1×10^{-9} m/s), and at least 900 mm thick.

In the limited areas where tree planting will take place over the final landfill surface, cap thickness will be increased to around 3 m.

4.4 Environmental Controls

A full leachate extraction system is incorporated in the design along with landfill gas control. However, this paper focuses on geotechnical aspects and these design components are not discussed further. For illustrative purposes a simplified cross-section through the landfill showing the relationship of the main leachate components is shown on Figure 11.

5.0 GEOTECHNICAL CONSIDERATIONS

5.1 Engineering Geology

5.1.1 General

In engineering geology terms the site materials can be divided into the principal units given in Table 5.1.

The following brief summary is based on observations from the excavation, exposure and usage of all material types in the works to date, focusing on the principal areas of work, namely the Phase 1 and 2 cells and the Phase 10 area bund.

5.1.2 Phase 1 Cell Excavation

The dominant material in the early phase excavations, which comprises the bulk of the temporary highwall and the sidewalls, is Northland Allochthon siltstone and mudstone. Along the southern (permanent) highwall and the eastern highwall softer, wet Pleistocene sediments are present, up to about 10 m thick. These softer sediments vary in thickness and generally thin to the north. As a result of the poor strength characteristics of these materials a mudstone buttress was first placed along the southern highwall.

A significant amount of information (in terms of material behaviour) was able to be gathered during excavation of the Phase 1 cell, the lower 10-20 m or so of which is in Northland Allochthon materials. In very general terms three lithotypes are exposed in the Phase 1 excavation. In the southeast corner of the Phase 1 excavation grey, weakly cemented siltstone with continuous wide spaced defects is exposed. While this siltstone has a very blocky nature, it has behaved well in the highwall.

Grey soft weak mudstone outcrops in the middle section of the Phase 1 excavation and is separated from a green mudstone by a prominent fault with a NE to SW trend. This mudstone is finely sheared and crushed giving it a powder-like appearance. The crushing has been so intense that there are now no continuous defects within the rock mass. Hence, it has a massive, homogeneous appearance with soil-like properties. However, the mudstone contains appreciable amounts of smectite, which swells when in contact with water or when stress

TABLE 5.1

PRINCIPAL MATERIAL TYPES AND CHARACTERISTICS

Unit	Description	Defects	Hydraulic Characteristics
LIMESTONE:	MARLY LIMESTONE (Main Type) Greenish or cream-grey-green muddy limestone typically with 63 to 77% Calcium Carbonate SANDY LIMESTONE: Light yellow-grey-green medium-fine grained glauconitic limestone $CaCO_3$ content 64%-70%	Highly shattered. Joints close, tight smooth and generally slickensided. Joints infilled with clay or calcite Highly shattered. Joints are close and tight, but frequently rough.	Low inter-granular porosity and permeability due to fine grained and cemented nature. Secondary porosity from shattering restricted by clay and calcite coatings on joints as well as joint tightness
MUDSTONE:	Cream-green to dark grey-green, occasionally calcareous. Often swelled during drilling (smectite clays)	Highly shattered and sheared. Joints and shear surfaces tight and clay filled. Slickensides common.	Low inter-granular porosity and permeability due to fine grained and compacted nature. Low secondary porosity due to tightness and clay coating on joints.
PLEISTOCENE:	Very soft-to-firm clays, silts, silty clays, sandy silts and peats, with gum and wood fragments. Bedding subhorizontal wavy and frequently indistinct.	No joints. Plastic to non-plastic in relation to clay and silt contents.	Low inter-granular porosity and permeability due to fine nature and poor sorting.
RECENT SEDIMENTS	Quaternary age clays and silts comprising slopewash colluvium and organic soils.	No joints. Very plastic to non-plastic.	Generally low permeability.
WAITEMATA GROUP:	Thick basal conglomerate comprising limestone clasts with mudstone fragments in a calcareous mudstone matrix. Contains beds of sandstones.	Widely to closely spaced joints. Joint openness and transmissivity vary, but joints are generally tight.	Low inter-granular porosity and permeability due to fine controlling grainsizes present. Defects control permeability.

relieved. In the present highwall the grey mudstone performs reasonably well and, where it is on its own, slopes gradually regress by fretting to slope angles of 20° to 25°, apparently without the development of major areas of deep seated instability.

Experience with these mudstone types at Redvale and with similar materials in the Waikato Coal mines has now led to development of an excavation methodology whereby the mudstones exposed in the temporary highwall (now up to 20m high) have an overall face angle of about 40°. Other than some localised slope failures, this approach appears to be working well by limiting the depth of softening resulting from stress relief.

5.1.3 Phase 10 Bund

The Bund 10 footprint is located along the northern boundary of the West Property (refer Fig. 2), and extends from a ridge formed by Waitemata Group conglomerates in the west through a low area to a ridge of mudstone at the eastern end. Residual soils derived from the underlying basement are developed at either end of the bund footprint with soft Recent sediments and Pleistocene materials. The thickness of these largely "unconsolidated" sediments ranges from 5 to 7 m.

Foundation strength characteristics vary widely along the footprint, with the softer materials occurring between the western and central sections. Stripping of these softer materials was necessary to form a toe key and otherwise prepare the foundation. Investigation indicated that deep excavation was required to remove organic clays from the central section, with a substantial toe key necessary to provide adequate lateral stability for ultimate landfill development, as shown on Figure 14.

Supplementary test pit investigations in the central (Stage 2) area indicated that significant parts of this area were underlain by pervasively sheared mudstones. Clear evidence of lateral displacement of the soil profile was observed in the pits, with the movement appearing to have occurred on a very thin, but relatively continuous shear zone near the mudstone interface with the overlying Recent sediments. It is noteworthy that these shear zones are difficult to detect and may only be a few millimetres thick, yet can represent areas of quite exclusive instability. A sample of gouge material retrieved from this shear zone was tested in a ring shear apparatus. The results indicated a very low peak and residual friction angle of 9° and effectively zero cohesion. The presence of such a weak residual failure feature was adopted as a controlling factor in the design of the central and western bund areas.

At the eastern end of the footprint, stiff to firm silts and clays form a thin mantle over the mudstone basement and generally only minor stripping was required to prepare the foundation.

5.2 Geotechnical Engineering Properties

The strength properties of the site materials vary widely and are extremely sensitive to water content, drying and re-wetting cycles, and handling methods. The mantle of Pleistocene and residual soils is generally of more variable water content and plasticity than the less weathered, Northland Allochthon materials.

Testing of the re-compacted materials was undertaken to assess their best utilisation in the landfill (all excavated materials have to be utilised and in this case, given the often poor material properties, the question was how best to achieve this).

As all excavated materials are to be used as fill the problem was approached in terms of performance criteria and various fill categories were defined, based on target strength or permeability criteria. Broadly, the wetter, plastic surface clays and silts were utilised for low strength embankment and buttress fills or were stockpiled for later re-working, drying and use as final cover.

The highly sheared mudstone from the Phase I and II areas is suitable for use as high strength structural fill, with re-working resulting in breaking down of the sheared faces and generally resulting in a very high strength, and low permeability fill.

The marly limestone is also used for fill, much like a weak 'rotten rock' material. It is used for semi-granular fill around culverts and as road subbase, but cannot be compacted under field conditions to a permeability as low as that achieved by the mudstone fills.

Of particular note is the range (and low range at that) of strength properties exhibited by many of the site materials. In particular, the presence of pervasively sheared zones within some of the mudstones, often containing layers of shear 'gouge' at close to residual strength was a key issue for design. An assessment had to be made of the true residual strength and the continuity of pre-sheared surfaces as a basis for stability assessments of weak highwalls and embankment structures founded on these materials. Typical strength values for the mudstones are given in Table 5.2 and make interesting reading.

Table 5.2

Measured Mudstone Strength Parameters

Test Type	Description	Results
CUP	Residual mudstone (CLAY)	$c' = 11$ kPa, $\varnothing' = 17°$
CUP	Residual mudstone (CLAY)	$c' = 11$ kPa, $\varnothing' = 15°$
Ring Shear	Shear zone within mudstone (gouge)	$c' = 0$, $\varnothing'_r = 9°$
Direct Shear	Subhorizontal shear zone (calcarcous mudstone) taken intact from shafts	$c' = 28$ kPa, $\varnothing' = 14°$ peak $c' = 10$ kPa, $\varnothing'_r = 12°$ residual
Back Analysis	Basal shear zone, Stage 1 foundations movement zone	$c' = 0$, $\varnothing'_r = 12°$
Ring Shear		$c'_r = 0$, $\varnothing'_r = 8.3°$
Ring Shear		$c'_r = 0$, $\varnothing'_r = 8.1°$
Ring Shear		$c'_r = 0$, $\varnothing'_r = 17°$

Table 5.3 presents compaction data for the upper Pleistocene soils in the Phase 1 cell area

5.3 Geotechnical Design Considerations

Aside from material utilisation the principal design consideration was the excavation and bund founding details necessary to ensure adequate short and long term stability. Some areas are less critical in stability terms (e.g. stockpiles, temporary highways), while others are very critical (permanent sidewalls and basegrade zones, Phase 10 bund).

Stability design requirements are given in Table 5.4.

Table 5.3

Compaction Test Results - Phase I Cell Upper Soils

Sample Depth (m)	Natural Water Content (%)	Optimum Water Content (%)	Shear Strength at Optimum (kPa)
0.3 - 2.6	25.7	21	170 +
3.8 - 4.8	56.9	30	157

Table 5.4

Design Requirements

Case	Strength Parameters	Required FOS
End-of-Construction	Effective (with $r_u = 0.55$)	≥ 1.30
Long-Term (static)	Effective	≥ 1.50
Long-Term (seismic)	Undrained	> 1.0

As an example, the soil strength parameters adopted for assessment of Stages 2 and 3 of the Phase 10 bund are summarised in Table 5.5. These parameters are based on the results of laboratory and field testing and demonstrate the high degree of material variability in this critical construction area.

Table 5.5
Summary of Soil Strength Parameters
- Phase 10 Bund Design

Location	Unit	γ (kN/m^3)	Cu (kPa)	c' (kPa)	Ø' (°)
Fill	Type 1A	20	150	5	35
	Type 3 (or 4)	18	60	5	25
	Existing	18	75	0	25
Foundation	Unit A	18	40	14	15
	Unit B	18	40	10	20
	Unit C	18	40	5	30
	Contact zone*	18	-	0	.12*
	Mudstone	20	200	25	20

* For Stage 2, Ø' for the contact zone was reduced to 9° as a result of ring shear test results on gouge material taken from a continuous shear zone observed in TP43. The in-situi undrained shear strength of this material was notable to be determined.

Stability Analyses

A range of saturation conditions was assumed for the stability analyses. For the end of construction case, an r_u value of 0.55 was assumed to allow for construction pore water pressures. This condition was assumed both in the fill and in the foundation materials. For the long term case, a groundwater level in the upper part of the slope was estimated, governed largely by the final geometry of the landfill pit excavation. Within the bund, the groundwater level was assessed to follow the existing ground surface.

For the seismic stability assessment of the bund undrained shear strength parameters were adopted. This is a conservative approach as it does not take account of the strength gains typically associated with long term confining pressure effects within the soils. This approach was adopted as specialist testing to better define these design parameters was not considered to be warranted given the relatively low risk of seismicity for the site. For the purposes of the seismic analyses a conservative shear strength value of 10 kPa was adopted for the low strength gouge material (present at the failure surface in parts of the Phase 10 bund foundation).

6. CONCLUSION

The main purpose of this paper was to summarise the design basis for the Redvale Landfill, New Zealand's first "state-of-the-art" Municipal Solid Waste disposal facility. A pro-active approach was taken to design by adopting a regulatory and management framework based on overseas experience and ahead of general New Zealand practice at the time.

The scope of the underlying USEPA Subtitle D regulations is discussed and advocated (possibly in slightly modified form), as a general design basis for New Zealand conditions.

The latter part of the paper reviews the challenging soil and rock conditions at the Redvale site and summarises principal aspects of the design approach. The designers have had to cope with marked changes in materials over short distances and with complex structural geology and material properties. Thus, while not an easy site to develop initially, the Redvale site's inherent advantage of predominantly low permeability saturated material types, and the excellent liner properties these soils offer, make it ideally suited to use for a Municipal Solid Waste Landfill facility.

7. ACKNOWLEDGEMENTS

Concept design of the landfill was carried out by Waste Management International Inc, with input from a large New Zealand-based design support team. Final design was carried out by Tonkin & Taylor Ltd. Permission to publish this paper by the site owner, Waste Management N.Z. Ltd is gratefully acknowledged.

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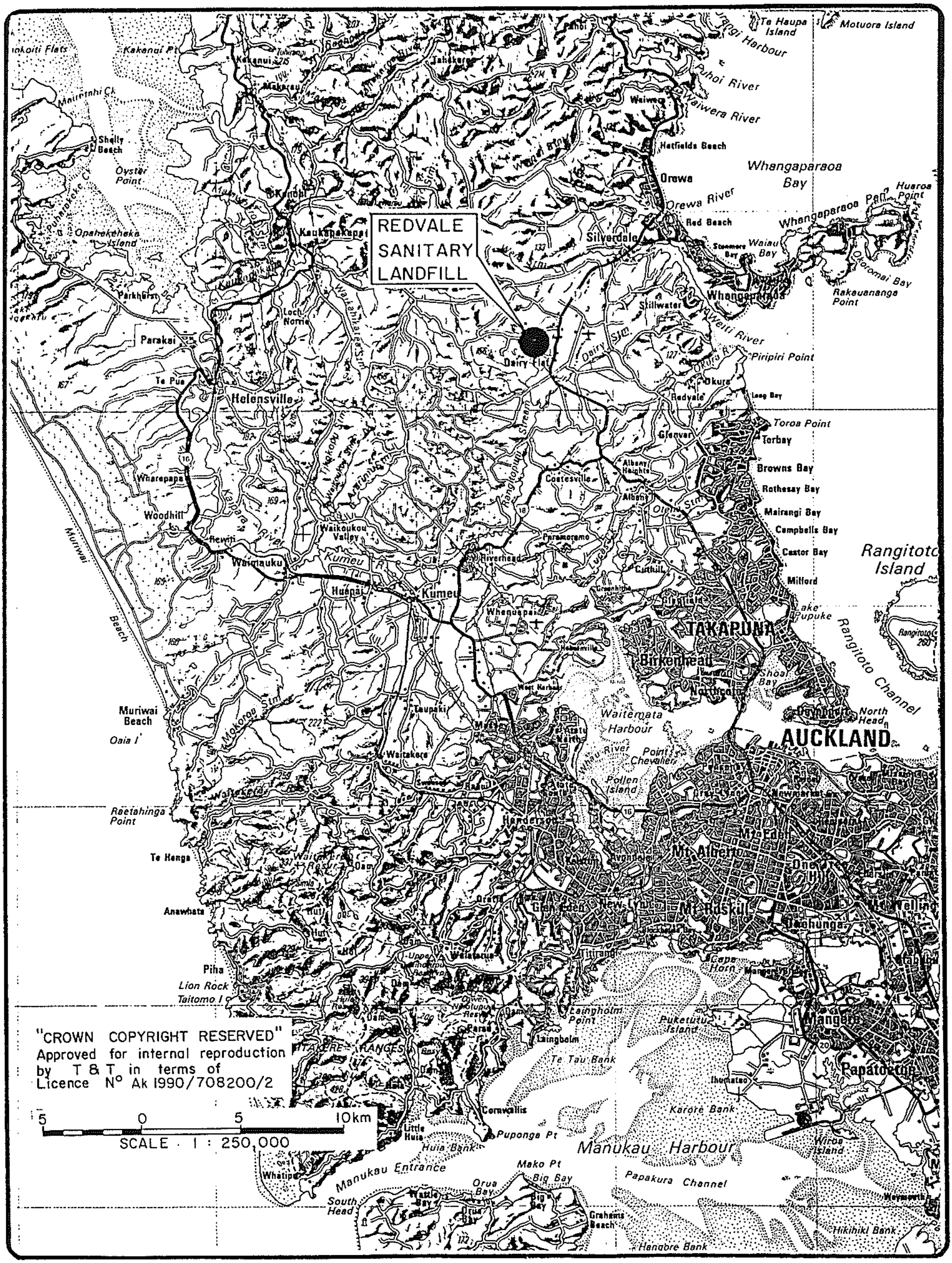


Figure 1
Site Location

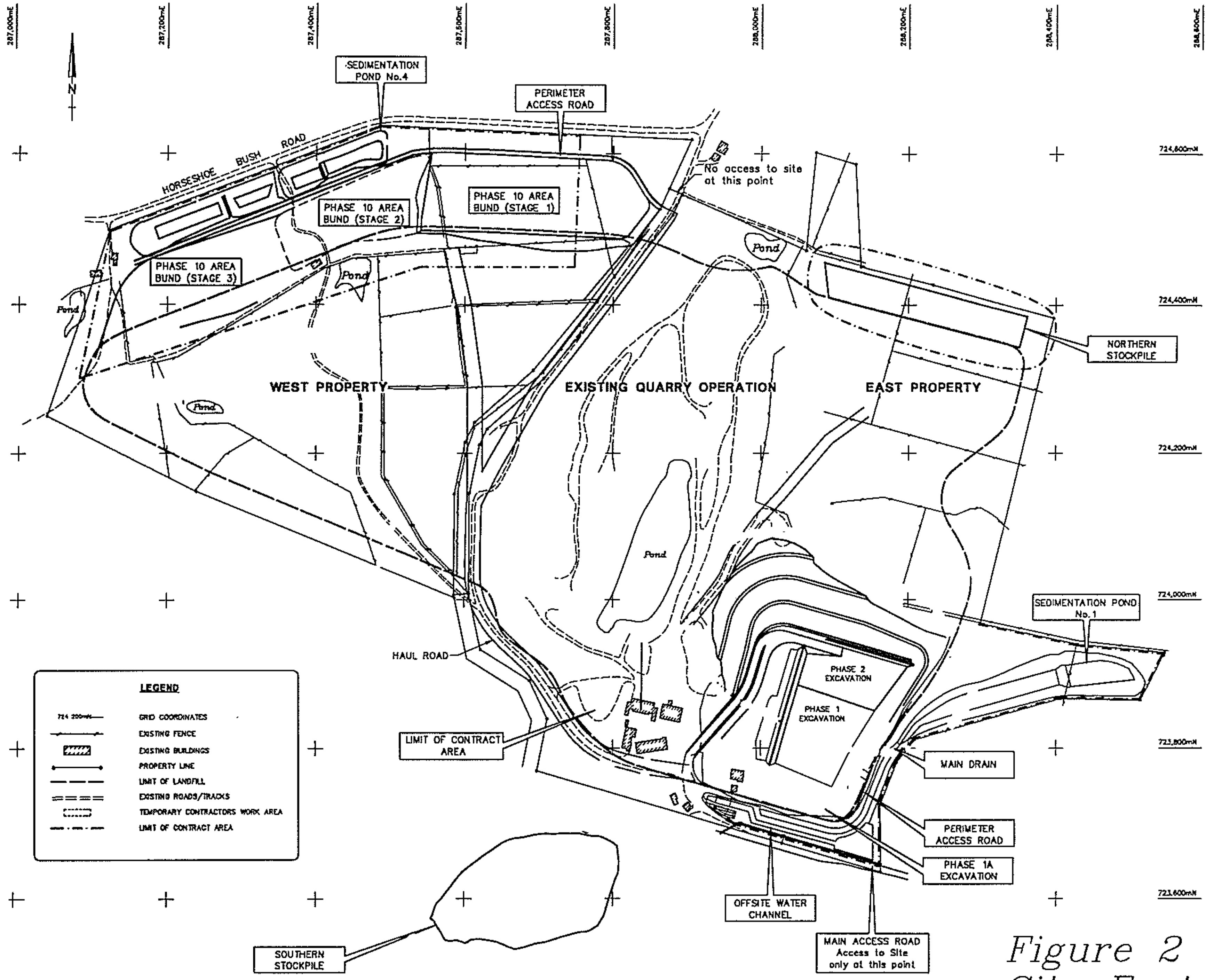


Figure 2
Site Features

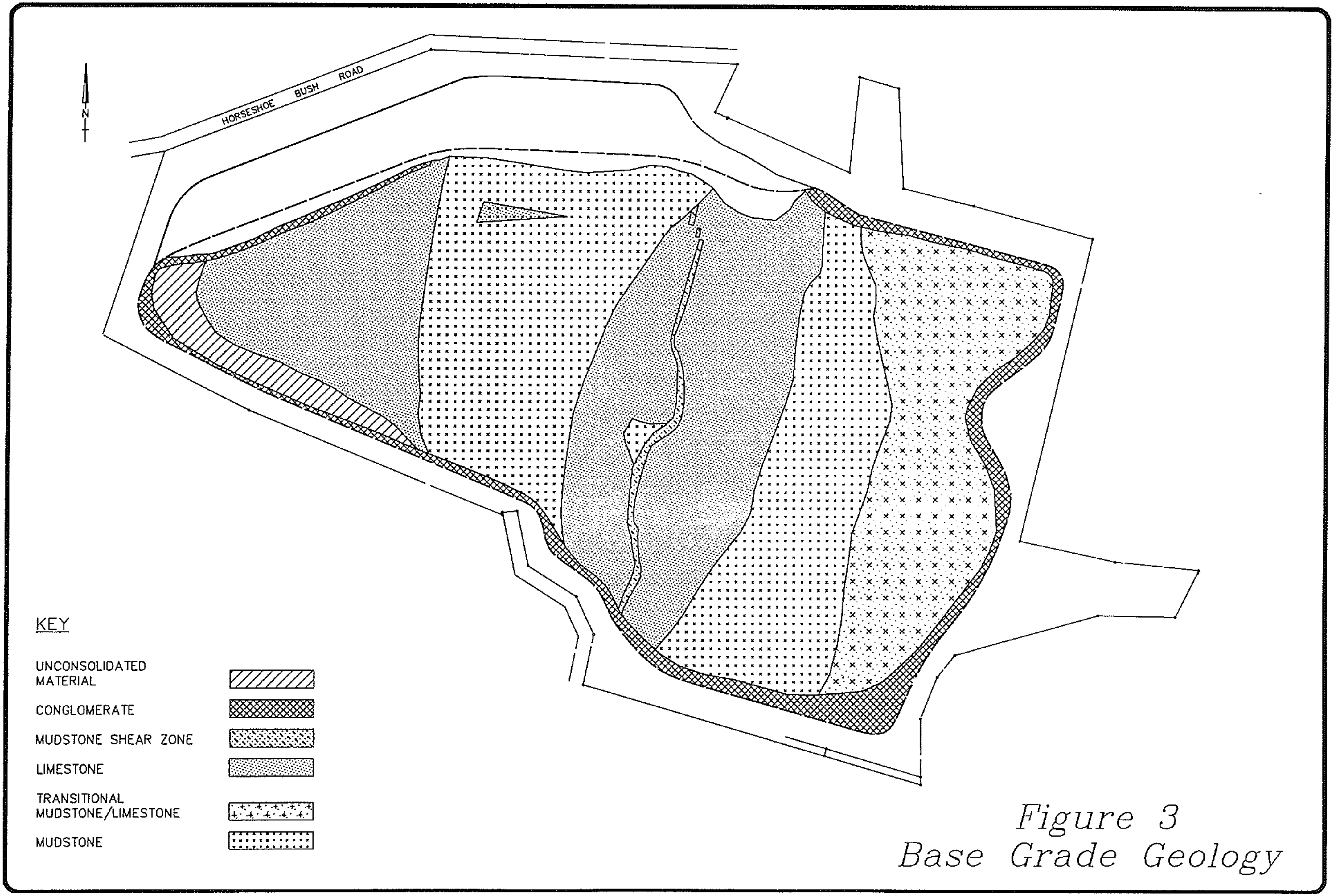


Figure 3
Base Grade Geology

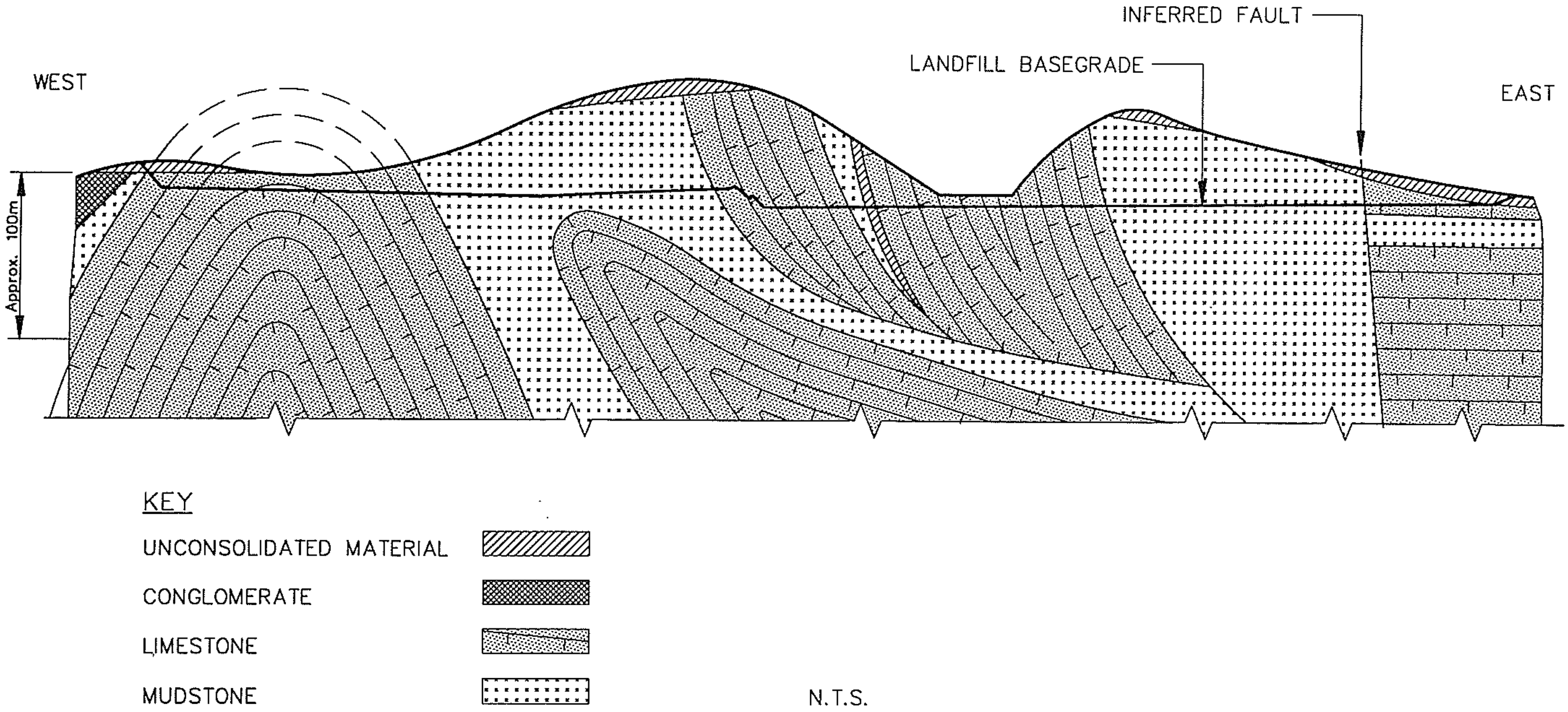


Figure 4
Site Geology

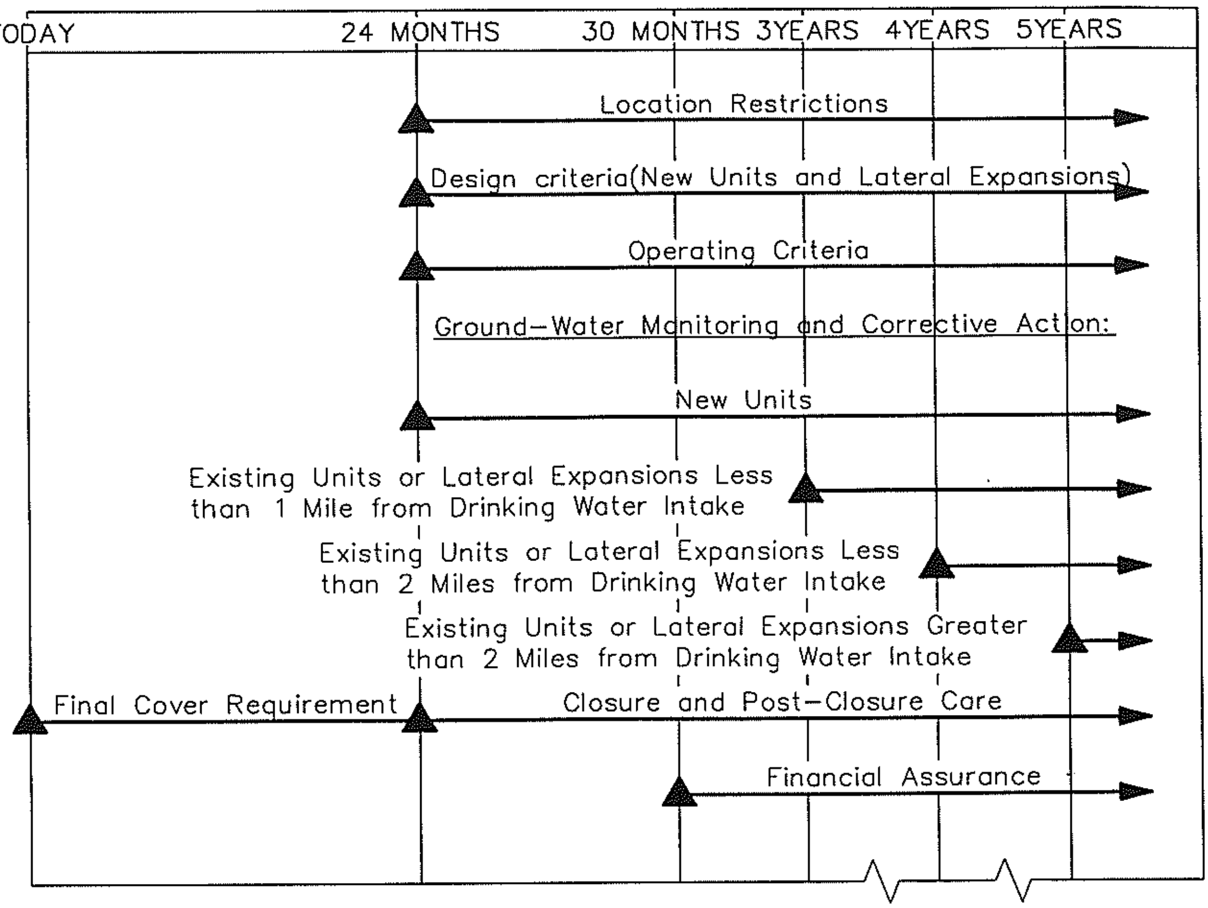


Figure 5 Subtitle D
Effective Dates of Compliance(Federal Register,1991)

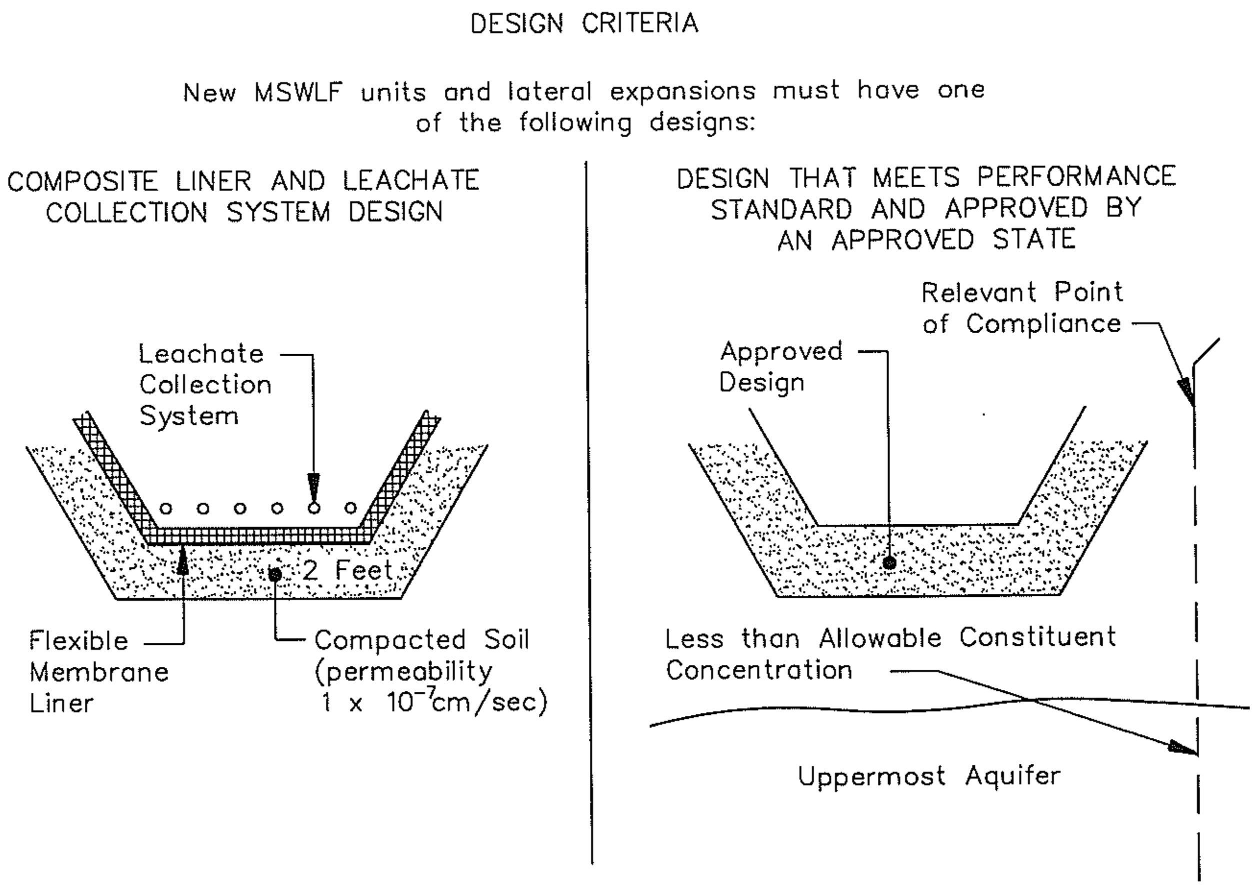


Figure 6 Subtitle D
Effective Dates of Compliance(Federal Register,1991)

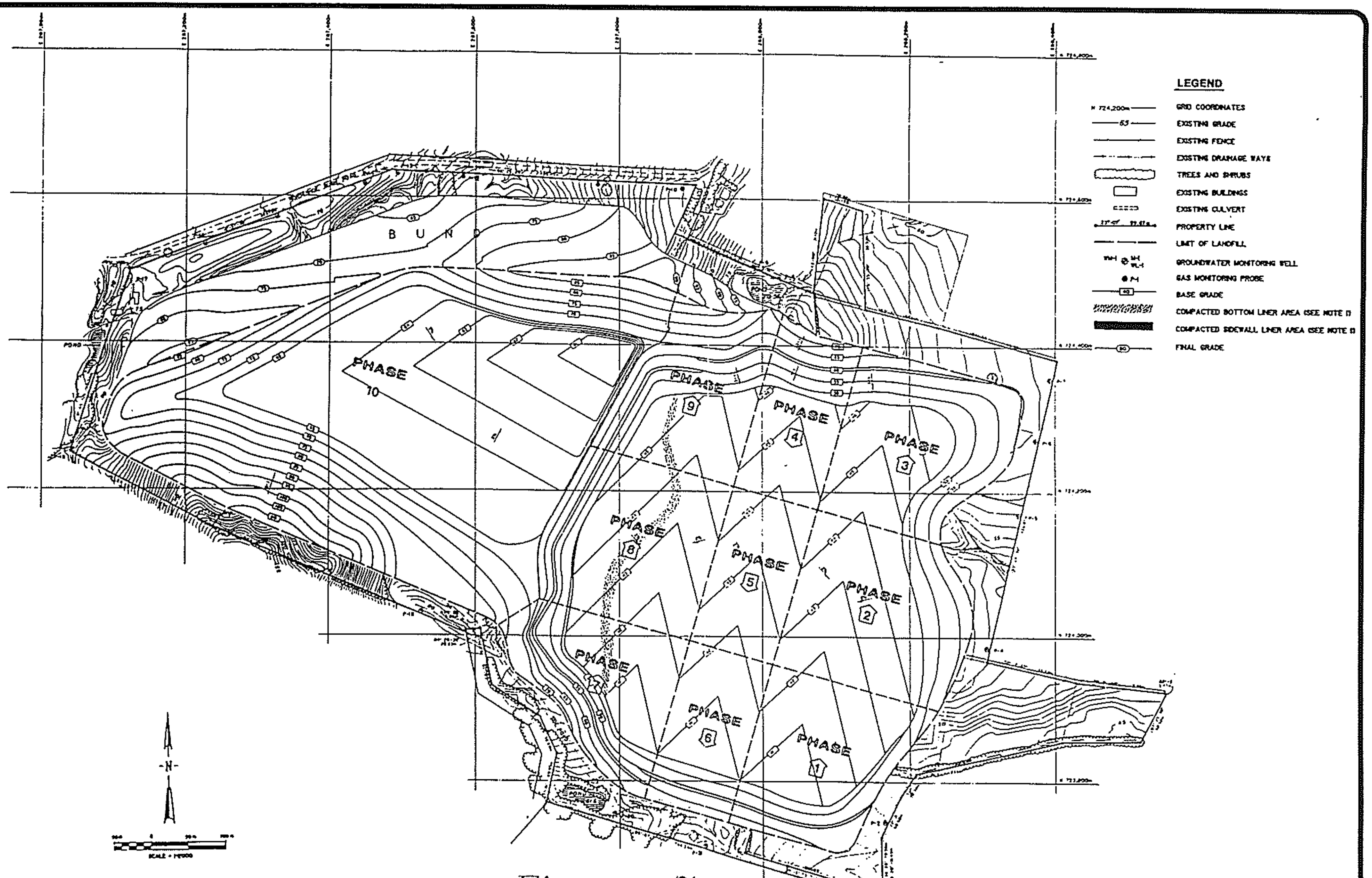


Figure 7
Base Grade and Phasing Plan

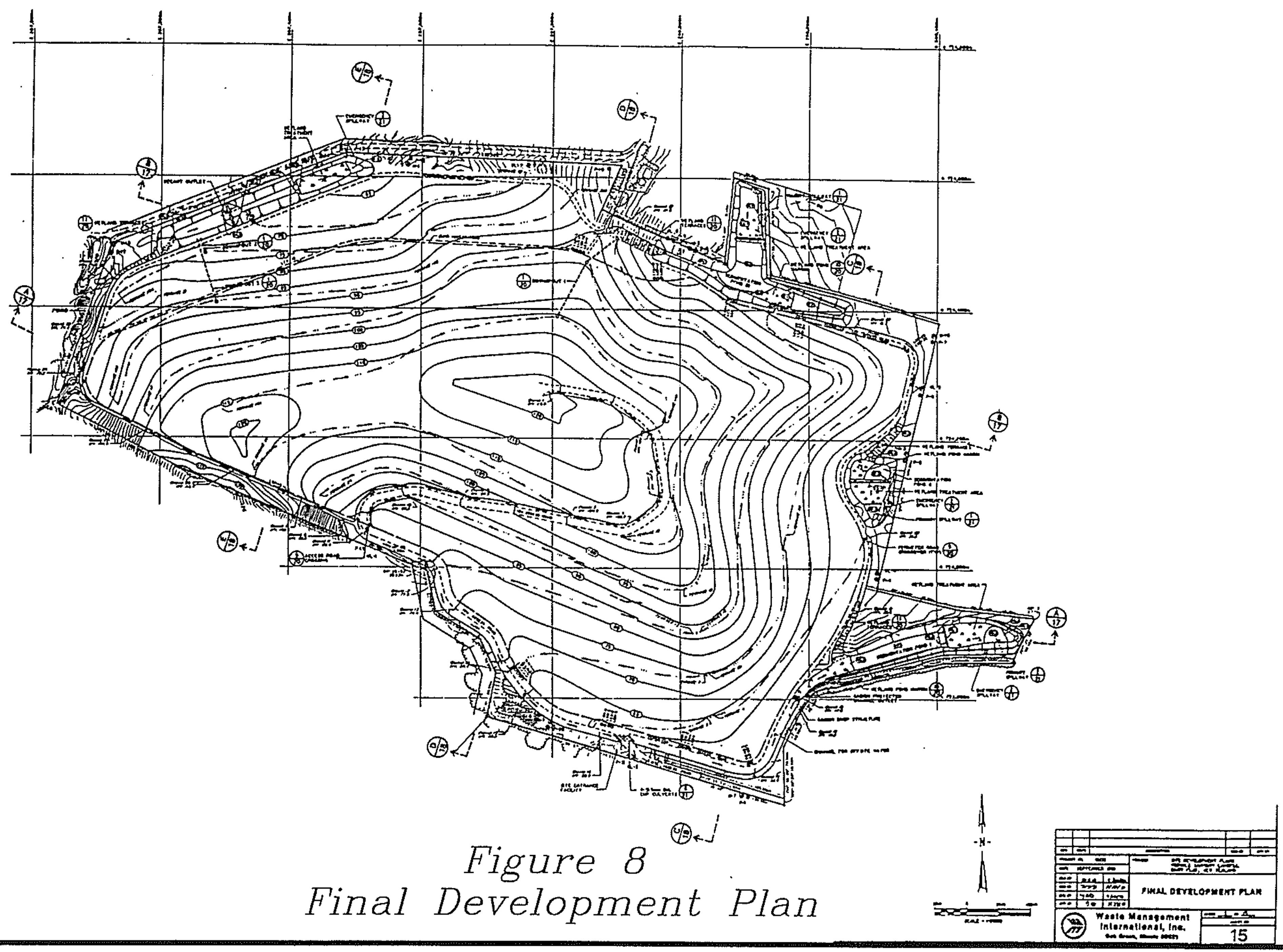


Figure 8
Final Development Plan

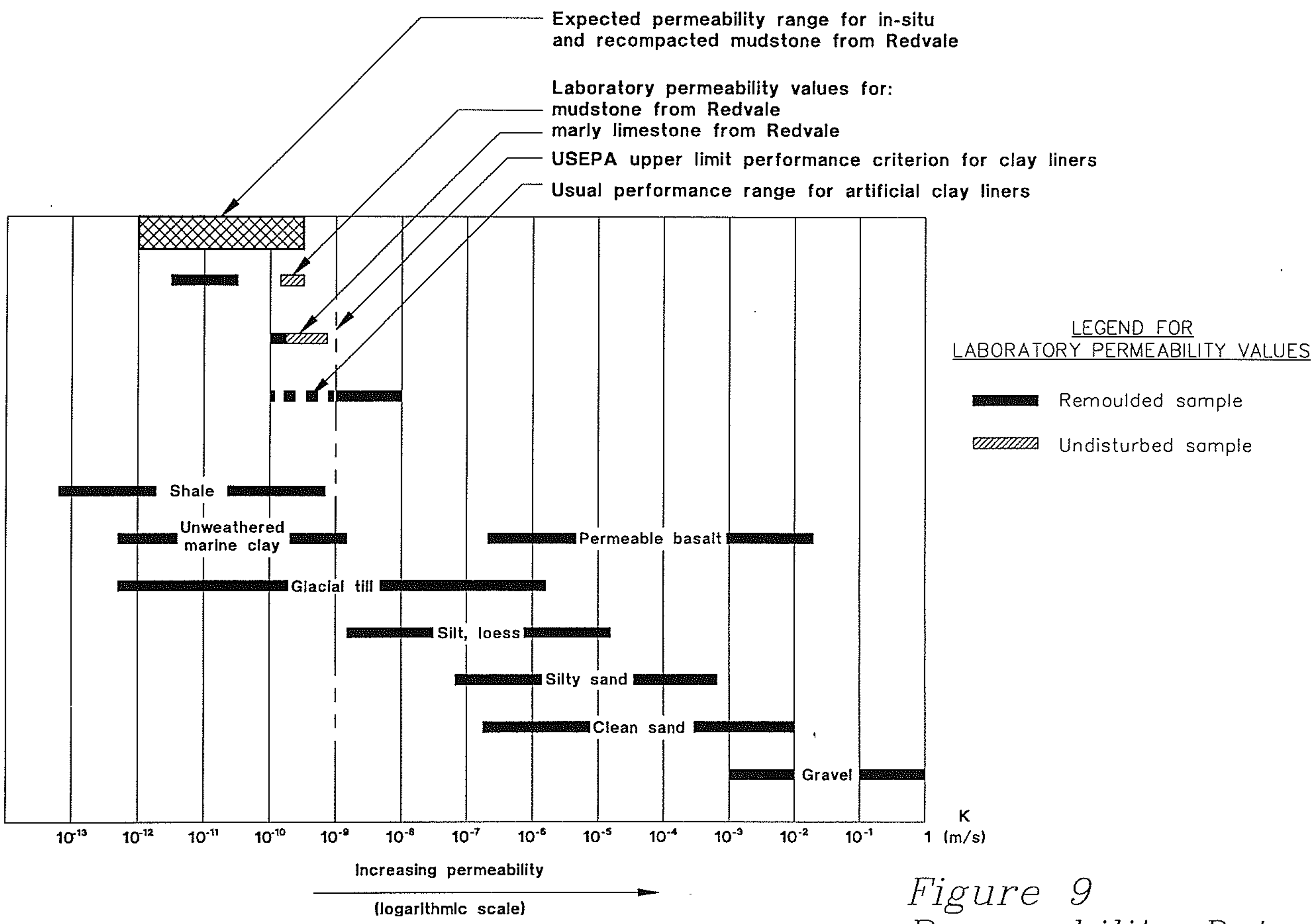
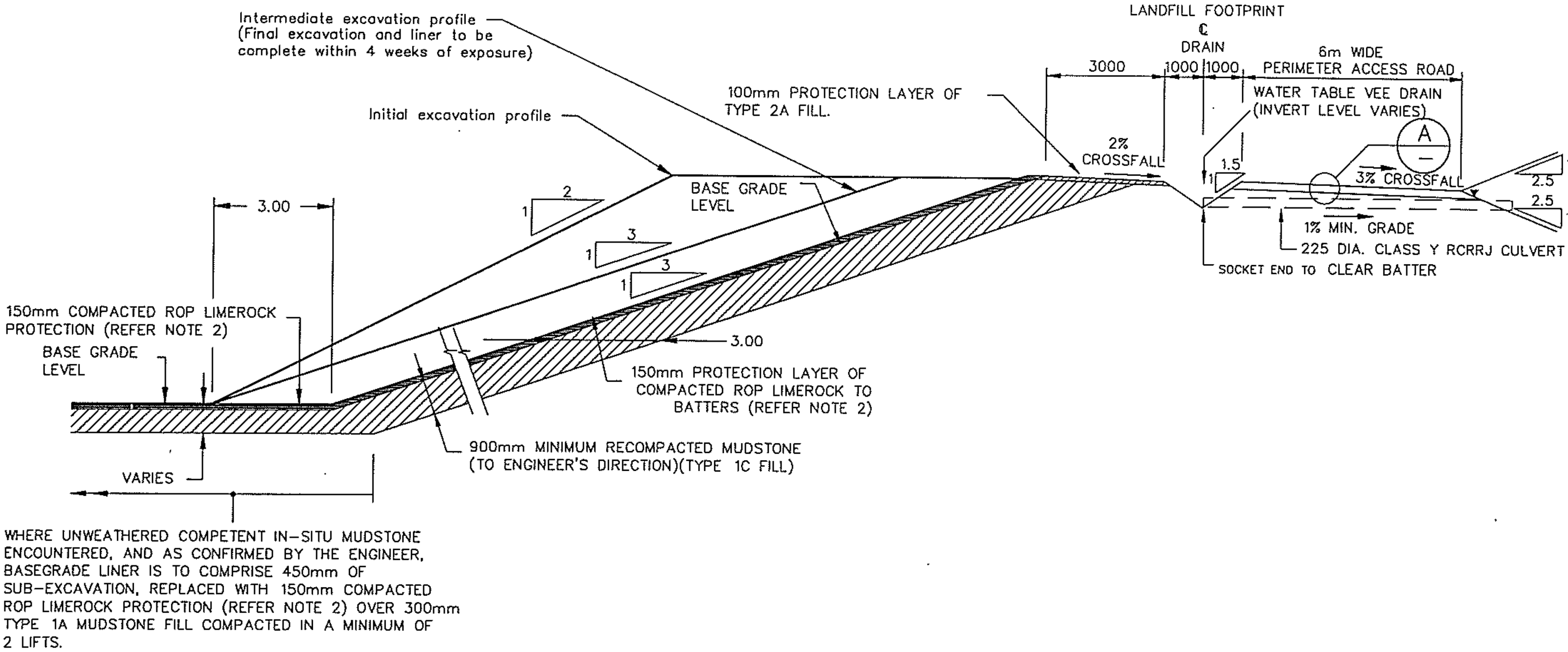


Figure 9
Permeability Data



TYPICAL CROSS-SECTION THROUGH PERIMETER OF CELL WALL, PERIMETER ACCESS ROAD

Figure 10
Typical Liner Detail

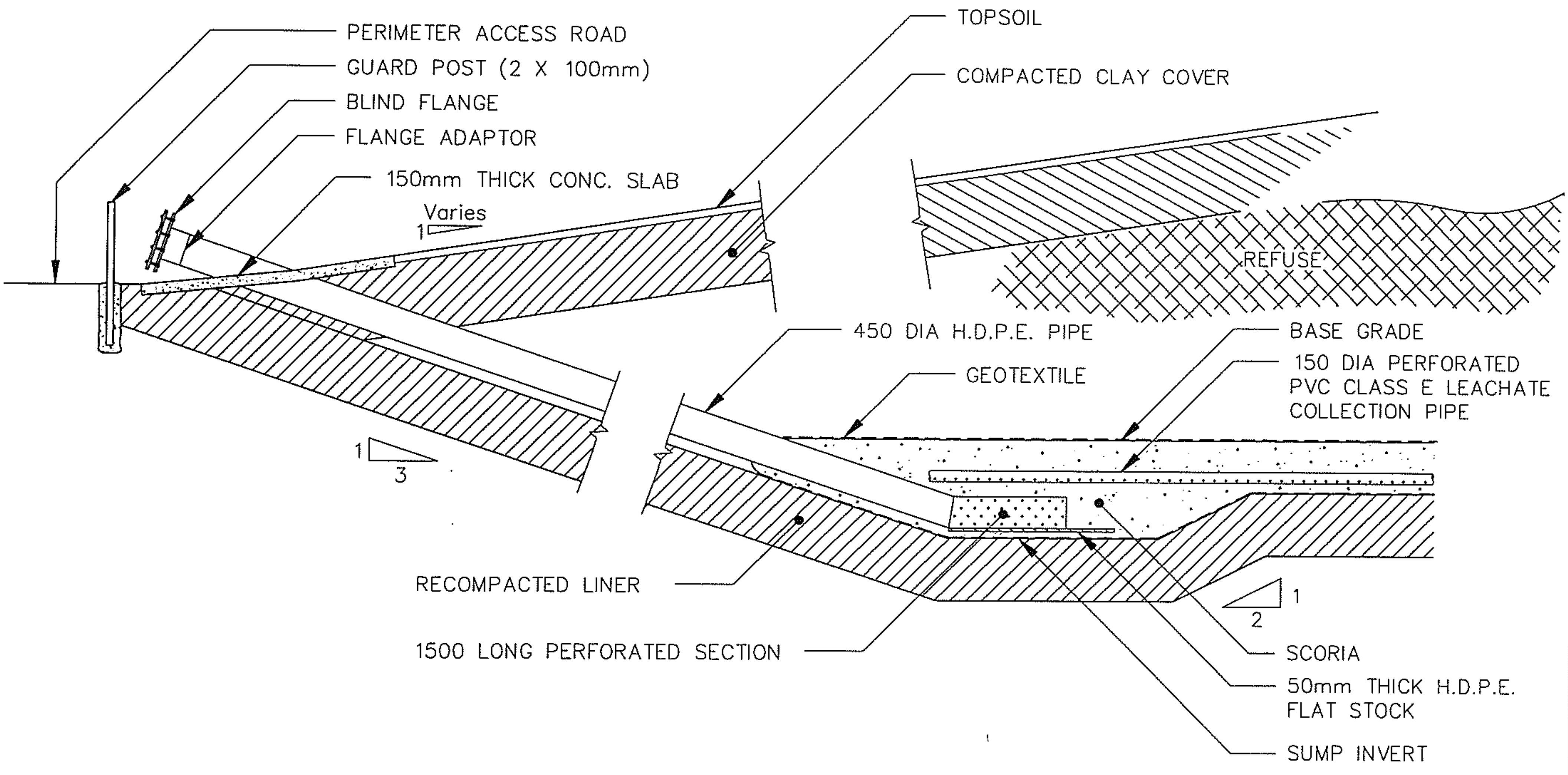


Figure 11
Typical Engineering Detail

GROUP	FORMATION	LITHOLOGIES	THICKNESS	AGE
TAURANGA		. Undifferentiated Alluvium, some consolidated sands. Multicoloured clays and conglomerates	0-10 m+	Pleistocene - Holocene
		UNCONFORMITY		
WAITEMATA	Onerahi	. Sheared green and chocolate mudstones, siliceous claystones, calcareous siltstones, argillaceous limestones, serpentinites (mixed age lithologies U. Cretaceous - Oligocene)	3-170 m	Pliocene - Miocene
		UNCONFORMITY		
	Cape Rodney	. Conglomerates, grits, sandstones volcanic intrusive . Grits, interbedded sandstones and siltstone, thin carbonaceous layers . Interbedded sandstones, siltstones and mudstones, thin carbonaceous layers . Basal beds, conglomerates, breccias, sandstones, mudstones and shell limestones	50 m+ 300 m+ 470 m+	L. Miocene
		UNCONFORMITY		
	Wainui Siltstone	. Grey white calcareous siltstone, rare volcanic grits		L. Miocene
		UNCONFORMITY		(Northland Allochthon Emplacement)
TE KUITI	Mahurangi Limestone	. Green-Grey argillaceous limestone and green sands . Blue-grey siltstone . Green-grey mudstone		U. Eocene Northland Allochthon Strata Mid-U. Eocene
		REGIONAL UNCONFORMITY		
WAIPAPA		Greywacke sandstone and argillite		Jurassic - Permian?

Figure 12
Site Stratigraphy

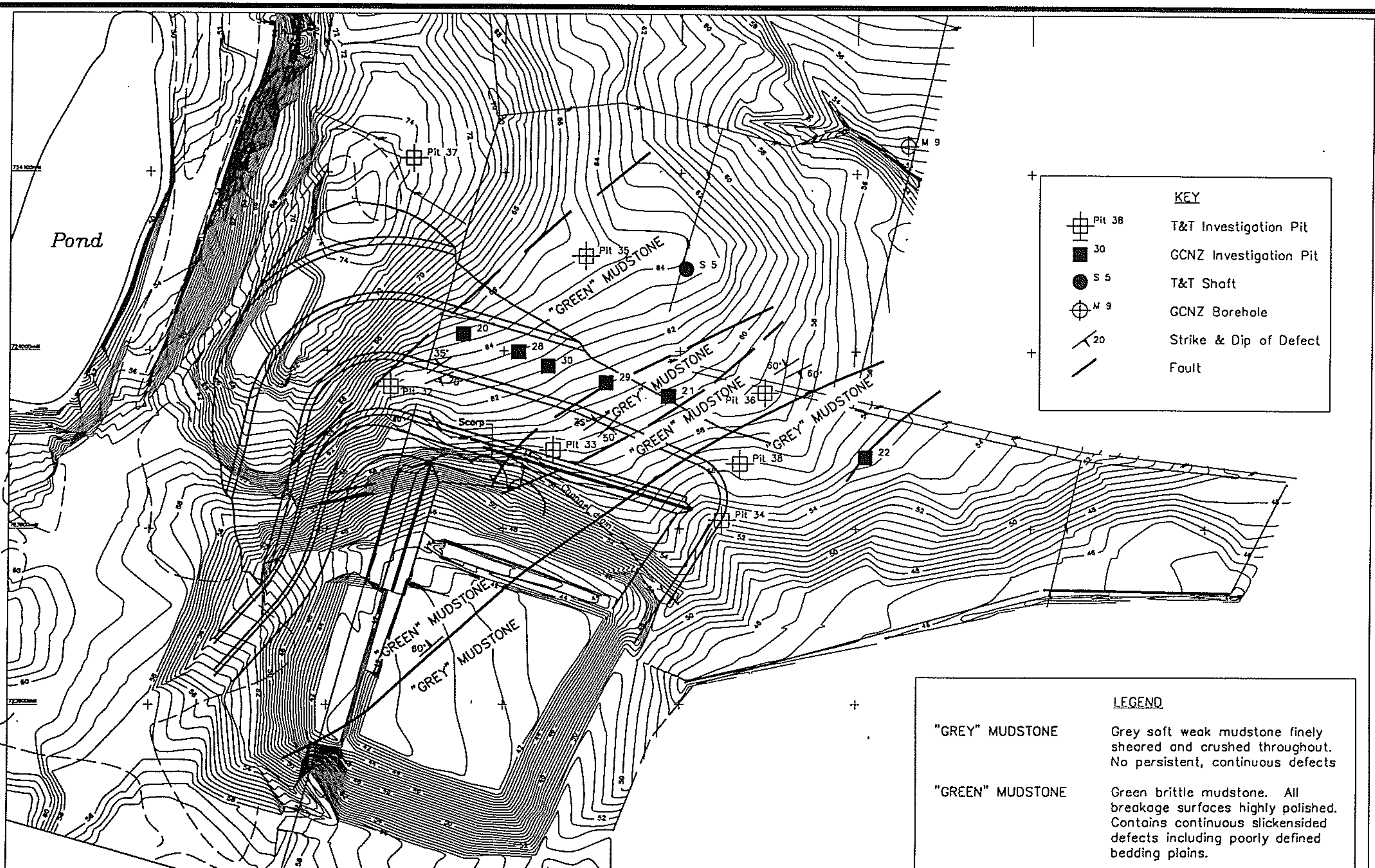
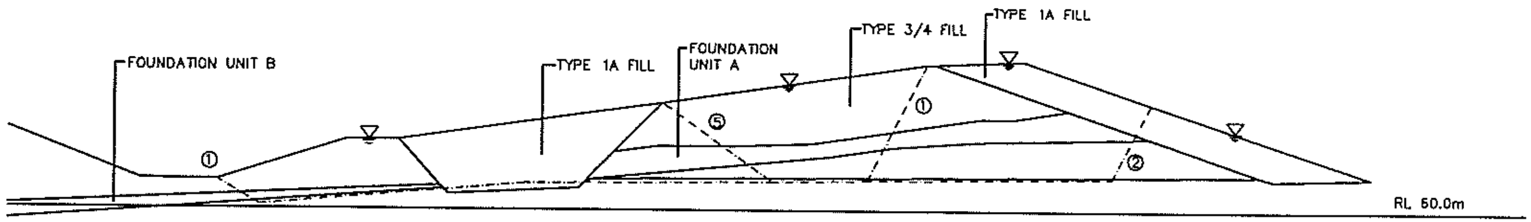
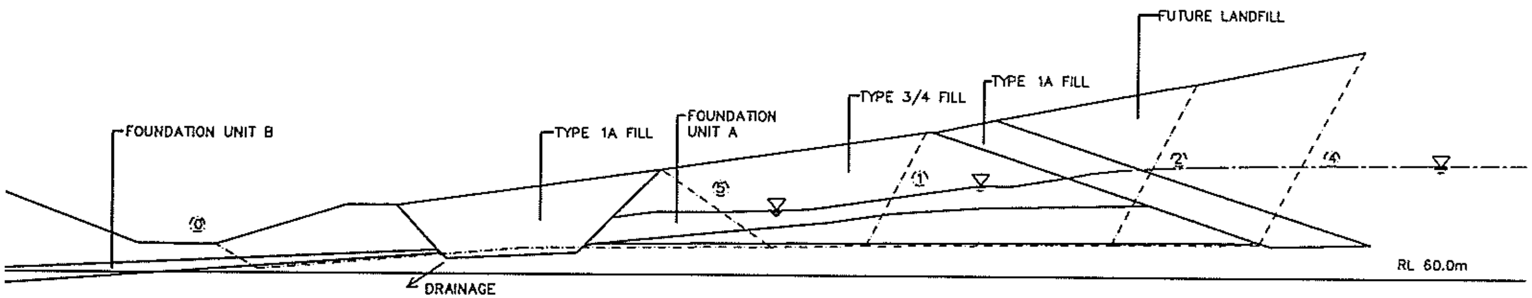


Figure 13 – Phase 1 Cell Excavation Engineering Geology



END OF CONSTRUCTION CASE
(@287305mE)



LONG TERM CASE
(@287305mE)

Figure 14
Phase 10 Bund –Typical Stability Analysis Model

LINER CONSTRUCTION AT REDVALE LANDFILL

B. M. Horide, BE, MIPENZ.
Waste Management NZ Ltd, Redvale Landfill Engineer.

0.0 SYNOPSIS

Redvale Landfill opened in 1993. The lining system includes a low permeability liner of compacted mudstone. Quality control testing during the first phase included Clegg strength tests, field air voids tests, field permeability tests and laboratory permeability tests on core-cutter samples. A correlation between air voids and permeability was established. Effective compaction equipment was identified.

1.0 INTRODUCTION

Redvale Landfill is 30 km north of Auckland, New Zealand. It is owned and operated by Waste Management NZ Ltd. The materials underlying the site include sheared mudstone. The mudstone has low hydraulic conductivity ("permeability").

The first phase of development was completed before opening in August 1993. This development involved excavation to form a containment basin of 15 to 20 m depth below the original ground level. Phase 1 is expected to be filled with 120,000 m3 of refuse within approximately 1 year after opening. The ground adjacent to Phase 1 will be excavated to a similar depth in preparation for Phases 2 to 10.

A low permeability liner of a minimum of 300 mm thickness was formed using compacted mudstone. While quality control was similar to that of typical large earthworks projects, quality control at this site included laboratory and field tests specifically for permeability.

2.0 GEOLOGY AND MATERIALS

The main material underlying the site is sheared mudstone of the Northland Allocthon (Tertiary geologic age). To a minor extent, limerock and inter-bedded mudstone and sandstone of the Waitemata Formation are expected to be exposed in the basegrade in Phases 7 to 10 (in 15 to 30 years' time).

The in situ sheared mudstone and limerock have low permeability, in the order of 1x10E-9 m/s. The natural water content of the mudstone is typically 15 to 20 %. The ground water level is about 2 to 4 metres below the ground surface.

The mudstone has been selected as liner material because its compacted permeability is below the target of 1x10E-9 m/s. Also, there is an adequate supply already on site and its natural water content is close to optimum for compaction.

3.0 LINING SYSTEM

The design of the lining system recognises several key features of the lining environment:

. leachate is pumped from primary collection sumps at the base of the refuse;
. the ground water level is close to the surface - this means that this is an inward-gradient site, i.e. ground water flows into the landfill excavation, reversing any tendency for leachate to seep away;
. the natural ground has low permeability - this means that the ground water seepage flow rate into the site would be small; it also means that if leachate seeps away, it would take a long time to go a significant distance.

The lining system profile comprises (from the top):

. 150 mm thickness of limerock as a protective layer (except at primary leachate collection drains);
. 300 mm thickness of compacted mudstone;
. subgrade.

The compacted mudstone thickness is increased to 900 mm over any subgrade which may have a high permeability (as identified during excavation), and 900 mm over the sloping side walls (3 horizontal : 1 vertical).

4.0 CLAY LINER SPECIFICATION

The key element of the ongoing fill control is to provide a low-permeability earth-fill liner. However, the permeability can not be measured sufficiently quickly in an earthworks situation. Therefore the specification provides for the control of air voids (for which a test result can be obtained on the day of the test), and reference to prior correlation tests between air voids and permeability.

Before construction, an initial correlation between air voids and permeability was developed by laboratory testing. During construction, the correlation is regularly being confirmed by laboratory permeability testing of samples taken from the field-compacted liner fill. The field samples are obtained by 105 mm diameter core-cutters described for in situ density testing in NZS 4402:1986 Test 5.1.3. The laboratory permeability test is an in-house method at an independent soil testing laboratory, involving pre-consolidation, pre-saturation and constant head during testing.

The compaction specification also provides for the control of strength measured with a Clegg Impact Hammer. The minimum allowable Clegg Impact Value (CIV) is 10. To relate that to indicative shear strength, Table 2 shows the correlation which was developed for the recompacted mudstone material at this site.

The field and laboratory test frequencies during Phase 1 are shown on Table 1.

TABLE 1: FILL CONTROL TEST FREQUENCY

Air Voids	1 set (two tests) per 500 m3
Clegg Impact Test	1 test per 50 m3
Permeability (laboratory)	1 test per 500 m3
Permeability (field)	1 test per 2 hectares

TABLE 2: CLEGG IMPACT VALUE / UNDRAINED SHEAR STRENGTH

CLEGG IMPACT VALUE (CIV)	UNDRAINED SHEAR STRENGTH (kPa)
12	200
10	150
8	100
6	60

5.0 LINER TEST RESULTS

Figure 1 presents the results (to August 1993) of permeability testing plotted against air voids. Superimposed on Figure 1 are the results from testing of a low-permeability zone on an earth fill structure in another part of the site using the same material and compaction standard (the "Phase 10 Bund"). The graph demonstrates that the permeability is less than (i.e. satisfies) the target maximum value of 1x10E-9 m/s in the range of 0% to 7% air voids. Therefore the air voids measured in the field in the compacted liner fill are required to be less than 7%.

In order to search for a pattern of changing permeability with increasing voids, a plot of permeability against total voids (porosity) is found to be similar to Figure 1, i.e. there is no obvious trend of changing permeability with increasing voids in the range of results obtained.

The Clegg test results showed no consistent correlation with either water content or air voids in the range of results accepted. It was considered that the strength test results are sensitive to and vary with the nature of the fill material. The mudstone particles were found to vary in hardness and ability to be compacted to a dense mass.

6.0 COMPACTORS

During Phase 1 construction, there was an apparent difficulty of achieving the fill control standards with material from a part of the excavation. Inspection of the fill revealed that gravel-sized pieces or shards of mudstone had not broken down to form a clayey mass as elsewhere, i.e. a portion of the material tended to behave like a granular fraction. In this material, it was difficult to achieve the desired low permeability fill.

However, it is considered that the majority of the fill material (derived from extremely weak mudstone) can provide a low permeability fill, provided that suitable compaction equipment is used. During construction, three compactors were tried. Details of the compactors are presented on Table 3

TABLE 3: COMPACTOR DETAILS

Compactor	Self-propelled Three-wheel	Self-propelled Four-wheel	Towed Single-wheel
Weight (t)	38	20	8
No. of wheels	3	4	1
Wheel dia. (m)	1.9	1.5	1.9
No. of rows of pads per wheel	6 (front) 4 (rear)	4	8
Pad length (mm)	150	250	100 (dia)
Pad width (mm)	100	100	
Length betw pads (mm)	100	100	250
Clean-out depth (mm)	220	110	130
Clean-out blade width & gap width (mm/mm)	80/100	80/100	15/100
Ground contact pressure (t/m2) on 30 degree arc	96	45	90

The towed roller is considered unsuitable due mainly to its poor clean-outs between pads, which have the ill-effect of making the roller tend to behave like a smooth drum roller with relatively low ground contact pressure. The self-propelled compactor is considered unsuitable due to its low ground contact pressure despite good clean-outs. Also, there may be doubt whether the broader, flatter pads knead the soil and adequately break up mudstone particles. Such working of the fill material during compaction is considered important for consistently achieving a low-permeability fill. These conclusions relate specifically to the materials encountered at the Redvale Landfill site.

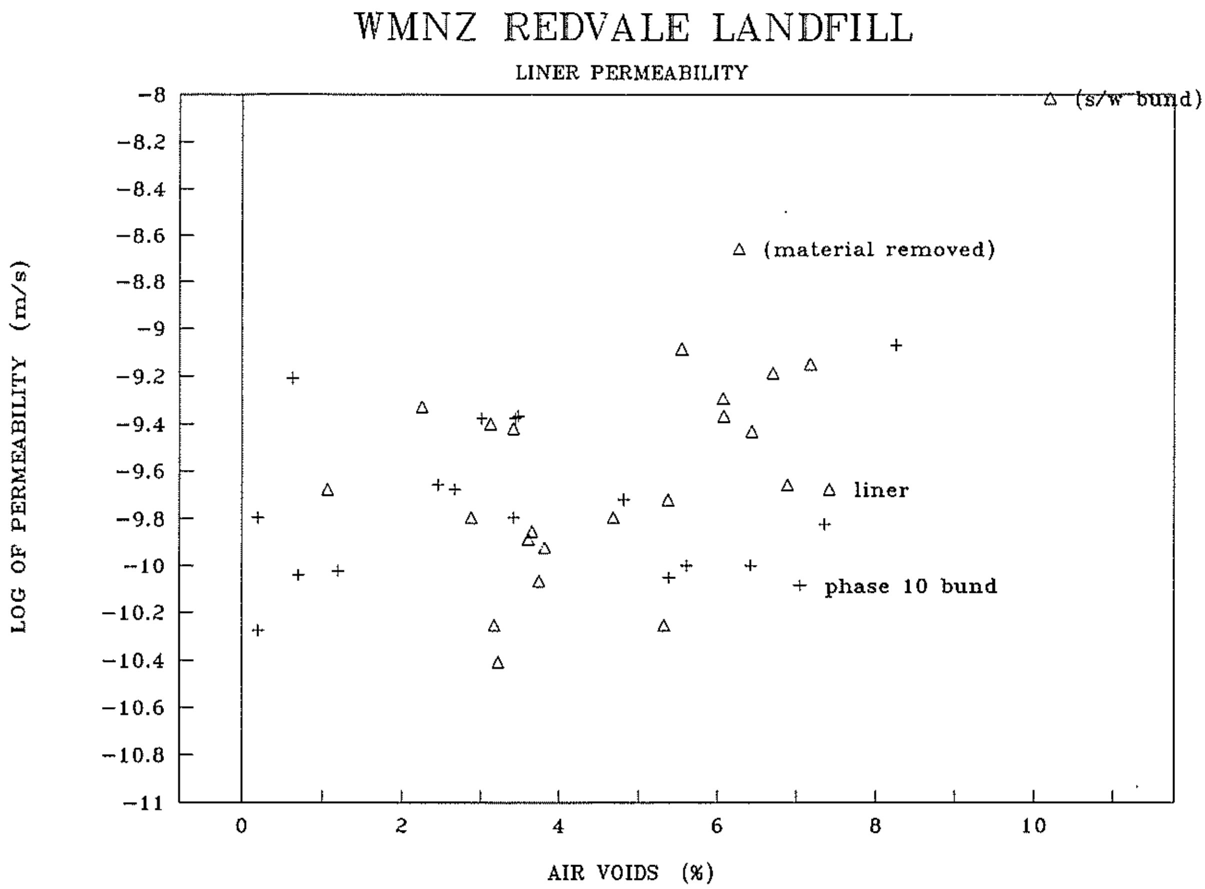
The most successful compactor has the highest ground contact pressure and the most effective clean-outs between rows of pads. These features have been written into the specification for ongoing work.

7.0 CONCLUSION

The compaction standard of a low-permeability fill can be controlled partly by air voids measurement in the compacted fill using a nuclear densometer. The quality control process includes the prior establishment of correlations between air voids and permeability and ongoing confirmations of this correlation.

In order to achieve a low-permeability fill, suitable compaction equipment would have sufficiently high ground contact pressure and adequate clean-outs between rows of pads.

Bruce Horide.
10 December 1993


WMNZ REDVALE LANDFILL
LINER PERMEABILITY
LOG OF PERMEABILITY (m/s)
AIR VOIDS (%)
-8
-8.2
-8.4
-8.6
-8.8
-9
-9.2
-9.4
-9.6
-9.8
-10
-10.2
-10.4
-10.6
-10.8
-11
0
2
4
6
8
10
(s/w bund)
(material removed)
liner
phase 10 bund

"OUR WASTE: OUR RESPONSIBILITY"
CENTRE FOR ADVANCED ENGINEERING'S WASTE MANAGEMENT PROJECT

John Lumsden, Projects Director
Centre for Advanced Engineering
University of Canterbury
Christchurch

SYNOPSIS

This paper describes CAE's second major project, Our Waste: Our Responsibility, completed in 1992. This project considered four aspects of waste management relevant to New Zealand; waste minimisation, hazardous waste management, landfill guidelines, and the impact of waste management practices on water supplies. An over-riding philosophy arising from the project is the increasing relevance of reduction of waste at source and the importance of this, not only in the minimisation of wastes produced, but also in the efficient use of resources including energy.

INTRODUCTION

The Centre for Advanced Engineering was established in 1987 to mark the centenary of the School of Engineering at the University of Canterbury. An appeal fund was launched at this time and to date $2.4 million has been raised. The earnings from this capital sum are used to run the Centre and fund its activities.

The Centre's objective is to enhance engineering knowledge within New Zealand in areas judged to be of national importance and to engage in technology transfer of the latest research information available from overseas.

The Centre's primary activity is in the form of an annual major project. The Centre also undertakes minor projects and arranges seminars and lectures on engineering subjects of current concern.

The Centre's objective is achieved for each major project by bringing together a selected group of practising and research engineers and experts in the particular field from both New Zealand and overseas to:

- consolidate existing knowledge;
- study advanced techniques;
- develop approaches to particular problems in engineering and technology;
- promote excellence in engineering; and
- disseminate findings through documentation and public seminars.

A unique forum for co-operation among industry, the engineering profession and university research engineers is thus provided.

CAE is currently completing its fourth major project, "Towards an Energy Efficient Future for New Zealand". The third project, "Reliability of Electricity Supply", was published earlier this year and the first two projects, "Lifelines in Earthquakes — A Wellington Case Study" and "Our Waste: Our Responsibility — Towards Sustainable Waste Management in New Zealand", were completed in 1991 and 1992.

BACKGROUND

The Centre chose waste management as the theme of its second major project in recognition of increasing environmental awareness in New Zealand. The purpose of this project was to contribute to better management of wastes in New Zealand.

Better waste management not only means appropriate management of the wastes already produced, but more importantly, aims to avoid or reduce future wastes at their source. Better waste management in this sense means less wasting of resources and minimising the risk of pollution from wastes. The word "better" is used because there needs to be increased progress towards the goal of sustainable management of wastes.

Waste management must serve a wide range of social, legal and environmental interests. The importance of these concerns needs to be recognised, and for this reason contributors were sought to provide insights to these issues. Part 1 of the project report discusses Maori concerns, risk assessment, legislative framework and public participation as they relate to waste management.

Four topics considered to be of national importance in the field of waste management were chosen and task groups as follows were set up. Parts 2 to 5 of the project report present the findings of each of these task groups.

- **Waste minimisation practices** — This topic brings together case studies of waste minimisation practices from different sectors of the economy: the domestic, community, commercial, industrial and primary sectors.

- **Hazardous wastes: appropriate technologies for New Zealand** — This topic assesses technologies for managing hazardous wastes considered appropriate for New Zealand.

- **Landfill engineering guidelines** — There are no recognised guidelines currently available for the siting, design, operation and aftercare of landfills in

New Zealand. Such guidelines are presented in this section of the project report.

- **Waste management in relation to water supplies** — The potential in New Zealand for a link between these two subjects is examined. Waste management practices that minimise the risk of polluting water supplies are recommended.

The four topics reflect features in the life cycle of materials and illustrate the integrated nature of waste management.

WASTE MINIMISATION — THE FIRST STEP

In the pursuit of sustainable management of wastes, one view is that the ultimate goal should be total waste prevention. That goal may be unattainable, but preventive options should always be the first priority.

Wastes are considered in the work on waste minimisation as "misplaced resources" — materials that are currently discarded. This definition differs from that used by the other task groups as waste minimisation emphasises preventative solutions to waste management.

Because the long-term storage of wastes is becoming more expensive and problematic with increasingly more stringent treatment requirements, the common sense advantages of waste minimisation are obvious.

The emphasis of this project is on the reduction of waste at source, especially during production. It is important to realise that this approach to waste management not only makes good environmental sense, but makes sound economic sense as well.

TREATMENT OF WASTES AND DISPOSAL OF RESIDUES

While recognising that the quantity of waste produced can be minimised, there will always be an ongoing need to dispose of some waste. The intent and effort must then be to treat wastes and dispose of residues in such ways that any risks to the environment are acceptable and minimal.

Two of the Waste Management Project's topics focus on treatment and disposal of residues: "Hazardous Waste — Appropriate Technologies for New Zealand", and "Landfill Engineering Guidelines".

These topics are closely related because landfills are inescapably physical, chemical and biological reactors that can provide ongoing treatment of wastes. If properly designed and operated, they can also play a useful role in the treatment of suitable hazardous wastes. Landfills need to be viewed, therefore, as both treatment facilities and disposal sites.

"Disposal" is a term that must be used with care. Placing wastes in storage, including containment landfills, where segregation of wastes and exclusion of water are intended to prevent any chemical or biological action occurring is not disposal — it is simply storage. In the future someone will have to decide what to do with wastes so stored, as the containment will not last forever.

Disposal inevitably means the ultimate return of substances to the environment. What we can and should do is convert substances that can harm the environment into ones that will not.

In some cases this may merely mean returning substances to the environment at rates that enable nature's assimilation processes to work without adverse effects. The substances that are finally disposed of after treatment can, for convenience, be called "residues".

THE RETURN OF RESIDUES TO THE ENVIRONMENT

Returning residues originating from treated wastes to the environment will involve all media: water, air and land. These residues will have arisen from chemical and biochemical reactions that change wastes into forms compatible with their return to the environment for ultimate disposal.

Residues may arise from activities that include chemical treatment plants, anaerobic bioreactions in landfills, aerobic composting, incineration, and the biological treatment of liquid wastes, including those from farm and other agricultural sources.

Other residues may enter water, either directly or indirectly via run-off from land. Soils are capable of retaining and breaking down some residues thereby providing residue disposal as well, but the principle of not overloading natural soil ecosystems must be respected.

The topic of Waste Management in Relation to Water Supplies considers an important aspect of the return of residues to the environment, namely, the protection of drinking water supplies and hence human health.

COMMON THEMES IN WASTE MANAGEMENT

Although the Waste Management Project has concentrated on some key technical aspects, links with the wider community have also been recognised. The information provided by the contributing authors is found in Part 1 of the Project Report and is briefly mentioned here.

MAORI INTERESTS

An important aspect of any waste management project is to take into account the concerns and ideas of Maori people, the tangata whenua. The imperative for doing so stems primarily from Article 2 of the Treaty of Waitangi, but also from the requirements of Section 8 and Section 6(e) of the Resource Management Act 1991. There is a clear duty stated in the Resource Management Act 1991 to consult with Maori and to consider Maori interests in such matters, and it is important that those working in the waste management field accept Maori values as a valid system of guiding principles in the management of wastes. The importance of early consultation is emphasised.

RISK ASSESSMENT

A risk is the chance of some undesired event arising from a source of hazard, and the effect of that event and the possible extent of its consequences for people and the environment. In other words, risk has two major components: a probability of occurrence, and a consequence, which has magnitude of loss (or perhaps gain).

Risk assessment has two main components:

- Risk Estimation — the identification and estimation of the probability and magnitude of the consequences of a hazardous event.
- Risk Evaluation — the determination of the significance or value of the estimated risks to those people concerned with or affected by a decision.

Risk assessment should always be followed by risk management. The risk management stage consists of decision-making with the aim of reducing, eliminating or controlling the risk.

The use of risk assessment techniques in waste management, as in some other disciplines, is relatively new. Sometimes the best that can be done is to identify the hazards and seek means of reducing or mitigating them. A precautionary approach in the face of uncertainty is clearly wise, and single lines of defence against hazards should not be relied upon.

Whatever the approach to assessing risks, three issues are always present:

- Who estimates the risk?
- Who evaluates what it means to society? and
- What is an acceptable level of risk? (and to whom?)

LEGISLATIVE FRAMEWORK

In New Zealand there is no **one** piece of legislation that provides a comprehensive framework for waste management.

The Local Government Act 1974 has provisions that enable (but do not require) local authorities to establish and operate water supplies, sewage collection, treatment and disposal facilities, and to collect and dispose of refuse.

Under the Health Act 1956 it is the duty of every local authority to promote and conserve the public health within its district. This includes the provision of waterworks, sanitary works, and the provision of works for the collection of refuse, control of offensive trades and control of nuisance.

The Resource Management Act 1991, although not providing specifically for waste management planning, does have a number of mechanisms that encourage this activity. Regional Councils are required to prepare a regional policy statement to provide an overview of the region's resource management issues. Waste management should, of course, be one of these issues.

There are many other Acts that are relevant to waste management issues. For example, hazardous substances are managed by eight different agencies through more than 15 separate pieces of legislation.

PUBLIC PARTICIPATION

Planning for waste management facilities usually requires that an assessment of environmental effects be submitted with an application for a resource consent.

Consultation is the first stage in any planning process to assess the environmental effects of a waste management facility. An initial consultation with all the stakeholders should assist in the early identification of the significant issues likely to be of concern.

The Resource Management Act 1991 does not require an applicant for a resource consent to undertake consultation, but it does raise the issue in the Fourth Schedule as a matter that should be included in an assessment of effects on the environment.

CONCLUSIONS AND RECOMMENDATIONS

Waste Minimisation Practices

- New Zealand is a wasteful society, therefore waste minimisation is a desirable social objective.
- Society acknowledges wastefulness and desires to change.
- There can be significant economic benefits in adopting waste reduction practices.
- Reduce, reuse and recycle.
- New Zealand produces predominantly organic wastes, which usually have less harmful impacts than the industrial wastes of heavily industrialised countries. A latent complacency exists, however, in assessing the effects of non-organic wastes on the environment.
- Industry in New Zealand is mostly small-scale and generates relatively small amounts of hazardous wastes that, in general, have not been adequately disposed of. Incentives are needed to encourage waste

prevention and the recycling or treatment of accumulated wastes.

- Commercial and domestic wastes are challenging to reduce because they are diffuse in origin and potential end-use. Therefore, emphasis should be on prevention and reduction.

- The project indicates philosophy and action for future direction, but there is also a need for quantitative surveys, information and education.

Hazardous Waste: Appropriate Technologies for New Zealand

- The highest achievable goal in the management of potentially hazardous wastes is the elimination of the production of such wastes in the first place or a reduction in the quantity or hazardous characteristics of those that are unavoidable.

- The OECD system of waste classification is appropriate for adoption and use in this country, with minor modification.

- Progress in the management of hazardous wastes requires consistent definitions and an established system for waste tracking and recording.

- A range of established treatment and disposal options are appropriate for the management of hazardous wastes arising locally.

- The potential for major incidents involving these wastes, and the effects from any such incidents, can be minimised by good management, anticipation and forward planning.

Landfill Engineering Guidelines

- Landfills are, and will continue to be, an essential component of an integrated waste management plan. All wastes directed to landfills for disposal should be subjected to waste minimisation, reuse, recycling and resource recovery practices first.

- The guidelines developed are applicable to all landfills, from small rural landfills to large metropolitan landfills.

- The siting of a landfill requires careful examination of many parameters to establish all potential impacts that such a facility could have on the surrounding physical and social environment. Site selection may well ultimately be determined by the expectations and resources of the affected community.

- The design of a landfill should ensure that it operates as a controlled reactor in which biological, biochemical and physical-chemical interactions are optimised to facilitate the degradation and stabilisation of wastes.

- The operation and management of a landfill requires a substantial commitment of resources to attain the required standard.

- The minimum time period for aftercare of a landfill site is 30 years.

Waste Management in Relation to Water Supplies

- The major risks to the integrity of New Zealand's water supply sources arise from either toxic substances or pathogenic organisms.

- There are many examples of waste management practices with the potential to discharge large quantities of nitrates predominantly into groundwater aquifers. Current technologies are adequate to minimise the effects of nitrate contamination from point source discharges. However, because of our diverse rural economy, current agronomic practices present a greater risk to groundwater supplies from nitrate contamination.

- Other toxic substances which may arise through the use or abuse of hazardous chemicals also present a risk to water supplies.

- There is little understanding of the fate of many pathogenic organisms contained within wastes once discharged into the environment.

- Current technology relies on the use of indicator organisms, which signal that a water supply may be unfit for human consumption. The use of such indicator organisms may not, in fact, offer the degree of protection the public have come to expect.

- There is also little available technology in relation to cross-species infections from various organisms. This has been identified as an area in need of future research.

FUTURE DIRECTIONS IN WASTE MANAGEMENT

A unique and important feature of the Waste Management project has been the extensive involvement of engineers, scientists and others in key positions with local authorities and service companies. The reports produced by the four task groups thus represent the best information currently available in New Zealand.

This work has been further enhanced by significant input from four Visiting Fellows and overseas review by several other recognised authorities.

Societies world wide are demanding more and more that producers of waste take responsibility for those wastes and New Zealand is no exception to this trend. The Centre believes it is important to emphasise what is arguably the most important outcome from this project — the realisation of the increasing tendency among the more enlightened countries to move away from pollution control or so-called "end-of-pipe solutions" that merely treat the problem, towards pollution prevention or waste reduction at source.

The argument for this approach is so compelling that it seems obvious, and yet it is still not widely advocated in this country. More importantly, it is extremely relevant to New Zealand's Resource Management Act 1991, which sets new standards in the efforts to achieve a sustainable future.

Embracing the principles of waste minimisation within a wider concept is the ultimate ideal of clean production, and the road towards this goal is "Cleaner Production". Formulated under the auspices of UNEP in May, 1989, "cleaner production" has been defined as:

"The conceptual and procedural approach to production that demands that all phases of the life-cycle of a product or of a process should be addressed with the objective of prevention or minimisation of short and long-term risks to humans and to the environment."

The Waste Management Project highlighted the need to establish a Centre or Foundation to promote waste minimisation through Cleaner Production in the wider context of more efficient management of resources, including raw materials and energy.

The Ministry for the Environment has set up a number of cleaner production demonstration projects and are publishing guidelines for cleaner production in industry. Similar projects overseas have demonstrated how there can be significant reductions in waste, as well as cost savings, for the companies involved.

DESIGN AND CONSTRUCTION OF AN EFFLUENT STORAGE POND

David N Jennings
Works Consultancy Services Ltd
Hamilton

SYNOPSIS

Twin ponds were constructed to provide 40,000m^3 of storage for effluent as part of the forest irrigation system on the Rotorua Effluent Purification Project. This paper describes aspects of the pond design and construction. Design of an effective lining system was a critical element in providing a safe facility in the highly variable volcanic soils of the site.

INTRODUCTION

The Rotorua Effluent Purification Project (REPP) was designed to reduce the impact of sewage disposal from Rotorua City on Lake Rotorua. Effluent is processed in a Bardenpho treatment plant to remove 80% of the nitrogen and phosphorus before being pumped into the Whakarewarewa forest immediately to the south east of Rotorua where it is spray irrigated on a rotational basis over 14 blocks (Figure 1) and there are two additional blocks to accommodate forest harvesting disruption of the rotational blocks.

The land treatment system is designed for a population of 75,000 and an average effluent flow of 27,000m^3/day. Effluent is irrigated over a forested area of some 300 ha at a maximum loading of 80mm/week.

The Katore Road ponds provide storage in the forest which enables the daily flow of effluent to be managed within the capacity of the forest irrigation system. Two 20,000m^3 ponds have the capacity to accommodate some 1.5 days of effluent production which provides flexibility for periods when irrigation may be restricted because of wet weather.

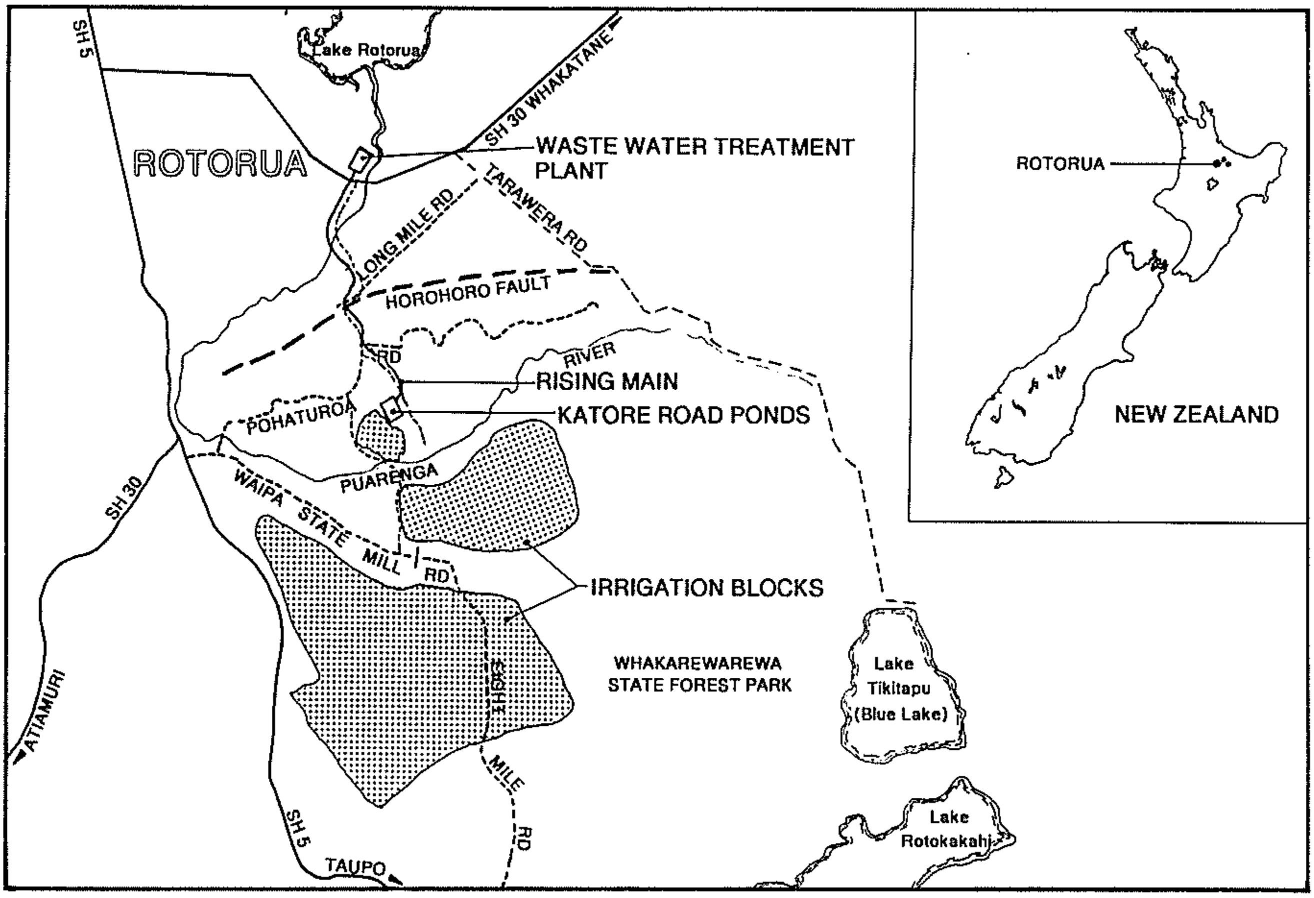


Figure 1 : Pond Location Plan

While the volcanic soils are suited to land based effluent disposal the highly variable recent tephra deposits at the site provided particular challenges for the design and construction of the storage ponds.

PROJECT DESCRIPTION

Twin ponds, each of 20,000m^3 capacity, are located adjacent to Katore Road in the Whakarewarewa forest. The ponds have dimensions of 85m x 75m x 7.3m deep and are separated by a 10m wide berm (Figure 2).

Features include:

- Rising main inlets to each pond
- Independent outlets for each pond
- Controlled pond interconnection
- Spillway structure
- Primary HDPE lining system
- Segmented underdrain leakage detection system

A plan showing the general arrangement of the scheme and the location of the ponds is shown in Figure 1.

SITE GEOLOGY

The Whakarewarewa Forest lies along the northeast end of the Ngakuru Graben along the margin of the Taupo Volcanic Zone. The Horohoro fault (Figure 1) along the northern margin of the Ngakuru Graben forms the southern scarp of the Pohaturoa ridge (the ridge feature between Rotorua (the Rotorua Caldera) and the Whakarewarewa Forest). The Pohaturoa ridge is a fault bounded remnant of the oldest rocks exposed in the region. Because of the history of subsidence the Ngakuru Graben has often been occupied by large lakes in which finer grained sediments have been deposited.

The entire forest effluent disposal area (EDA) is mantled with tephras with variable thickness and composition. Laid down at intervals of a few thousand years the tephras originated from the Taupo and Okataina Volcanic Centers (Table 1). Over much of the region the most conspicuous tephra is the Rotorua Ash (lapilli) erupted from the rhyolite dome east of Lake Tikitapu (Blue Lake) about 13,500 years ago (Nairn 1987).

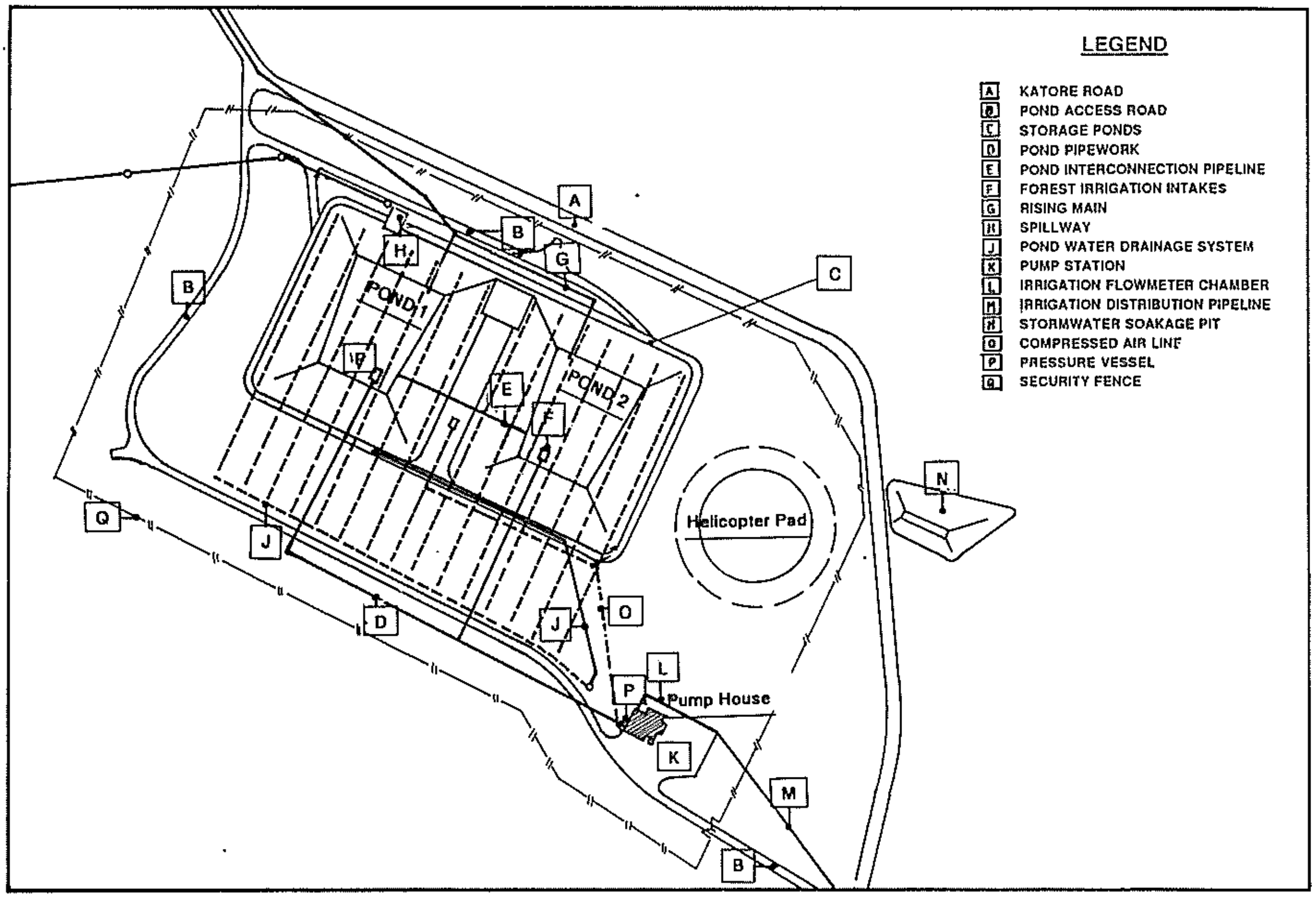


Figure 2 : Pond General Arrangement

Tephra	Age (yrs BP)	Lithology (in EDA)
Mamaku	7500	ash
Rotoma	9000	ash
Waiohau	11000	ash
Rotorua	13000	ash and pumice
Rerewhakaaitu	15000	ash
Okareka	17000?	ash
Te Rere	19000?	ash
Kawakawa	20000	ash

Table 1 : Main tephra deposits which mantle the geology

SITE SELECTION

The location of the storage ponds was constrained by:

(a) position between treatment plant and irrigation blocks
(b) elevation in relation to the irrigation blocks

These constraints dictated a site located in the afforested area along Katore Road.

Two possible sites on Katore Road were considered. Criteria for site selection included:

- storage volume 40,000m³
- minimum water level RL 380m
- terrain characteristics
- site materials

Site investigation and laboratory testing indicated that there was little difference between the two sites and site 2 was selected primarily on the basis of the more gentle contour. The typical soil profile indicated by the investigations was:

Typical Depth	Soil
0-1m	Brown silty SAND with organic material [Topsoil and reworked (Mamaku) ash]
1-2m	Brown silty SAND (Rotoma and Rotorua Ash)
2-5m	Yellow brown coarse SANDS and GRAVELS, pumiceous, size increases with depth (Rotorua lapilli)
5-6m	Yellow brown silty SAND (Rerewhakaaitu Ash and Loess)
6m -	Yellow brown SAND, pumiceous (alluvial sands).

Tree roots from the mature forest had extensively penetrated the upper soils.

SOIL PROPERTIES

Field investigations involved a programme of cone penetration tests (to establish the uniformity of the site and soil properties), plate bearing tests (to measure local compressibility) and excavated test pits (to inspect and sample the soils). The test pits were particularly valuable in evaluating tree root penetration of the soils.

Testing of the undisturbed soil samples indicated permeabilities of:

- Rotoma Ash 1×10^{-5} m/s
- Rotorua Ash 1×10^{-5} m/s
- Rerewhakaaitu Ash 5×10^{-7} m/s

Because of the distribution of soils at the site testing for compaction properties was based on composite layer samples.

Typically the volcanic ash soils were found to be near to or wet (up to 10%) of optimum water content. Maximum dry densities when compacted NZS 4402 standard compaction were typically low (1.06-1.19 t/m^3). Compacted at OWC a 50/50 mixture of Rotoma/Rotorua ashes indicated permeability of less than 1×10^{-7} m/s.

Pinhole dispersion tests on the combined Rotoma/Rotorua ash soils were ND1 (non-dispersive) with no indication of erosion.

DESIGN CONSIDERATIONS

At an early stage in the project it was concluded that the selection and design of an appropriate pond lining system would be essential for the effective performance of the storage ponds. Aspects considered in the design process included:

- storage capacity
- lining systems
- earth works
- material compatibility/erodibility
- variability of the site materials
- site settlement
- leakage
- pond management and maintenance
- seismic stability

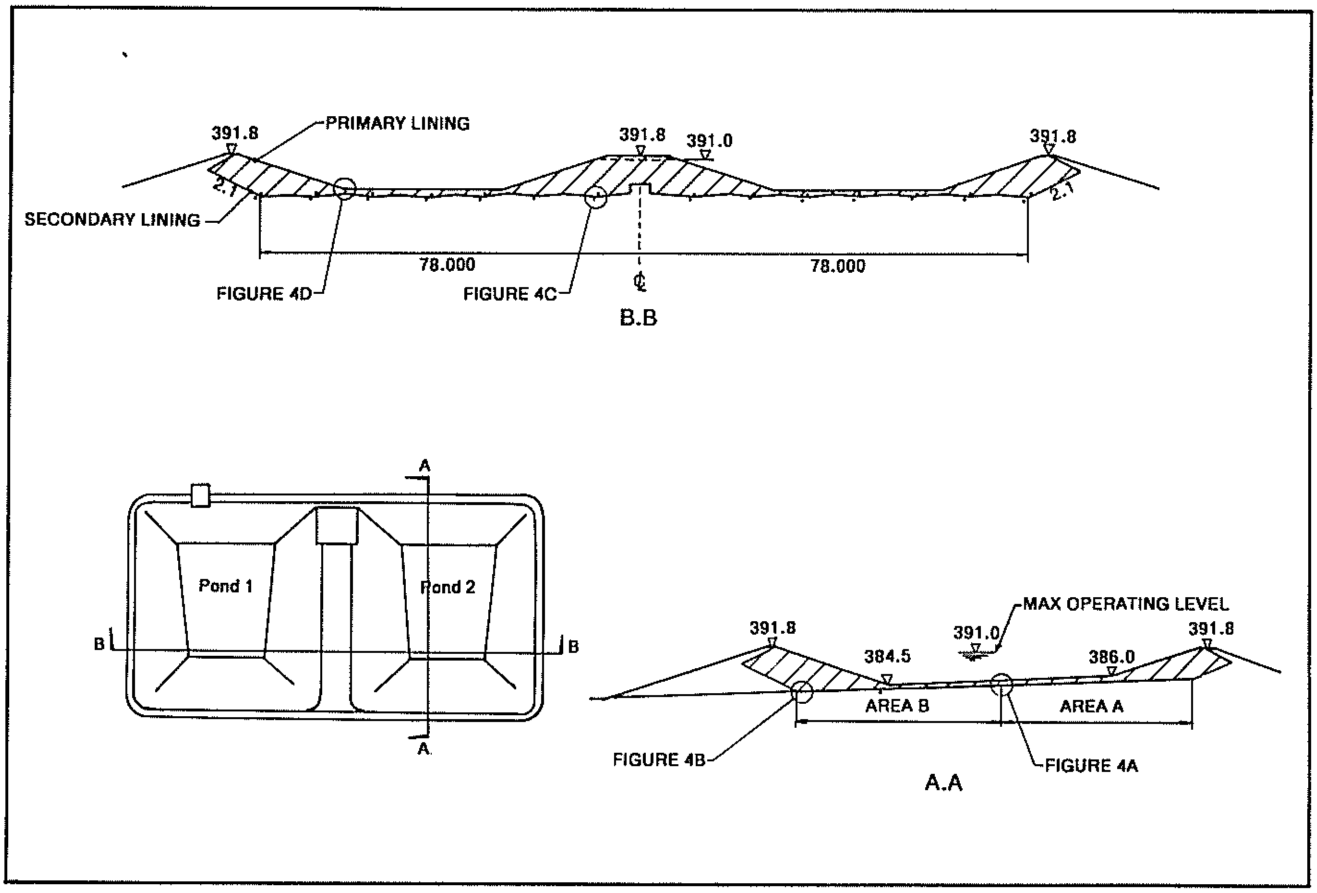


Figure 3 : Pond Lining Arrangement

installed on top of the secondary lining with the general arrangement subdividing the total pond area into 30 subcatchments (Figure 4). The system was designed to enable leakage sources to be traced to a subcatchment of the pond, thus reducing the difficulty in tracing defects. With the primary pond liner permanently exposed and the number of concrete structure penetrations through the primary liner, the assurance provided through early detection of lining damage and leakage was considered essential.

Drain outlets are located adjacent the pond perimeter road which enables easy visual observation during routine site operational visits.

CONSTRUCTION ASPECTS

The construction of the Katore Road ponds involved several important aspects of earthworks operations as listed:

1 Site stripping
2 Preparation of areas for filling
3 Selection and compaction of bulk fill in accordance with the specification
4 Construction of inlet and outlet structures
5 Selection and compaction of brown ash secondary lining materials
6 Application of PMB membrane
7 Placement of drainage layers
8 Further placement of bulk fill
9 HDPE lining construction
10 Access road construction

Construction aspects of particular interest included:

- Bulk fill comprising a mixture of approximately 2.5 parts pumice gravels (Rotorua Lapilli) and 1 part silty sand (Rotoma/Rotorua ash)was found to provide a very good fill provided it was not over compacted. Excessive compaction led to the breakdown of the pumice gravels releasing the water contained in the pumice voids.

- The construction contract extended from July 1989 through to February 1990. Wet weather significantly constrained earthworks activity in the highly weathered ash soils.

- Careful attention was paid to quality assurance for the HDPE liner both in terms of physical damage and weld integrity. Vacuum box testing was found to be difficult and of suspect effectiveness. Ultrasonic testing and weld strength tests were the main quality control tests utilised.

- The sprayed 2mm polymer modified bitumen, with a 2% residual polymer, membrane was found to provide a uniform consistent membrane.

- Installation of the leakage detection pond water drainage system was labour intensive but the contractor achieved a good end result.

POND PERFORMANCE

The ponds were commissioned in mid 1990. A staged filling programme was adopted which involved filling the ponds to RL 386, 388, 390 through to RL 391.0m progressively. Hold periods of 48 hours (minimum) to enable observations and monitoring were included at each stage. A full set of commissioning procedures was prepared for the project. Alarm criteria defining acceptable discharge limits were included for commissioning defining acceptable discharge limits.

Typical drain discharge measurements are presented in Figure 5.

Generally drains exhibited small flows during pond filling (eg 2B, 11 Int) which was interpreted to be related to fill compression/consolidation. Larger flows were experienced in some drains and these were considered indicative of effluent leakage. Because of the range of flow results experienced leakage was confirmed through water quality testing.

Generally ammonia-nitrogen levels were low and did not indicate effluent leakage. It was recognised that the concentrations may have been influenced by the soils. Increasing conductivity and chloride results were considered to be more reliable indicators of effluent leakage because they were less likely to be influenced by the soils.

The ponds were dewatered for inspection. Detailed visual inspection revealed only one minor fault in the HDPE liner. It was concluded that the lining integrity was satisfactory. Long term leakage limits were established for pond operation of:

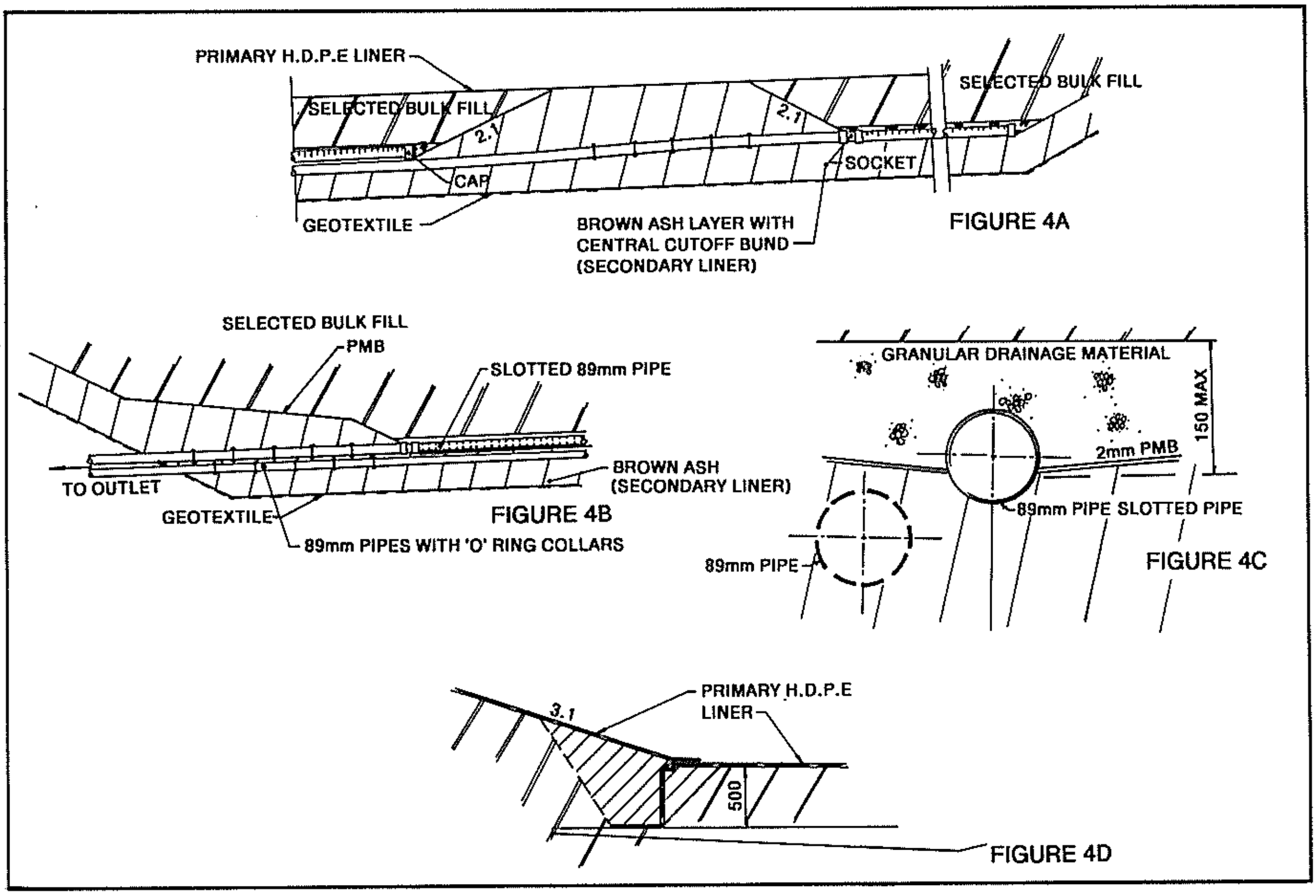


Figure 4 : Pond Leakage Detection Drain Details

While it was observed that the site soils were highly variable, potentially erodible and permeable, (ie not the most suitable on which to construct a pond), construction of a pond was feasible provided a synthetic lining system was incorporated. Because of the potential for erosion instability, as a result of leakage, it was considered a double lining system was essential to enable leakage detection and hence early lining damage identification and maintenance.

Related design issues included:

- potential use of pond for fire fighting
- potential pond lining damage (vandalism, monsoon buckets, etc)

Pond leakage was not considered to be a significant issue (cf irrigation concept for project) provided soil erosion and site stability could be assured.

The two key factors of the pond design were:

- storage capacity
- lining system

Early design options focused on twin ponds with a double HDPE lining system and a physical armour layer. A two pond arrangement was adopted to facilitate inspection and maintenance activities to be undertaken without disrupting the pond serviceability and effluent irrigation operations (ie one pond operating with one out for maintenance) (Figure 2). A physical armour layer was proposed to protect against:

- physical impact (eg, monsoon buckets, wind blown debris, etc.)
- fire
- uplift ballast
- vandalism
- wind/wave movements
- lining UV protection

The design concept involved:

Protection layer	(concrete slabs)
Primary lining	(1.5mm HDPE)
Drainage medium and monitoring drains	(Geomesh/drainage gravel)
Secondary lining	(1.5mm HDPE)

This system was considered to provide a high degree of reliability for the pond in soil conditions that were less than ideal albeit at a significant cost. Following a detailed assessment of costs, the client indicated that a lower cost/higher risk option should be considered further. Risks associate with the ponds, pump station and the irrigation system were reviewed. Areas of modification were to:

- delete protection layer (ie accept risk of damage and premature UV degradation)
- adopt 2.0mm HDPE lining
- adopt composite sprayed membrane/soil secondary liner
- incorporate compatibility layer.

The revised design concept involved:

Protection layer	(none)
Primary lining	(2.0mm HDPE)
Bulk fill	(ash/pumice)
Drainage medium	(selected granular aggregate)
Secondary lining	(2mm PMB over selected ash $k \approx 2x\ 10^{-7}$ m/s)
Compatibility layer	(geotextile)

The general pond lining arrangement is shown on Figures 3a and 3b.

Selected ash involving combined Rotoma/Rotorua ash soils. Testing indicated the combined Rotoma/Rotorua ash soils would provide the required permeability. With a secondary lining permeability of $k \approx 2x\ 10^{-7}$ m/s it was assessed that detection of any leakage exceeding 0.1 litre/sec would be achieved. In addition the pond elevations were established to ensure continuity of the low permeability Rerewhakaaitu Ash across the whole site (ie, effectively a tertiary interception layer).

Bulk fill under the primary HDPE liner was selected on the basis of relative permeability and earthworks/compaction properties and the cohesiveness of the stability against wave action under the primary liner.

Embankment slopes of 3(H):1(V) were adopted and the overall pond geometry is shown in Figures 2 and 3. Seismic stability was not a major issue. Calculated gross embankment stability critical acceleration was $\approx 0.5g$ indicating low risk of seismic induced displacement.

A primary design concern related to the long term performance of the "brittle" volcanic ash secondary lining which would be susceptible to cracking particularly with any seismic settlement or displacement. To reduce the risk of secondary liner failure a flexible 2mm sprayed polymer modified bitumen (2% PMB) membrane was applied over the brown ash surface.

Compatibility of the various materials was considered on the basis of conventional filter criteria. The primary objective was to protect against internal erosion should there be any leakage.

Leakage detection was incorporated into the design with a pond water drainage system. A series of drains was

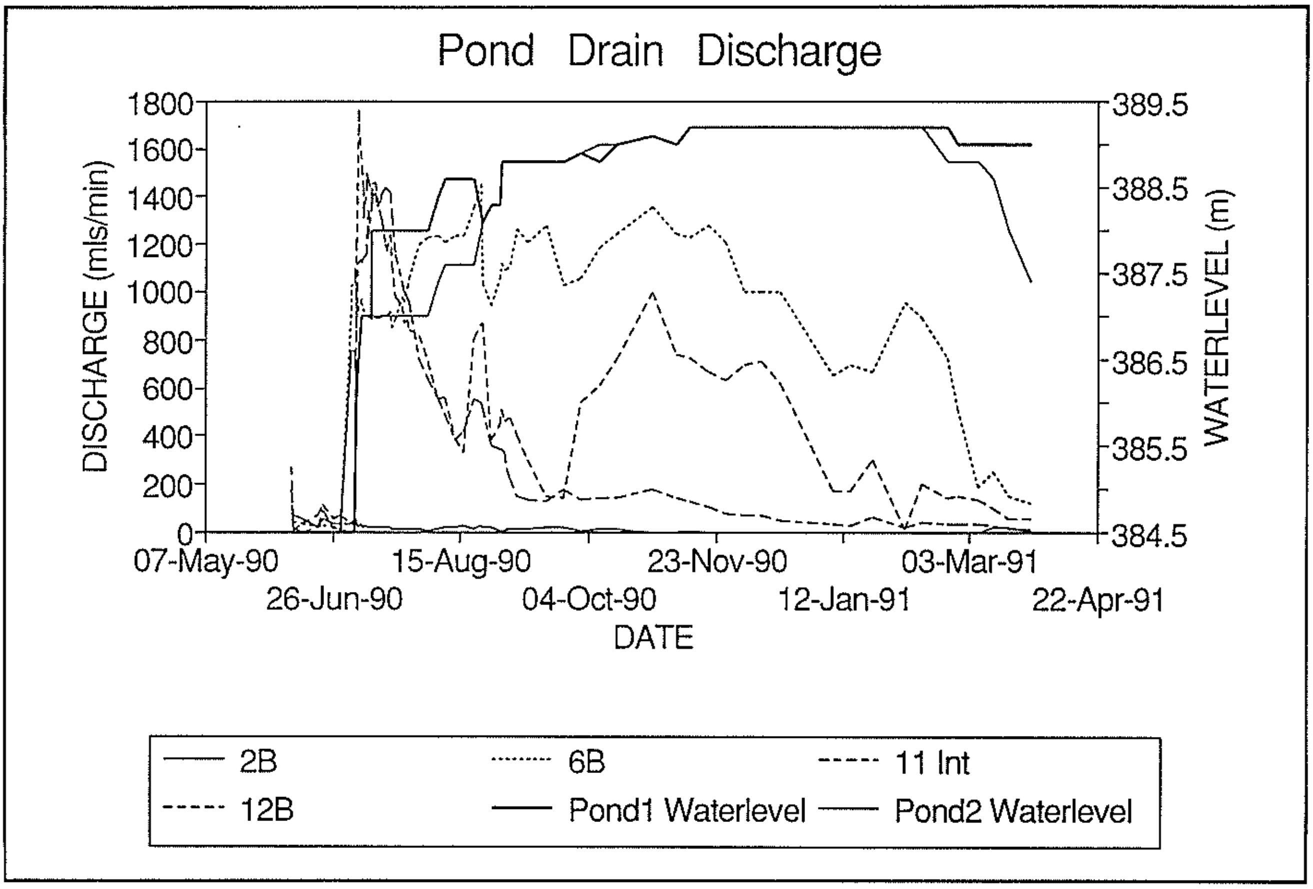


Figure 5 : Leakage Discharge Results

- Individual drain 200 ml/min
- Total all drains 2000 ml/min

SUMMARY

This paper has described some of the features of the twin effluent storage ponds which form an integral component of the Rotorua Effluent Purification Project. The ponds have been constructed in difficult volcanic derived soils some of which are highly permeable.

An integral feature of the successful design and subsequent operation of the pond is the double lining construction and leakage detection system. The ability to monitor leakage provides a high degree of confidence in the performance of the ponds.

The REPP was officially opened on 17 May 1991.

ACKNOWLEDGEMENTS

The author acknowledges the support and agreement of Rotorua District Council in preparing this paper. Support from the many Works Consultancy Services Ltd team members involved with the project is acknowledged as jointly they have produced a sensitive and effective solution to a significant environmental challenge.

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INVESTIGATION OF CONTAMINATED GROUND

R J Parker
Golder Associates Pty Ltd
Melbourne
Australia

T C Logan
Central Laboratories
Works Consultancy Services Ltd
Wellington

A P Smits*
NIWA (Marine)
Wellington
New Zealand

***formerly Central Laboratories, Works Consultancy Services Ltd, Wellington**

SYNOPSIS

There has been a growing awareness in the last decade of the need to identify and clean-up sites which have been contaminated by products from industrial and mining activities. Many countries have followed the lead set by the USA and developed environmental regulations and policies towards soil and groundwater contamination. Within the general population and industry the expectation that the environmental control of potential contaminants is a desirable long term goal is widely held.

INTRODUCTION

There has been a growing awareness in the last decade of the need to identify and clean-up sites which have been contaminated by products from industrial and mining activities. Many countries have followed the lead set by the USA and developed environmental regulations and policies towards soil and groundwater contamination. Within the general population and industry the expectation that the environmental control of potential contaminants is a desirable long term goal is widely held.

From both a social and a legislative perspective, it is no longer acceptable to allow uncontrolled discharge of waste products to the environment. As governments regulate to minimize and eliminate environmental abuse, industry is realising that it will incur significant liabilities if contamination of air, soil and water is allowed to occur or continue.

This paper outlines an approach to the investigation of contaminated soil and groundwater. The paper does not purport to be a manual for conducting environmental investigations but provides an outline of the approaches and discusses some associated critical issues.

INVESTIGATION STRATEGY

It is extremely difficult and uneconomical to complete the investigation of contaminated land in a single stage. To attempt to do so would require a large effort such that investigation costs might bear little relationship to the significance of any contamination. Furthermore it is not possible to determine the best approach to remedial treatment of land without having some understanding of the nature and distribution of contamination. It is for these reasons that contaminated land assessments are generally undertaken in stages.

The philosophy of implementing a staged approach to subsurface contamination investigations will be familiar to those experienced in conventional geotechnical, geological and hydrogeological investigations. In the particular case of contaminated ground the investigation strategy will typically comprise the following stages:

Stage 1 Preliminary Site Assessment (PSA) - The objectives of a preliminary site assessment are to evaluate the potential for site contamination based on the known site history and determine the need for a Stage 2 Site Investigation. A limited programme of sampling and analysis may be required.

Stage 2 Site Investigation - In the event that the Preliminary Site Assessment shows that there is potential for contamination on the site, a site investigation is usually undertaken to verify whether contamination actually exists and if so to identify the principal contaminants and obtain an indication of their concentration and extent.

Stage 3 Systematic Site Assessment - A systematic site assessment may be required if the Stage 2 Site Investigation has positively revealed unacceptable levels of contaminants. The objectives of the systematic assessment will vary depending on the findings of previous stages and the expected end use of the site. The data acquired is used to evaluate the need for site remediation or management, prepare a cost/benefit assessment of remediation work and to provide design inputs for these objectives.

Stage 4 Remediation and Monitoring - At the final stage remedial measures may be required in order to comply with regulatory requirements or other specifications related to the future use of the site. Once remedial design commences a need for further site assessment may be apparent.

DISTINCTION FROM CONVENTIONAL INVESTIGATIONS

While the staged approach of a contaminated site assessment will be comfortably familiar to professionals involved with conventional investigations it is important to appreciate that there are some important distinctions between conventional and contaminated site assessments. A failure to recognise the significance of these distinctions may result in inaccurate data, leading to poor decisions regarding possible remediation measures.

In particular, contamination assessments require careful consideration of the quality of the samples collected with respect to maintenance of chemical characteristics and the inherent problems associated with the handling of contaminated materials. An understanding of the mechanisms and consequences of errors in sampling and analysis is imperative since experience or engineering "gut-feel" cannot readily provide a check of data quality.

To ensure that data from a contaminated site investigation reliably represents the actual in-situ conditions requires stringent sampling and analytical protocols to be adopted from the outset. Decisions on where to sample and how many samples are required for adequate assessment become even more complex than for conventional site assessments.

The following sections outline some of the specific requirements of contaminated site investigations.

IDENTIFICATION OF A POTENTIALLY CONTAMINATED SITE

Contamination assessments can be initiated for a variety of reasons. ANZECC (1992) lists the following possible reasons for initiating a contamination assessment:

- routine surveillance of industrial premises and those generating industrial waste (environmental audits);
- during statutory and regulatory appraisal of industrial sites;
- appraisal following notification to a regulatory authority of accidents or spills at a site;
- appraisal of land which is to be redeveloped;
- appraisal of land where localised environmental effects are noted or suspected;
- environmental assessment on change of ownership;
- health or safety of employees, contractors or general public are exposed to, or potentially exposed to, hazards.

INITIAL EVALUATION

The first step in planning of any contamination assessment is to collect relevant information about the site and waste characteristics. Without this basic information it is not possible to predict where contamination might be found, how contaminants should be sampled and analysed, or what effect contamination may have on personnel conducting the investigation.

In the USA, the Association of Engineering Firms Practising in the Geosciences (ASFE, 1991) surveyed its members to assess the "state of the practice" with respect to contaminated land assessments. From the survey they concluded that the best description for these types of assessments or audits was Preliminary Site Assessment (PSA). They considered that this term could be used to describe two levels of investigation which might involve either one or the other of the following activities:

- **Level 1:** Historical, ownership and regulatory review and site visit. This stage involves collection of "background data" including historical information such as property titles, records maintained by state and local government agencies, operating information including raw materials, products and wastes, published documents, reports, anecdotal information from existing or former employees of a site and published geological and hydrogeological information.

- **Level 2:** Level 1 plus geophysical or nominal intrusive exploration and sampling; soil, groundwater or surface water analyses. The objectives of this stage of work would be to detect contamination (if present) rather than to define its extent.

In conventional application a PSA is conducted to determine the likelihood of a site being affected by substances considered "contaminants" based on applicable authorities' definitions of contaminants, hazardous materials, pollutants, etc. There is no clearly defined scope of work required for a PSA although ASFE (1991) suggest four main components:

(a) initial interview with owner/client;
(b) review of public and other historical records;
(c) site reconnaissance and interviews;

(d) environmental evaluation report.

Specific requirements for each of these tasks are listed in Table 1. This table is based on the ASFE scope of work but has been modified to suit more general applications rather than reflect specific US conditions.

The level of detail ultimately required during conduct of a PSA will depend on a number of factors, including:

- the history and use of the site and risk to the environment
- the expected future use of the site
- the physical characteristics of the site, including sub-surface conditions
- the purpose of the assessment
- the risk to the assessor (i.e. what qualifications the assessor can include with his report)

The decision to conduct a Level 2 rather than a Level 1 assessment will be dependent on the likely risk of contamination associated with the site and the level of confidence in conclusions required by the organisation requesting the PSA. There are some sites where contamination of soil or groundwater might be considered extremely unlikely. Nevertheless, there are cases where significant contamination has resulted from inappropriate dumping of wastes or from farming activities, even though the current use would suggest that the site is likely to be clean, residual contamination may still be present.

In most cases in New Zealand, at least some limited sub-surface investigation will be required as part of the PSA. Where contamination is considered likely or is expected to exist then sub-surface investigation should be performed during the PSA.

DESIGN OF INVESTIGATION PROGRAMME

There are many techniques for investigating sub-surface conditions where contaminated soil and groundwater is anticipated. Many of these are familiar to geotechnical engineers and geologists, including test pits, borings and geophysical methods such as resistivity, seismic refraction, ground penetrating radar, electromagnetic induction and borehole geophysics, all of which are readily available in New Zealand.

In all sub-surface contamination assessments there is a need for the collection of samples for chemical analysis. The most common methods of investigation are:

- sampling of soil and rock from boreholes
- sampling of groundwater from monitoring wells
- sampling of volatile constituents from boreholes or probes.

A fundamental requirement in the design of a sampling programme for contaminated soil and groundwater is the need to ensure that samples are representative of the chemical conditions in the ground. In particular, it is necessary to ensure that contaminants are not introduced to the sample from adjacent parts of the borehole, from other boreholes on the site, or from sources off the site. Consideration must be given in the design of a sampling programme to provide for quality assurance procedures during drilling, sampling, handling and transport to the laboratory. It is common practice to document the quality assurance procedures in the form of a work plan. ANZECC (1992) contains a useful discussion of issues that may be important in the development of a work plan.

Given the potential hazards associated with some of the materials that must be investigated, it is necessary to consider the health and safety of field personnel involved in the contamination assessment. Health and safety plans should be prepared for all assessments where significant contamination is expected. All field personnel should be trained to cope with unforeseen circumstances that may result in deleterious effects on health. These should include description of the possible hazards, protection methods available, emergency response plans, action levels and responsibilities.

Critical to the design of contamination assessment programmes is the need to have a clear understanding of the nature of the contamination and its likely behaviour in the ground. Without this understanding there is the risk that sampling locations will be in the wrong place or the laboratory analyses will be inappropriate to detect the contaminants of concern. It is for this reason that a staged programme and results of the PSA are important aspects of any major contamination assessment and remedial treatment programme.

Problems in Investigation Programme Design

Although it is beyond the scope of this paper to discuss the migration and behaviour of contaminants in the soil, rock and groundwater, it is worth considering the following problems which illustrate some of the complex issues that need to be considered in planning contaminated land assessments.

The first problem is that the concentration of contaminants in the ground may change over time due to chemical degradation, sorption or evaporation. The rate of such changes is dependant on the level of microbial activity and the physical and chemical characteristics of each site.

For example trichloroethylene (TCE) may degrade to form dichloroethylene (DCE) which in turn degrades to form vinyl chloride (VC). The consequence of this type of reaction on a monitoring programme is illustrated in the following simple examples.

(a) TCE release is suspected from a site. Analysis of groundwater indicates reducing concentration of TCE down gradient of the site. This may be a consequence of degradation of TCE rather than definition of the edge of the plume.

(b) TCE release is suspected from a site but a VC plume is detected separate to the main TCE plume. The VC may be contamination from an alternative source or from the TCE source if degradation has occurred.

A second problem is that the distribution of contaminants in the soil may be very complex depending on their density, water solubility, volatility, chemical reactivity etc. An example of this is ground contamination by a dense non-aqueous phase liquid (DNAPL), such as one of the polychlorinated biphenyls (PCB). These chemicals, as the name implies, are heavier than water and will therefore sink through the groundwater. They also have very low solubility in water.

It is very difficult to define the extent, concentration and rate of migration of all vapour, liquid, dissolved and sorbed DNAPL's especially at sites where there are significant lateral and/or vertical changes in the physical properties of the soil. Even a very carefully designed investigation programmes might not accurately define the distribution of the contaminant in the ground.

DRILLING FOR CONTAMINATION ASSESSMENT

In designing a investigation programme it is necessary to consider how boreholes, test pits or hand auger holes can best be drilled so that contaminants from external sources are not introduced into the bore or excavation. For shallow investigations, hand augers and test pits are commonly used for investigation and sampling purposes. If disturbance of the ground and large excavations are not of concern, backhoe pits may be appropriate although it is sometimes difficult to obtain discrete samples without entering the pit, which is often inappropriate for sampling in contaminated ground.

The favoured method for drilling bores in soil for contaminant studies, though rarely employed in New Zealand, is by use of hollow stem augers since these do not introduce fluids to the bore and thus reduce the volume of potentially contaminated material for disposal (i.e. no fluid return). Samples can be recovered through the annulus of the augers.

In many cases, drilling with augers is not possible or inappropriate and wash boring must be considered. Careful planning to collect and dispose of cuttings may be required where contamination is significant.

Although rock is not very often sampled for contamination assessment (do you sample the rock matrix or the defects?), drilling in rock is common for installing groundwater bores. In contaminated ground, the most appropriate method for rock drilling is by cable tool which requires no introduction of fluids and minimises the volume of cuttings brought to the surface. However, because this method is slow, alternatives such as down-the-hole hammers or rotary drilling are often considered.

It is important to ensure that the drilling rig and all drilling equipment are clean prior to site mobilization and prior to setting up at each new bore location. This can generally be achieved by use of portable steam cleaning equipment. If all equipment is not thoroughly cleaned there is a risk of introducing contaminants to the site or transporting contaminants between bores which may result in difficulties in detecting the edge of a contaminant plume or may completely invalidate results when detection of low concentrations of contaminants is required.

Extreme care must be exercised during design and drilling of a bore to ensure that the drilling process does not drag contaminated soil from near the surface down to the groundwater, or provide a pathway for migration of contaminants from a contaminated aquifer to a clean aquifer. These problems may be overcome by appropriate design and installation of multistage casing to isolate contaminated zones prior to the continuation of drilling and sampling to greater depths.

Groundwater bores require documentation of design and installation details to provide an assurance that the materials and methods used to construct the monitoring bore cause misleading analytical results. Material selection for bores is usually influenced by the possibility of physical or chemical interaction with contaminants, the analytical accuracy required, material cost and the design life of the sampling or monitoring installation.

In most circumstances unleaded PVC is considered to be sufficiently inert, although the use of PVC glue may be inappropriate and threaded casing is commonly used. In extreme cases stainless steel or teflon casing may be required. Mild steel pipe is generally not favoured for use as casing for contaminant monitoring. Casing should generally be steam cleaned before use and prepackaged clean sand, gravel and bentonite should be used for packing of bores. Monitoring bores should be developed after installation to remove sediment and improve flow.

SAMPLING

Soil: Care must be exercised in the sampling of soil and groundwater to ensure that samples are representative of in-situ conditions and that the sampling procedure itself does not lead to contamination of the material recovered. The Standard Penetration Test sampling spoon is a useful technique for recovering soil samples from boreholes. The major advantages of this sampling technique are that samples can be removed quickly from the device and the equipment can be easily cleaned. However, there are also many "push-in" type devices for recovery of samples with minimum chemical disturbance, particularly for sampling where volatile chemicals are involved.

Groundwater: In addition to development of a bore after installation, it is generally advisable to purge water from the bore prior to sampling so that formation water is sampled rather than water that has been standing in the bore. Typically three to five bore volumes of water should be removed prior to sampling. A groundwater property such as pH or conductivity may be monitored to assess the effect of purging.

A variety of methods are available for sampling or purging water from a monitoring bore, including bailing, air lifting and pumping. If the introduction of oxygen to the bore or reduction in pressure is likely to change the nature of the contaminants in the groundwater when air lifting and some methods of pumping may not be appropriate.

All equipment used for sampling of soil and groundwater and for purging should be thoroughly cleaned before being inserted into the bore. For some sampling programmes this might require washing with acid and/or phosphate free detergents and several rinses with deionised water. As a minimum requirement, all equipment should be rinsed with deionised water prior to use in a bore.

Blanks: Field blanks should be prepared to check the adequacy of cleaning and handling procedures. These are prepared by pouring deionised water over the sampling equipment and collecting the water for analysis. Duplicate or spiked samples should also be prepared to assess laboratory handling and testing procedures.

HANDLING OF SAMPLES

Once a soil or groundwater sample has been removed from a bore, care must be exercised to ensure that the constituents do not degrade or become altered prior to it reaching the analytical laboratory. Advice from the analytical laboratory should be obtained prior to sampling to ensure that samples are handled and transported in an appropriate manner. Most analytical laboratories will supply suitable clean sample vessels with preservatives already added, if necessary.

Degradation of some chemicals is temperature dependent and it is good practice to store samples in a cool environment, e.g. packed in ice or stored in a refrigerator. It is also good practice to deliver samples to the laboratory as quickly as possible after sampling. Groundwater samples recovered for metals analysis should generally be filtered in the field.
All samples should be maintained under "chain-of-custody" documentation which enables the tracking of parties responsible for the samples at any given time. This may be required for legal identification of samples and to be able to demonstrate that the samples analysed were in fact the ones collected in the field.

VOLATILE CONSTITUENTS

Particular care must be exercised when sampling soil or groundwater contaminated with volatile chemicals. If correct sampling and handling procedures are not adopted then the concentration of volatile materials can be grossly under-estimated or even go undetected.

Because of the problem of loss of volatile constituents after sampling, it is common practice to perform field assessment in addition to the laboratory analysis. A range of field equipment is available for the detection and quantification of volatile constituents. These include:

- Flammable gas detectors - commonly calibrated for methane.
- Photo-ionisation detectors (PID) which can be used for a range of organic volatile species. Lamps with differing ionisation potential can be used to expand the range of species that can be detected.
- Flame ionisation detectors which are capable of detecting a wider range of organics than the PID.
- Portable gas chromatographs which can be used for identification and quantification of concentrations of volatile organics in the same way that laboratory GC's are used.
- Colorimetric tubes, e.g. Draeger tubes. These simple devices are available for detection and quantification of a wide range of organic and inorganic gases.

These devices can be used as part of site health and safety monitoring or can be used for detection and quantification of concentrations of volatile constituents. Laboratory analysis is normally required to confirm field monitoring results.

Field monitoring devices for volatile constituents are typically used for:

- atmospheric monitoring
- monitoring at the collar of a borehole or well
- monitoring in tubes driven into the ground
- field head space testing

SAMPLING FREQUENCY

The determination of how many and where to sample is a common problem for all assessors of contaminated land. Given the inevitability of budget constraints, it is always necessary to carefully plan an investigation to gain maximum information for reasonable cost. It is obvious that all of the soil or groundwater on a site cannot be sampled and therefore design of a programme must consider the number of samples that are required to be statistically meaningful.

There are no universally recognised techniques for determining sample frequency, and it is generally up to the assessor to determine the acceptable number of samples. In doing so the assessor must consider:

- Previous site use. Sampling may target specific areas where contamination is suspected (e.g. adjacent to an underground storage tank) or, alternatively, grid sampling may be adopted where either a uniform distribution of contamination is suspected or where little is known about the site.

- The intended use of the site. For example, if the site is to be used for residential purposes then the sampling frequency should reflect the block size, possibly requiring that each and every block is sampled. Alternatively, if the site is to be used for industrial purposes then a lesser number of samples may be appropriate. In addition the local authority will have to be satisfied before it issues a building consent if a development of some kind is planned.

- The type of investigation. The sampling frequency and layout for a preliminary investigation will be very different to the frequency and layout for a validation programme after cleanup.

- The mobility of the contaminants. The mobility of the contaminants will be dependent on both the physical characteristics of the site (e.g. permeability and depth to watertable) and the characteristics of the contaminants (e.g. ability to sorb to the soil or solubility in water).

- The liability or risk to be assumed by the assessor or auditor. Although this is not a technical consideration, it is nevertheless a critical aspect of the design of the assessment or audit programme.

LABORATORY ANALYSIS

It is important to understand that analytical procedures are chemical specific and the laboratory will only analyse for the chemicals which are requested. For organic chemicals, techniques are available for identifying and quantifying a very wide range of chemicals (gas chromatography/mass spectrometry). However, this testing can be very expensive and may be inappropriate for many investigations. If chemical specific identification and quantification is not required, screening techniques are available to identify classes of compounds present.

Fortunately there is a limited number of chemicals that are considered to be hazardous and can be expected to occur in the environment. In the USA for example, the EPA has published lists of hazardous chemicals for the various regulatory programmes. These lists include no more than a few hundred compounds. For example, for groundwater assessment there is a list of 133 chemicals which are generally considered in any analytical programme ("Priority Pollutants"). Where there is no regulatory list of contaminants, some professional judgement is required in the design of an analytical programme.

It is critical in the design of any analytical programme to have a sound understanding of the nature of chemicals used on a site through an appropriate background study. By completion of such a study it is often possible to place a reasonable limit on the number of analytes.

When specifying a laboratory analysis programme it is important to consider the detection limits for each analyte which depend on a number of factors, including the analytical method used, extraction methods, sample matrix, etc. There is little point analysing for a particular chemical constituent if the critical limit for that constituent is lower than the detection limit of the analytical method specified.

Quality assurance is as important in planning laboratory programmes as it is in the field sampling programme. Typically the following points should be considered:

- Wherever possible, laboratory analyses should be performed using standard methods, preferably recognised by the appropriate authorities.

- Appropriate laboratory quality control procedures should be used. These should include analysis of duplicates, spikes and appropriate use of reference standards. In budgeting for an analytical programme, an allowance should be made for quality control testing, generally 10-15% of budget.

- There should be periodic auditing of laboratories, including both systems audits and performance audits.

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TABLE 1: Checklist for Preliminary Site Assessments

(Note: All items may not apply to each site)

A. Initial Interview with Owner/Client

1. Identify third parties (e.g. financing entity).
2. Nature of surrounding area and land use.
3. Nature of specific site; age of existing buildings; known underground tanks (and coverings); known asbestos (if included in scope).
4. History of the site.
5. Permits; applications; notifications, inspections.
6. Regulatory violations.
7. Size and specific location of site.
8. Ownership and access.
9. Chronology of ownership; title review.
10. Site utilities; stormwater drainage; sewer systems.
11. Wells; water supply.
12. Proposed use of property, intended excavation.
13. Site sampling and analysis.
14. Site plans; show specific site boundaries; define limits of study area.
15. Confidentiality and legal privilege.
16. Ultimate recipient of report and any special requirements concerning content and/or preparation; request for certification; clarify purpose of report.
17. Contact person(s); written entry permission.
18. Schedule.
19. Additional areas of concern.
20. Health and safety concerns or incidents

B. Review of Public and Other Historical Records

1. Central and local government concerns. Local legislation and regulation with respect to air emissions, surface water release, trade waste agreements, discharge agreements.
2. Local concerns.
   (a) Planning authority, territorial authority, local water/sewer authority.
   (b) Fire service.
   (c) Previous owners, occupants, workers or residents.
3. Evidence of past activities.
   (a) Newspapers.
   (b) Libraries.
   (c) Local historical societies.
4. Legal record of past ownership.
   (a) Title review.
   (b) Other records.
5. Review existing maps and similar data.
   (a) Aerial photos.
   (b) Historical maps.
   (c) Soils map.
   (d) Geological maps.
6. Review company records.
7. Review geological/hydrogeologic setting.
   (a) Municipal/residential water supplies.
   (b) Local wells, reservoirs.
8. During review, note sites within a selected radius that are:
   (a) Contaminated site registers.
   (b) Suspected contaminated sites.
   (c) Operating or inactive landfills.
   (d) Hazardous waste facilities.
   (e) Industrial or wastewater discharge to surface waters run through or near the site.
   (f) Underground tank records.

C. Site Reconnaissance and Interviews

1. Safety plan.

2. Visual reconnaissance.
   - (a) Topography/fill areas.
   - (b) Surface conditions.
   - (c) Drainage.
   - (d) Ponds or ponded water, streams, rivers, wetlands.
   - (e) Wells.
   - (f) Utility lines.
   - (g) General housekeeping.
   - (h) Soil.
   - (i) Odour.
   - (j) Vegetation.
   - (k) Debris.
   - (l) Vent pipes.
   - (m) Tanks.
   - (n) Storage buildings and storage areas.
   - (o) Drums and miscellaneous chemical containers.
   - (p) Transformers.
   - (q) Potential need for asbestos study.

3. Photographic documentation.

4. Interviews with site personnel.
   - (a) Handling of hazardous materials.
   - (b) Spills.
   - (c) Underground storage tanks.
   - (d) Monitor wells.
   - (e) Environmental monitoring.
   - (f) Asbestos.

5. Use of adjacent properties.

6. Sampling and analyses (need for, if stage 2 study is warranted).
   - (a) Surficial soil/sediment samples.
   - (b) Test pits/soil borings sampling.
   - (c) Surface water/monitor wells.
   - (d) Sampling.
   - (e) Analyses.

D. Environmental Evaluation Report

1. Introduction.
2. Topography.
3. Existing site conditions.
4. Historical records review.
5. Geology, hydrogeology
6. Public record review.
7. On-site reconnaissance.
8. Analytical results.
9. Findings and recommendations, need for additional work.
10. Limitations.

USING NON-INVASIVE, NON-DESTRUCTIVE GEOPHYSICAL METHODS TO MAP AND TO MONITOR THE SOURCE AND EXTENT OF CONTAMINATION

David C. Nobes
Department of Geology
University of Canterbury

SYNOPSIS

Sampling wells and drill holes are, by their very nature, single points. Geophysical survey methods provide lateral control between wells, and are best used in advance of any drilling or sampling, so that any holes can be placed to best advantage. Geophysical methods are, by their very nature, non-invasive and non-destructive, and are thus well suited for the delineation of the source and/or the extent of contamination, when the integrity of the site must be preserved. The physical basis of some common geophysical methods will be outlined, particularly electromagnetic and radar methods. Some recent case histories from the Christchurch area will be presented which illustrate the use of geophysical methods for mapping the extent of leachate from landfill sites.

INTRODUCTION

Holes drilled for water supplies or for monitoring may not be representative of the regional picture. Even closely spaced holes can give an erroneous view of the structure of the sediments, or the extent of contamination. For example, the patterns evident in downhole geophysical logs are often used to follow certain beds from hole to hole. The real complexity that is present is often overlooked, as shown by the example in Figure 1 (from Slatt et al., 1992). Surface geophysical methods provide the additional lateral continuity that is needed.

In addition, geophysical surveys are best carried out before any drilling. In hydrocarbon and mining exploration, no drilling is done until after the geophysical surveys are carried out, since the anomalous areas are first identified using surface geophysical surveys. This approach is the most cost-effective way of using limited resources. The cost of a geophysical survey is similar to that for drilling one borehole, and the wells that are drilled are fewer in number and are placed in the most effective manner to yield the maximum amount of information for the least amount of time, effort and money.

A few geophysical methods have been used to a limited extent. Electrical resistivity and seismic refraction have been used in engineering and environmental work for some years. More recently, the use of electromagnetic (EM) methods and ground penetrating radar (GPR) is expanding. EM methods were the mainstay of mining exploration for decades, but are now being applied to groundwater exploration and monitoring. GPR, also called ground probing radar and subsurface interface radar (SIR), is relatively new, however, and its range of applications is still growing. The object of this paper, then, is to review the basic principles of EM and GPR methods, and to present some case histories from the Canterbury region, particularly around Christchurch. The examples will, in keeping with the theme of this symposium, concentrate on the mapping and monitoring of contamination, in this case leachate from landfill sites. For an excellent overview of the many geophysical techniques in use and their application to geotechnical and environmental work, I refer the reader to the set of volumes edited by Ward (1990).

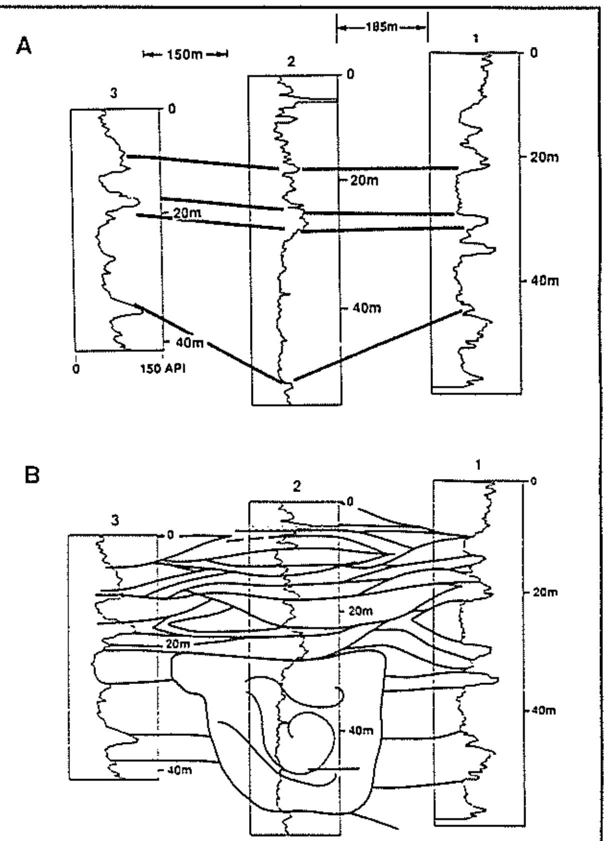

Figure 1. Gamma logs taken at a quarry wall and interpreted in the standard way (top), compared with the observed *complex* geological structure (bottom). Note the rather large discrepancy between the two. (From Slatt et al., 1992.)

GEOPHYSICAL BACKGROUND

Electric and Dielectric Properties

Electromagnetic (EM) methods and ground penetrating radar (GPR) are based on straightforward physical principles, and are to a large extent complementary. EM techniques respond to changes in the subsurface electrical properties, which are in turn governed by three dominant factors (e.g., McNeill, 1990): (1) the water content, (2) the water quality, and (3) the clay content. This dependence is best expressed as

$$[1] \qquad \sigma = \sigma_w \phi^n + \sigma_c$$

where σ is the formation electrical conductivity, in siemens/metre (S/m) or more usually in millisiemens/m (mS/m), σ_w is the pore water conductivity, and ϕ is the (fractional) porosity raised to an exponent n that is a function of the pore shape which is in turn governed largely by grain size. σ_c is the contribution from any clay minerals that are present. The electrical conductivity is a measure of the ease with which a unit volume of material conducts electric current.

Most natural solids, with the exclusion of metals, are not naturally conducting. Water, however, can dissolve ions, and greatly improve the conductive capabilities of natural materials. The greatest effect of water on the electrical conductivity occurs at low water saturation, when the grains of the solids in the formation are coated; the resultant surface conduction effects are significant. The conductivity does increase with increasing water saturation, but not in a simple fashion. It is highly non-linear, and depends on whether the water table is rising or falling (Endres and Knight, 1991; Knight, 1991). The exact process is not of concern here, except to note that we normally deal in the relative changes in the electrical properties across a site, not in absolute values, because of this hysteresis in the physical properties. I note, as an aside, that the seismic properties, both acoustic and shear velocities, and the dielectric properties, which govern the propagation of radar waves, are similarly affected.

In addition to the presence of water, per se, the water quality has a marked effect on the electrical properties of the ground. The ground conductivity varies linearly with the pore water conductivity, which in turn increases linearly with the pore water quality, as measured by the total dissolved salts (TDS). The variation of pore water conductivity with water quality depends, however, on the particular ion in the water. The chloride ion, for example, is about 1.5 times more conductive than the bicarbonate ion (Fig. 2), two of the most commonly occurring natural ions. Because the addition of chloride ions will enhance the electrical conductivity, electrical and EM methods are particularly well suited to mapping and monitoring saline ground waters, whether the salinity is due to sea water incursion or due to leachate from a landfill site.

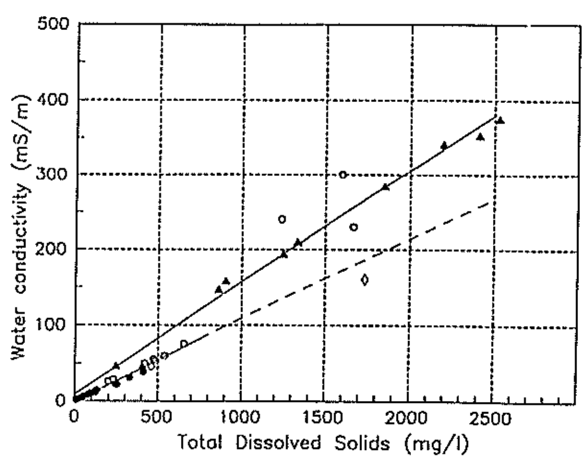

Figure 2. Pore water conductiv...
and leachates. The s...
the open symbols are
indicates the trend for
for chloride ions, whic
solid triangles are from

The GPR response depends on the dielectric permittivity, i.e. the extent to which the subsurface material can be polarised by an electromagnetic wave at high (radar) frequencies. The dielectric permittivity for water, however, is an order of magnitude greater than most other naturally occurring materials. The dielectric properties are thus, as for the electric properties, dominated by the presence of water, but are, in contrast, relatively unaffected by the water quality. Instead, the electrical conductivity, the ease with which a material can carry electric current, affects the attenuation of the radar signal, and the depth of penetration of radar is thus affected by the water quality. This is actually a useful property for radar surveys, since plumes of leachates from landfill sites will appear as blank zones on radar profiles.

Because the electrical properties affect the depth to which a radar survey will penetrate, an electrical or EM survey should always be carried out prior to a radar survey, so that the depth of penetration can be estimated in advance (Theimer et al., 1994). For the delineation of leachates, this is, again, an advantage, since the EM survey will yield the location of any leachate plume, and the radar survey lines can be positioned to cross the plume.

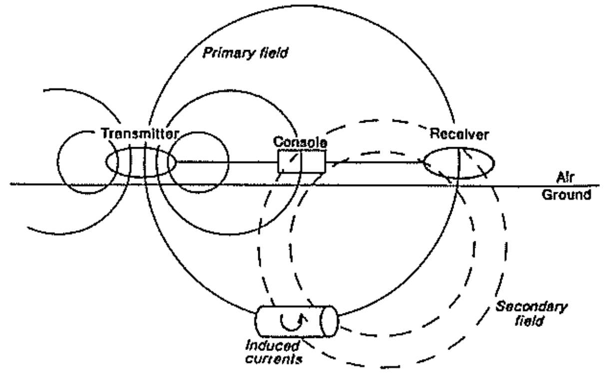

Figure 3. Schematic of the principles of EM surveys. An electromagnetic signal (the *primary field*) is generated at the transmitter, and generates a response both at the receiver and in the ground. The currents that are induced to flow in the ground generate, in turn, a *secondary field*, which also causes a secondary response in the receiver. The primary and secondary signals can be separated in the console, and the response of the ground can thus be determined.

Principles of EM

Electromagnetic methods work by generating induced currents in the ground. A time-varying magnetic field is generated at one antenna, the transmitter, normally a loop of wire. This signal travels through and interacts with the ground; the time-varying magnetic field generates electric currents within the ground. These *induced currents* produce another time-varying magnetic field, a *secondary* response, which is measured at the receiver, a second antenna that is essentially identical to the transmitting antenna. The entire system - transmitter, ground and receiver - can be considered as an electric circuit, and the various elements contribute to the response of the system (Fig. 3). The effect of the receiver is known, and the ground response can then be determined.

There are two basic EM systems used in environmental and groundwater work: frequency-domain EM (FEM), in which a signal of a given frequency is transmitted; and time-domain EM (TEM), in which the decay of a signal is measured. In the first case, the depth of penetration depends on the frequency, the electrical properties of the ground, and the separation between the transmitting and receiving antennas. If the separation is small, then the depth of penetration is normally about 1.5 times the separation. If the separation is large, then the depth of penetration is limited by the *skin depth*, the depth at which the signal has decayed to 37 % (1/e) of its original strength. For most systems used in engineering and environmental work, like the Geonics EM31 and EM34, the depth of penetration is limited by the separation of the antennas.

In FEM systems, the transmitted signal is often a single frequency, e.g. 9.8 kHz for the EM31, though some systems used for deeper work use multiple frequencies. The response at the receiver is altered both in *amplitude* (the size of the signal) and in *phase* (the signal is shifted in time). If the ground were not present, then the effect of the receiver would be to delay the transmitted signal so that the response measured by the receiver would be *exactly* out-of-phase with the transmitted signal (Fig. 4). Perfect (metallic) conductors have this effect on the signal. Imperfect conductors, like the ground, will not shift the signal as much. Superimposed on the shift caused by the receiver will be any shifts caused by the electrical properties of the ground, which will thus be a mix between a signal lined up with the transmitted signal, and one perfectly out-of-phase (Fig. 4). When added to the receiver shift, a very resistive ground will have little effect, and the net signal will be small and entirely out-of-phase. A large metal drum, on the other hand, will shift the signal almost as much as imposed by the receiver, so that the resultant is almost lined up with the transmitted signal.

The EM response in environmental and engineering surveys is usually expressed in terms of these two components, which are normally called the *real* or *in-phase* and *quadrature* or *imaginary* components. The quadrature component yields a measure of the electrical

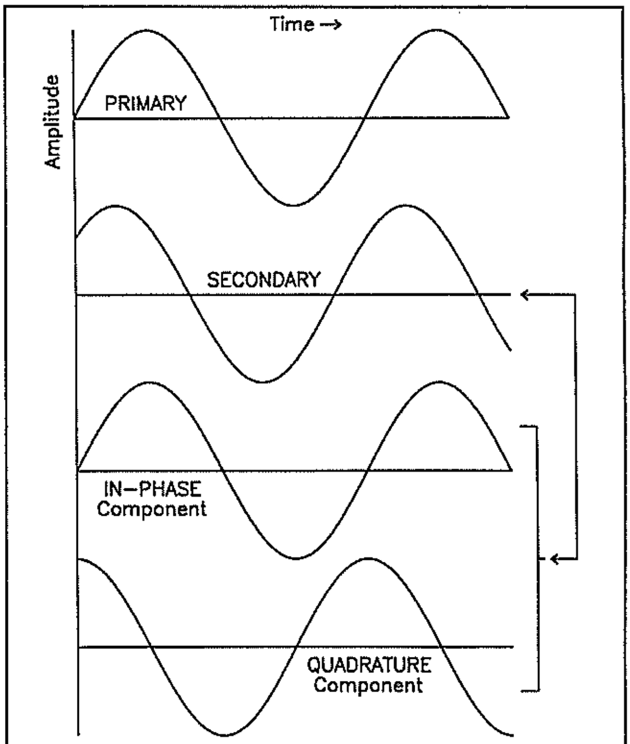

Figure 4. The primary signal (top) will be shifted in time (phase) and altered in amplitude during interaction with the ground and the receiver coil. This secondary response (second from top) can be separated into two components (bottom): the in-phase part, which lines up with the primary signal, and the quadrature part, which is exactly out of line with the primary signal.

conductivity of the ground, in the absence of a significant amount of metal, while the real component can be considered to be the metal detector mode.

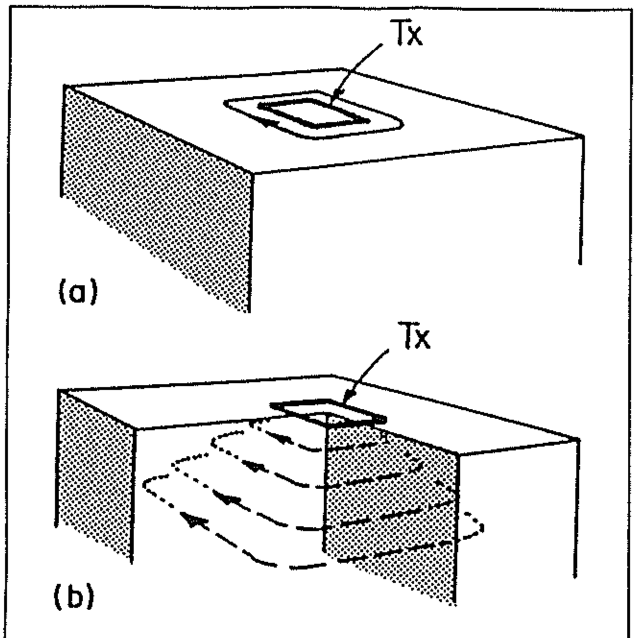

Figure 5. Principles of time-domain EM (TEM). (a) A steady-state current is generated in the ground adjacent to the transmitter loop, in response to a steady current in the loop. (b) When the current is switched off, the ground current gradually decays in strength and moves out and away from the transmitter. (From McNeill, 1990.)

In TEM, on the other hand, in the most common mode of operation, a steady current is set up in a loop laid on the ground. An electric current is generated in the ground in response to the current in the transmitter. The current in the transmitter loop is then switched off, which can be done quickly; modern transmitters can attain microsecond (10^{-6} sec) switching. The ground, of course, is not a good electric circuit. The current induced in the ground takes some time to "switch off". The current in the ground moves out and down away from the transmitter loop (Fig. 5, above, from McNeill, 1990), and the current strength gradually decays with time. The secondary field generated by the induced current similarly decays, both due to the decrease in the strength of the current flowing in the ground and also due to the movement of the current deeper into the ground away from the receiver. The rate of decay of the secondary field depends on the underlying electrical structure, and can be used to determine the thicknesses and conductivities of the ground immediately below the transmitter and receiver (Fig. 6, opposite, from Parasnis, 1986). No New Zealand examples using TEM are available at the time of writing; there is, however, a project currently underway using TEM to examine the underlying electrical structure of some selected Canterbury landfill sites.

EM methods are particularly sensitive to conductive layers and objects, and are less able than resistive methods to delineating resistive zones (e.g. Fitterman and Stewart, 1986). Thus, EM methods are well suited for mapping and monitoring areas of sea water invasion, zones occupied by leachate from landfill sites, and buried containers which may be filled with potentially hazardous waste.

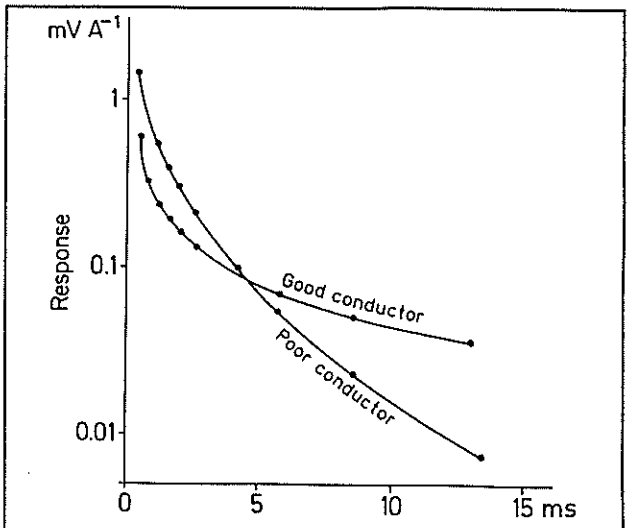

Figure 6. The current in the ground generates a secondary field which is detected at the receiver. At early times, the field is due to shallow currents; at late times, the field is due to deeper currents. The shape of the decay curve is controlled by the underlying structure. (From Parasnis, 1990.)

Ground Penetrating Radar Principles

Davis and Annan (1989) provide a good overview of ground penetrating radar. GPR uses a pulse of high-frequency EM energy to probe the ground. The signal moves through the ground as a wave pulse, in a manner that is similar to the propagation of seismic energy (Fig. 7, next page); the velocity, however, is that of light. GPR methods are able to detect or resolve objects that are larger than one-quarter of the wavelength of the radar wave, λ, which is related to the velocity, V, and frequency, f, as:

[2] $\lambda/4 = V/(4f)$

Layers as thin as $\lambda/8$ or $\lambda/12$ can be resolved. For example, typical velocities in saturated soils are of the order of 0.10 m/ns (metres/10^{-9} sec), and for a frequency of 100 MHz, objects as small as 25 cm can be resolved, and layers as thin as 10 cm can be detected.

Energy is lost through attenuation and scattering. Scattering occurs from objects that smaller than, but nearly the same size as, the minimum detectable dimension. As noted previously, the electrical properties affect the depth of penetration. The signal strength, A, falls off exponentially in the ground as:

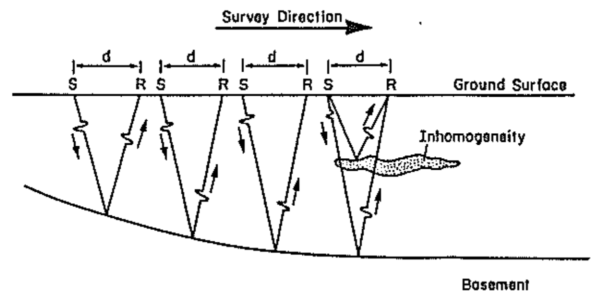

Figure 7. Survey geometry for the common offset GPR and seismic survey mode. The source (S) and receiver (R) are kept at a constant separation (d), and are moved along a given profile at constant steps. The two-way reflection travel time (down and up) depends on the depth to the interface or object.

[3] $A \sim e^{-\alpha z}$

where α is the attenuation factor, the rate at which the signal decays, which is usually expressed as dB/m. The attenuation, in turn, depends linearly on the electrical conductivity, s, and on the inverse square root of the velocity, V, as:

[4] $\alpha = k\sigma/\sqrt{V}$

where k is a constant. However, the variation in the electrical conductivity is many orders of magnitude greater than the variations in the velocity, so that the electrical conductivity dominates the depth of penetration of GPR. Thus, if the background electrical properties are known, then the depth of penetration can be predicted, as illustrated in Figure 8 (from Theimer et al., 1994).

Topographic Effects

Both EM and radar can be affected by the topography of a site. In the case of radar, the effect is direct - the underlying reflectors appear to dip since the reference level is the surface where the GPR survey is carried out. We thus also attempt to carry out a topographic survey along the geophysical survey lines, and then adjust the survey lines appropriately. For surveys of contamination, we can determine the position and depth of the contamination without the need for topographic correction, and the profiles presented here have not been corrected for changes in elevation along the survey lines.

For EM, on the other hand, the effect due to topography is more subtle. As the survey line climbs a sand dune, for example, the water table will tend to lie at a greater depth relative to the EM survey instruments, and the apparent electrical conductivity will decrease. If a leachate plume is present beside and under a portion of the dune, the plume will not appear to be connected across the dune, since the associated electrical anomaly

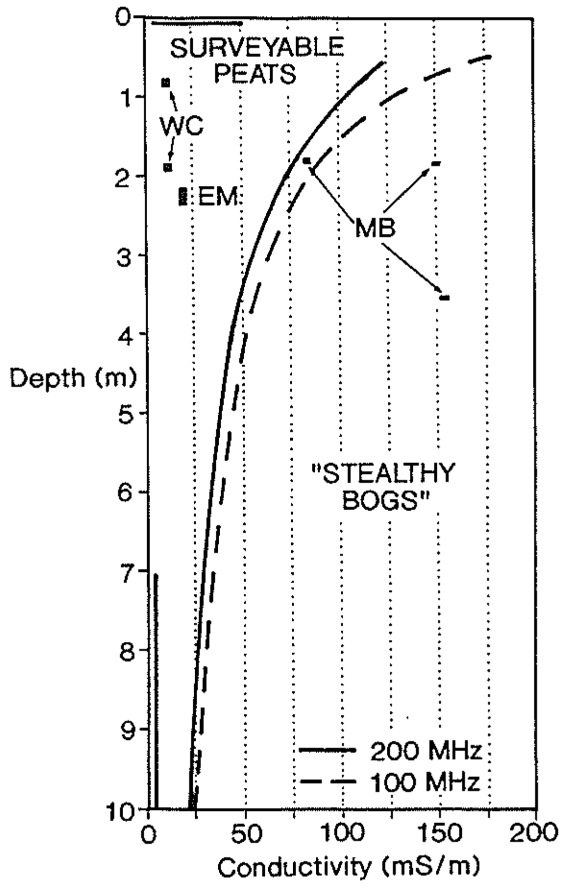

Figure 8. The depth of radar penetration is dependent on the electrical properties. Shown here are the calculations for two frequencies, 100 and 200 MHz (curves), and for observations (solid labelled boxes) from a set of peatlands. (From Theimer et al., 1994.)

will be disconnected. The apparent conductivity must therefore be corrected for elevation for a correct interpretation to be made. The general form will indicate the presence of a leachate plume, but may not correctly yield the extent and position.

Monier-Williams et al. (1990) have examined the problem of topographic correction in detail, and have proposed an empirical approach, which we use here. The basic procedure is as follows: The survey should cover an area large enough to encompass regions where there is no contamination, so that the electrical readings include the background, uncontaminated conductivity, s_B. The conductivity is plotted as a function of elevation, z, only, regardless of its lateral position. A background electrical conductivity level, $s_B(z)$, should be readily apparent. The electrical conductivity is *normalised* with respect to the background conductivity, by dividing the reading at a given location by the background conductivity for the elevation of that location. The result is expressed in decibels (dB), so that the conductivity corrected for topography, σ_C, is:

[5] $\sigma_C = 20 \log_{10}(\sigma(x,y,z)/\sigma_B(z))$

A value of 0 dB indicates that the reading is the same as the background conductivity, whereas a value of 6 dB indicates that the electrical conductivity is twice the background value. Specific examples are presented with the case histories, below.

RECENT CASE HISTORIES

Delineation of leachate plumes at Kaiapoi landfill

The local Kaiapoi landfill site is located on beach sands, 3.5 km from the coast. The surficial unconfined aquifer is not used as a potable water supply, and is underlain by a confining layer which protects the deeper aquifers. A leachate plume had been detected adjacent to the Kaiapoi landfill, using galvanic resistivity methods (Broadbent, 1992). That is, the electrical properties of the ground were determined using direct current injection into the ground. The procedure is simple but time consuming. Followup surveys were carried out using a Geonics EM31 soil conductivity meter, owned by the Department of Geology at the University of Canterbury, and a Sensors and Software pulseEKKO IV ground penetrating radar system, rented from the Australian distributor. The EM and GPR surveys were completed rapidly, and the results agreed with the general form of the resistivity results.

While the raw EM31 apparent conductivities indicate the presence of the leachate plume from the first stage of the landfill site (Fig. 9, from Armstrong, 1993), there are other areas where the conductivities are anomalously high or low, for example at the southern end of the survey area where the conductivities. Note also the slightly elevated readings to the east, bounded on the west by a zone of lower conductivity. The lower conductivities follow the trend of a 2 m high sand dune.

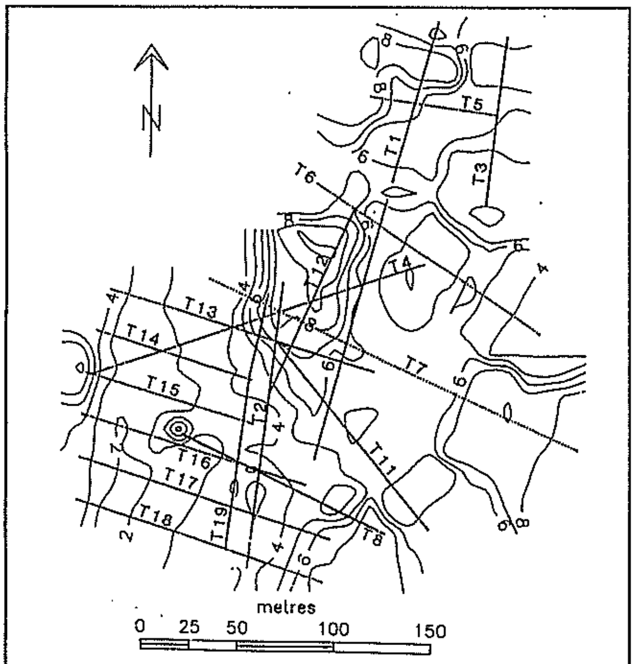

Figure 9. Raw EM31 apparent conductivity adjacent to the Kaiapoi landfill site. Note the conductivity high in the centre, suggestive of a plume. However, there are other highs and lows scattered about the area, largely due to topographic variations. The survey lines are indicated. (From Armstrong, 1993.)

The effect due to topography is clear (Fig. 10, from Armstrong, 1993), and the corrected EM31 results better delineate the extent of the leachate plume (Fig. 11, From Armstrong, 1993). The boundary of the plume appears to be indicated by the 4 dB contour line (shown bold), that is the plume electrical conductivity readings are 60 % above the background values. The additional anomalous high and low readings are largely due to topographic variations; for example, the areas of higher conductivity to the south and to the east are due to topographic lows, and the recorded values are similar to the background electrical conductivities in those areas. The low conductivity trend along the sand dune masks a second plume, which may originate from the current stage of the landfill; this second plume appears to follow a subsurface channel.

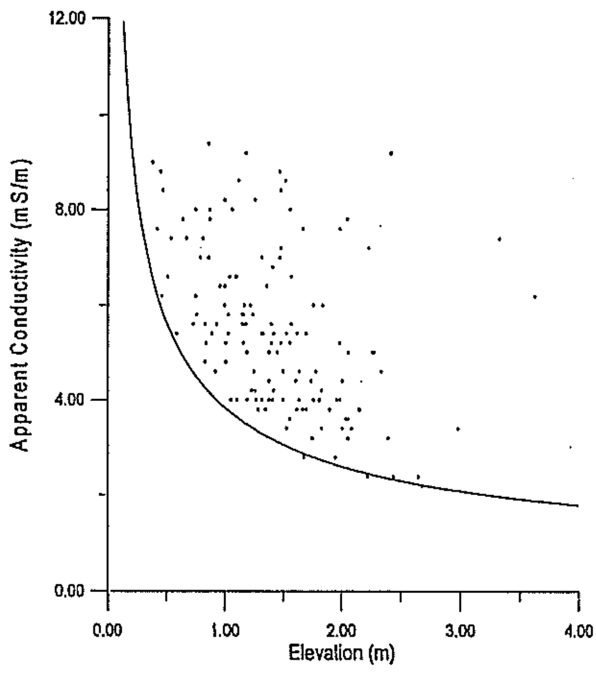

Figure 10. Plot of EM31 apparent conductivity (in mS/m) vs elevation (in m), showing clearly the background base level (solid curve). This base level is then used to calculate the corrected EM31 values, using the approach outlined by Monier-Williams et al. (1990). (From Armstrong, 1993.)

GPR surveys were also carried out along the lines shown in Figure 10. Line T1 was split into two because of fences surrounding the landfill site. One segment extends from north of line T6 to the southern end of line T1, crossing the edge of the plume as defined by the corrected EM31 results. We thus expect the radar signal to be significantly reduced as the line crosses the leachate plume. This is indeed the case (Fig. 12, bottom, from Armstrong, 1993); the reflections due to the subsurface layering are readily apparent on either side of the plume, but are strongly attenuated where the plume is present.

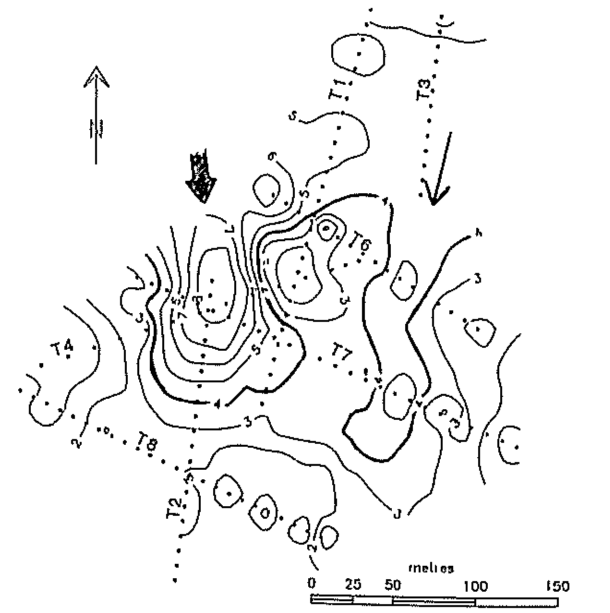

Figure 11. Data from Figure 9 corrected using the base level shown in Figure 10. Note the elimination of the highs and lows due to topographic variations, and the isolation and enhancement of the leachate plume. A by-product of the analysis is the identification of a second, weaker plume to the east originating from the current landfill operation. The locations of the topographic control points are indicated. (From Armstrong, 1993.)

Delineation of leachate plumes at Burwood land

Previous studies of the Christchurch Metropolitar municipal landfill site near the coast at Burwood indicated the presence of a leachate plume which would move toward the coast at a rate of 2.3 to 7.4 m/month (Close 1991, 1992). The Burwood landfill is situated on dune sands; the surficial unconfined aquifer is underlain by silts and clays which serve to protect the deeper aquifers which are used for the regional water supply. The groundwater flow is to the east towards the coast; if leakage occurs through the confining layers, then flow is directed upwards towards the surface, since the deeper waters are under pressure. Thus any leachate will flow east towards the coast and cannot enter the regional water supply. There is still some concern about the extent of leachate, however, since the area is being used and developed for recreational use. A suite of monitoring wells were established adjacent to the first stage of the landfill, and along the coastal firebreak. The geophysical surveys were run to supplement the monitoring wells, and to delineate the position of the plume at the time of the surveys.

An EM31 survey was carried out across the area adjacent to the former and current landfill sites. In addition to topographic effects, we were concerned with any interactions with the tides along the coast. A suite of repeated measurements were taken along the coastal firebreak 100 m from the coast, along a trail 50 m from the coast, and along the high tide mark. To our surprise, there is essentially no tidal influence observed in the readings

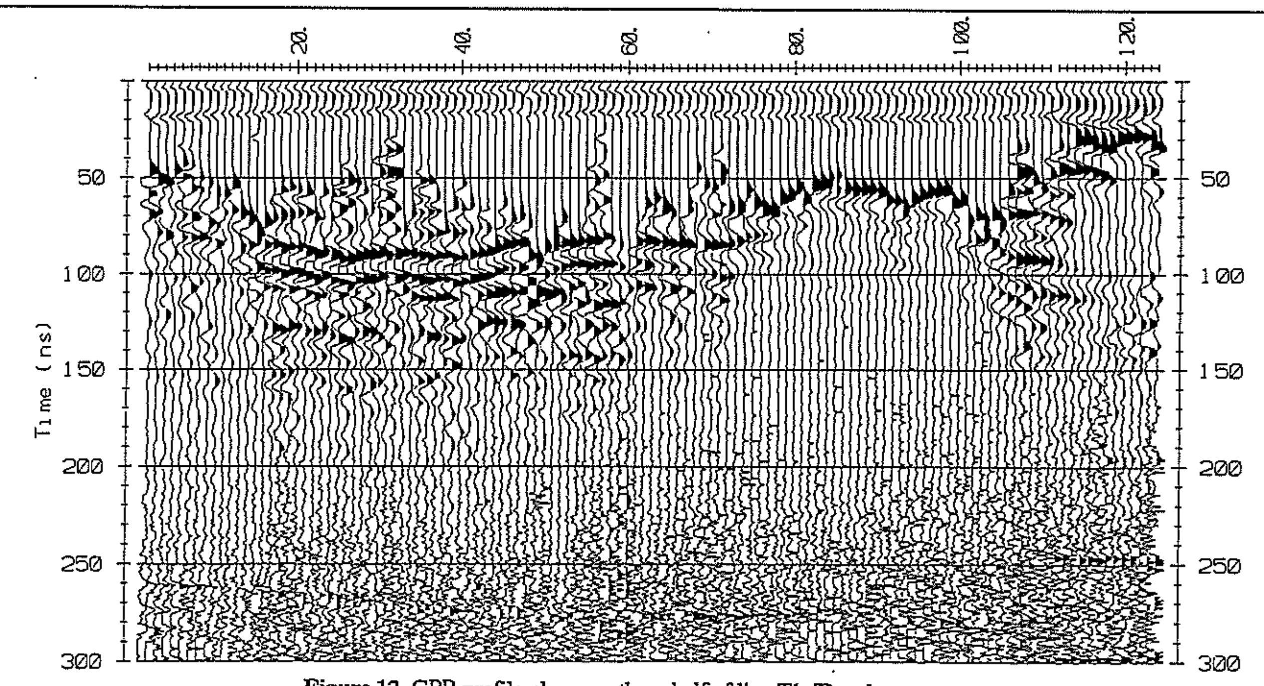

Figure 12. GPR profile along southern half of line T1. The plume appears as a zone of weak reflectors below the water table. The stratigraphy that is present on either side of the plume does continue through it, but the energy is attenuated by the plume. (From Armstrong, 1993.)

along the coastal firebreak (Fig. 13, from Armstrong, 1993). There appears to be a channel that provides a connection to the coast; whether the channel continues far enough inland to provide a conduit for leachate is not yet determined. There is also almost no effect observed along the high tide line; the coastal beach sands appear to remain saturated, except for a relatively thin surficial layer, and thus are not affected by the tidal cycle. Only the line along the trail which lies 50 m in from the coast shows any consistent tidal influence.

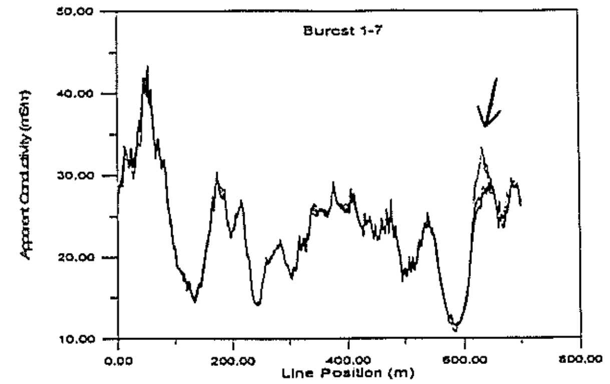

Figure 13. Plot of successive records of EM31 apparent conductivity along the coastal firebreak road near the Burwood landfill site. The readings were taken during a tidal cycle, from low to high tide. Note the lack of change except near location 650, where there may be a channel connection to the shore. (From Armstrong, 1993.)

The location and extent of the leachate plume is apparent in the raw EM31 data (Fig. 14, from Armstrong, 1993), but the details are uncertain, given the dune topography that is abundant across the area. In particular, we were interested in the constriction in the contours of apparent conductivity in the southwestern corner of the survey area, along lines Burwood 1 and 2 near the boundary of the first stage of the landfill. Topographic surveys were carried out along a subset of the survey lines; many of the shorter survey lines could not surveyed because of low dense scrub. As for the Kaiapoi survey, the relationship between the apparent conductivity and the topography is well defined, and the corrected EM31 results provide better control and the lateral extent of the plume (Fig. 15, from Armstrong, 1993). The leading eastern edge of the plume is not well-defined, since we do not have good topographic control in that area. However, the 25 mS/m contour (Fig. 14) appears to provide a good approximate location for the leading edge of the plume. Calculated flow rates are then 3.2 to 6.3 m/month, in very good agreement with but more tightly constrained than the results of groundwater modelling (Close, 1992).

The corrected EM31 results also appear to confirm the presence of a constricting channel in the southwestern corner of the area. The contours of the anomalies close

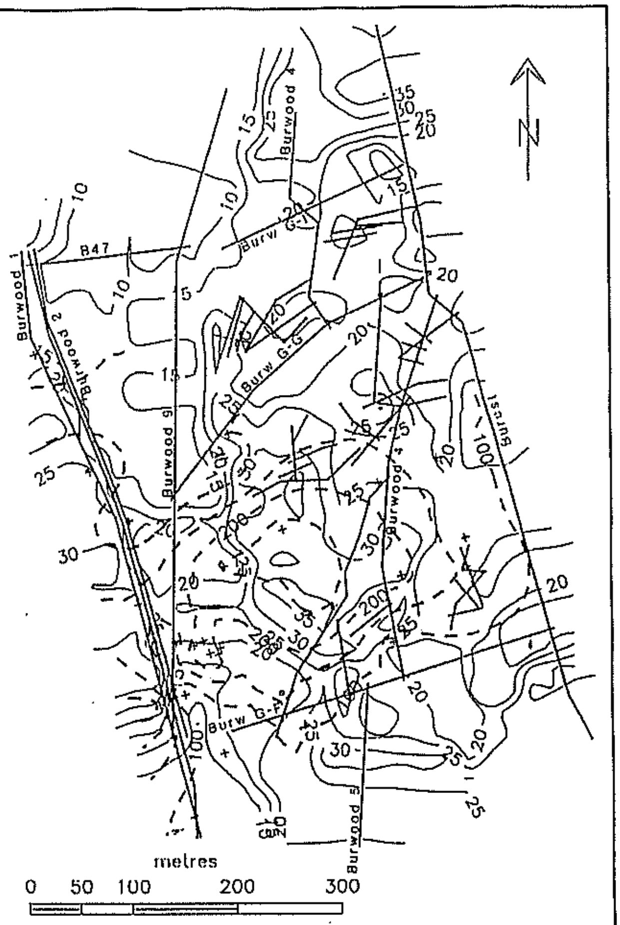

Figure 14. Raw EM31 apparent conductivity (in mS/m) from Burwood study area. Survey lines are indicated as bold solid lines. The water sampling wells are shown as crosses (+), and the contours of pore water conductivity (in mS/m) are shown as bold dashed contours. Note the good correspondence between the two data sets, in spite of the fact that the measurement locations rarely coincide. (Adapted from Armstrong, 1993, and Close and Budhia, 1993.)

in at first, before opening up again to the east, suggesting that leachate flows east through the channel and then spreads out to the north and south past the channel. In addition, the location of the plume is almost centred between the monitoring wells that are located along the coastal firebreak. In other words, the leading edge of the plume would have passed between the wells before any indication of the plume would have been detected in the monitoring wells. Based on the results of the geophysical surveying, new monitoring wells have been put in place, and the results of water sampling confirm the results of the geophysical surveys (Figs. 14 and 15, and Close and Budhia, 1993).

SUMMARY

Geophysical methods, generally, and electromagnetic methods and ground penetrating radar, specifically, should be used to delineate anomalous areas in advance of drilling holes and sampling wells. EM responds to

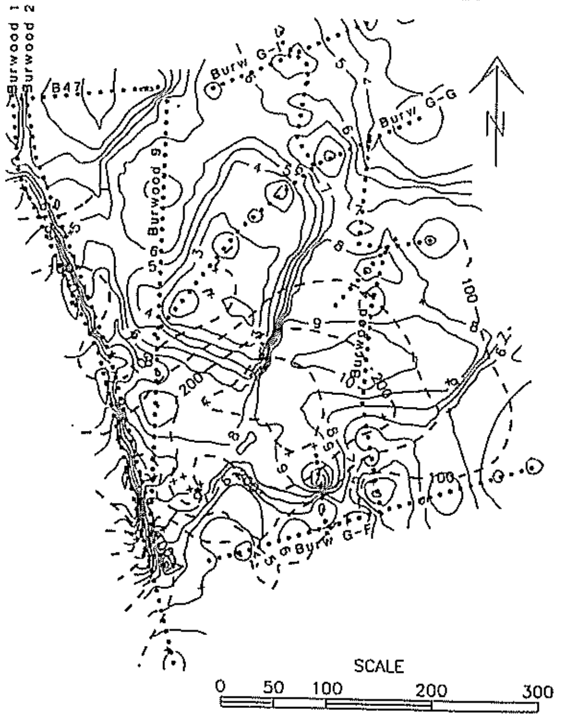

Figure 15. Corrected EM31 (in dB) for the Burwood study site. Only those lines with topographic control were used; a by-product of this is the lack of control on the leading edge of the plume, which is better defined by the 30 mS/m contour in Figure 14. Again, the pore water conductivity results of Close and Budhia (1993) are shown superimposed (bold dashed contours). (Adapted from Armstrong, 1993.)

changes in the electrical properties of the ground, and the depth of penetration of GPR is strongly controlled by the electrical properties; plumes of leachate or sea water will significantly affect the EM response and the appearance of GPR reflectors. This allows the extent of contamination to be determined accurately, and the level of contamination can then be established using samples taken from boreholes that have been placed to maximum advantage. Fewer holes placed more accurately save time and effort. Geophysical survey techniques provide the non-invasive, non-destructive first step in mapping contamination due to leachate or sea water incursion, and can be used to monitor the movement of leachate or sea water plumes.

ACKNOWLEDGMENTS

It has been my honour and pleasure to work with and to supervise many good people over the years, and much of the material here is distilled from our joint efforts. In particular, I want to note the contributions of Mark Armstrong (a former Honours student), Mike Broadbent (formerly of the Institute of Geological and Nuclear Sciences), Murray Close (of the Institute of Environmental Health and Forensic Sciences), Maurice Melis (currently in his Honours year in Environmental Science), Viv Smith (currently with the Canterbury Regional Council), and Brian Theimer (a former University of Waterloo M.Sc. student). Funding for the work was supplied by the Department of Geology of the University of Canterbury and by the Canterbury Regional Council.

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SOUTHLAND PESTICIDE DUMPS - LOCATING BY GEOPHYSICAL METHODS

T M McConachie
Woodward-Clyde (NZ) Ltd
Christchurch

SYNOPSIS

Delineation and characterisation of hazardous waste dumps can be a difficult, time consuming and hazardous task particularly where the specific location of the site is unknown or poorly defined. Depending on the physical characteristics and nature of the host soils, waste dumps may generate geophysical anomalies detectable by appropriate geophysical survey techniques. This paper presents a case history of the application of magnetic and electromagnetic survey techniques employed in the detection and delineation of two hazardous waste dumps in Southland and highlights the advantages of non-invasive exploration techniques.

BACKGROUND

In 1961 New Zealand introduced regulations banning the use, without a permit, of dieldrin and related chemicals. Under the direction of the Ministry of Agriculture and Fisheries, approximately 530 tons of pesticide were collected and stored in 45 locations throughout the country, some of which was ultimately disposed of in dump sites.

Preliminary inquires by the Southland Regional Council led to the identification of five possible pesticide dump sites in two areas of Southland. The nature and location of these dumps was based solely on anecdotal information supplied by persons involved in or witnessing the dumping operations.

Three sites were reported in the Te Anau Basin and two in the Mokoreta area. Four of the sites are located on former Lands and Survey farm development blocks. The fifth, known as Mokoreta was created in the early 1980s and is located in a timber production forest.

PRELIMINARY INVESTIGATION

Two dump sites, one located at Mokoreta north of Invercargill and the other near Manapouri, were considered of particular concern due to the nature of their locations.

Flaxy Creek
At Flaxy Creek, the dump was believed to be located within 15 to 20 m of a domestic and stock water supply well and therefore posed a significant immediate hazard.

Containers of pesticide were reportedly brought to the farm at night and the contents sprayed by light aircraft onto the surrounding land. The empty containers were then disposed of in a pit in or near a gravel quarry located on the property. The quantities of pesticides brought to the site were unknown, but were believed to be in excess of several truck loads.

Mokoreta
The dump site was believed to be located on the edge of a forest access road and could potentially be disturbed during future logging operations. Secondary growth native bush and over planted eucalypt trees surround the site. Pesticides sourced from a nearby nursery were dumped at the site around 1980-81.

A trench was reportedly excavated in clay and the drums were punctured by rifle shot prior to burial. Eyewitnesses to the dumping were unable to precisely locate the dump site during site visits due to significant vegetation growth in the intervening years. However, several possible locations were identified, one of which was located 80m from the road in dense native bush.

PHYSICAL SETTING

Flaxy Creek Site
The suspected location of the dump site was on a small terrace adjacent to Flaxy Creek which is bounded by a 4 to 5m high scarp, created during excavation of the quarry. A farm access track forms the northern boundary of the quarry as shown in Figure 1. Semi-permanent ponding of surface water in the swampy area around the base of the scarp indicates poor drainage through the silty humus surface soils which were well vegetated with pasture grass with some tussock in the swampy areas. Subsoils in the search area consisted of loose sandy coarse gravels to about 1.5m depth underlain over most of the site by very dense silty gravel.

Mokoreta Site
The suspected location of the dump site was in an area of dense bush, including natives and exotic plantings, located on a hillside above farmland in an area of relatively deep soil up to 1.2m deep. Highly jointed sandstone bedrock underlies the soil layer. Surrounding this localised area of deep soil, the bedrock is overlain by a saturated layer of topsoil typically 0.1-0.3m deep. The bedrock exposed in excavations was highly jointed at the bedrock surface becoming significantly less jointed at a depth of 0.5m below the bedrock surface. The bedrock surface appears to dip toward the northwest and northeast in the vicinity of the dump site. Several tributary streams run in gullies through the bush and onto the farmland several hundred metres north of the site.

GEOPHYSICAL SURVEY

Introduction
Preliminary historical information indicated that pesticides had been disposed of in both metallic and nonmetallic containers at both locations. The geophysical methods chosen therefore had to be effective at detecting both types of container or ground disturbances associated with their burial.

The information gathered during the review of historical information was used to delineate initial target areas for site investigations at the Flaxy Creek and Mokoreta sites. These initial target areas were surveyed using magnetic and electromagnetic geophysical methods in order to locate buried chemical containers and disturbed ground associated with the dumping at these sites.

Methodology
High resolution magnetic and electromagnetic surveying is non-invasive and therefore well suited to the detection of hazardous substances. The field equipment used in this investigation consisted of a Geonics EM 34 electromagnetic transceiver and an EG & G Geometrics G856 portable proton magnetometer.

The distance between measurements, known as the station spacing, was selected based on the anticipated size and depth of each dump, as assessed from historical information. A station spacing of two metres between magnetic measurements and four metres between electromagnetic measurements was considered appropriate.

The initial target areas were gridded and survey lines pegged prior to geophysical surveying.

All magnetic data were corrected for diurnal variation and contoured for final presentation and interpretation.

No corrections were required for the electromagnetic data which were contoured prior to interpretation.

The geophysical surveys were carried out in two stages at each site. During the first stage both magnetic and electromagnetic surveys were carried out with the magnetic method providing the best results and the electromagnetic method only confirming these results. The dump sites were not located during the first stage survey. During the second stage survey the magnetic equipment only was used and the dump sites were located.

Flaxy Creek Survey
Historical and anecdotal evidence indicated that a single dump existed at the Flaxy Creek Site. The first stage of the geophysical surveying at the Flaxy Creek Site was carried out over an area of 1,500 m^2 centred on the farm track which was the initial target area delineated from the historical and anecdotal evidence. The second stage survey covered an additional area of 3,500m^2 and was carried out in response to additional historical information obtained during the investigation.

First stage survey
The first stage survey area was bounded to the west by Flaxy Creek and to the east by a line parallel to and approximately eight metres to the west of the top to the quarry face. To the north and south the area was bounded by farm fences and a survey line respectively. The first stage survey area covered an area between Lines 0 and 38, with two metre spacing between stations (Figure 1).

Assessment of the contoured magnetic and electromagnetic data presented on Figures 1 and 2 reveals several significant magnetic anomalies, all but one of which were associated with farm fences including isolated metal fence posts (railway iron) characterised by intense short range anomalies along the northern boundary of the survey area. The remaining anomaly centred at Line 8, Station 28 indicated the presence of a significant mass of buried metal. This location was investigated by trenching with an excavator and found to be a buried car body. No evidence of a chemical dump was noted at this location.

Second stage survey
The second stage survey covered an area between Lines 0 and -78, with two metre line spacing between Line 0 and -38 and four metre line spacing between Lines -38 and -78 to allow the remainder of the quarry floor to be surveyed rapidly.

Assessment of the contoured magnetic data presented on Figure 1 reveals two significant anomalies, centred at Line -16, Station 34 and Line -12, Station 16. The former was associated with an old pump shed which contains the original farm well and associated pumping equipment. The latter anomaly approximately 10m to the east of the first could not be reconciled with any obvious surface features and indicated the presence of a buried metallic mass. This site was investigated with an excavator and found to contain buried chemical drums.

West of Line -38 the magnetic field was relatively uniform and varied by only 40 nanoTesla (nT). This indicates that buried metallic containers do not exist in this area.

Mokoreta Site
Historical and anecdotal evidence indicated that a single dump site existed at the Mokoreta Site. The first stage of the geophysical surveying at Mokoreta was carried out over an area of 1,100 m^2 centred on the reported location of the dump site. The second stage survey covered a much larger area of 30,000 m^2 to the northeast of the first area and consisted of a roving magnetic survey carried out in response to additional historical information obtained during the investigation. The stage one area surveyed at the Mokoreta Site is shown on Figure 3.

First stage survey
The first stage survey revealed only one small anomaly centred on Line 4, Station 26 - 28 shown on Figure 14. This was revealed by excavation to be a wire rope.

Second stage survey
Due to both the size and dense bush growing on the additional area requiring surveying it was not feasible within the budgetary constraints of the project to grid the second stage survey area. Instead, those areas which had been indicated to be the most likely locations were systematically surveyed. The data revealed a generally uniform magnetic field gently increasing to the west. A single anomaly was detected approximately 80m from the forest access road and indicated the presence of a small metallic mass buried just beneath the ground surface. This site was investigated by excavator and found to contain the dumped pesticide containers.

EXPLORATORY EXCAVATION OF THE DUMP SITES

The outer perimeter of the pesticide dumps were inferred from the contoured geophysical data and pegged out. The aim was then to excavate sections of the edge of each dump to assess the placement and condition of the containers. Excavation using a tracked excavator commenced several metres away from the inferred perimeter of the dump and proceed toward the centre of the geophysical anomaly. The excavated soil was carefully logged and assessed for signs of contamination. Excavation ceased when the side of the dump or chemical containers were found. An attempt was then made to define the base of the dump by excavation. Excavation findings were correlated to the interpreted geophysical data as excavation proceeded. Through this approach exploratory excavation ceased when good correlation was achieved reducing the requirement for unnecessary hazardous excavation.

Flaxy Creek Site
Each significant anomaly detected during the two stage geophysical survey was investigated by exploratory excavation.

The anomaly detected in the first stage geophysical survey was excavated and found to be a carbody. It was suspected that the carbody may have been placed over a chemical dump and therefore exploratory excavation was carried out around the carbody and the carbody was lifted up to allow excavation underneath.

Exploratory excavation of the dump site commenced from approximately four metres to the north of the site working towards the dump using an excavator to dig a trench to about 0.5-1m depth.

On reaching the dump site, the excavation continued at up to one metre depth around the full perimeter of the dump site in order to determine its lateral extent. The dump is eight metres long and four metres wide aligned approximately north-south. The dump appeared to extend only to about 0.5m depth below ground level and was covered by a 0.1m layer of topsoil.

Metal chemical containers were uncovered and found to be empty and extremely rusted and broken down.

Early anecdotal evidence indicated the possibility of the dump being located beneath the farm access road. Although no geophysical anomalies were evident at this location, excavation was carried out along three traverses perpendicular to the road. No evidence of any dumping activity were noted in any traverses and together with the absence of any geophysical anomalies, excavation below the 1.8m wide road was not warranted.

The remaining geophysical anomalies, around obvious surface features, were investigated in order to confirm that these features were not masking another chemical dump. No evidence of any chemical dumping was detected in any of these excavations.

Mokoreta Site

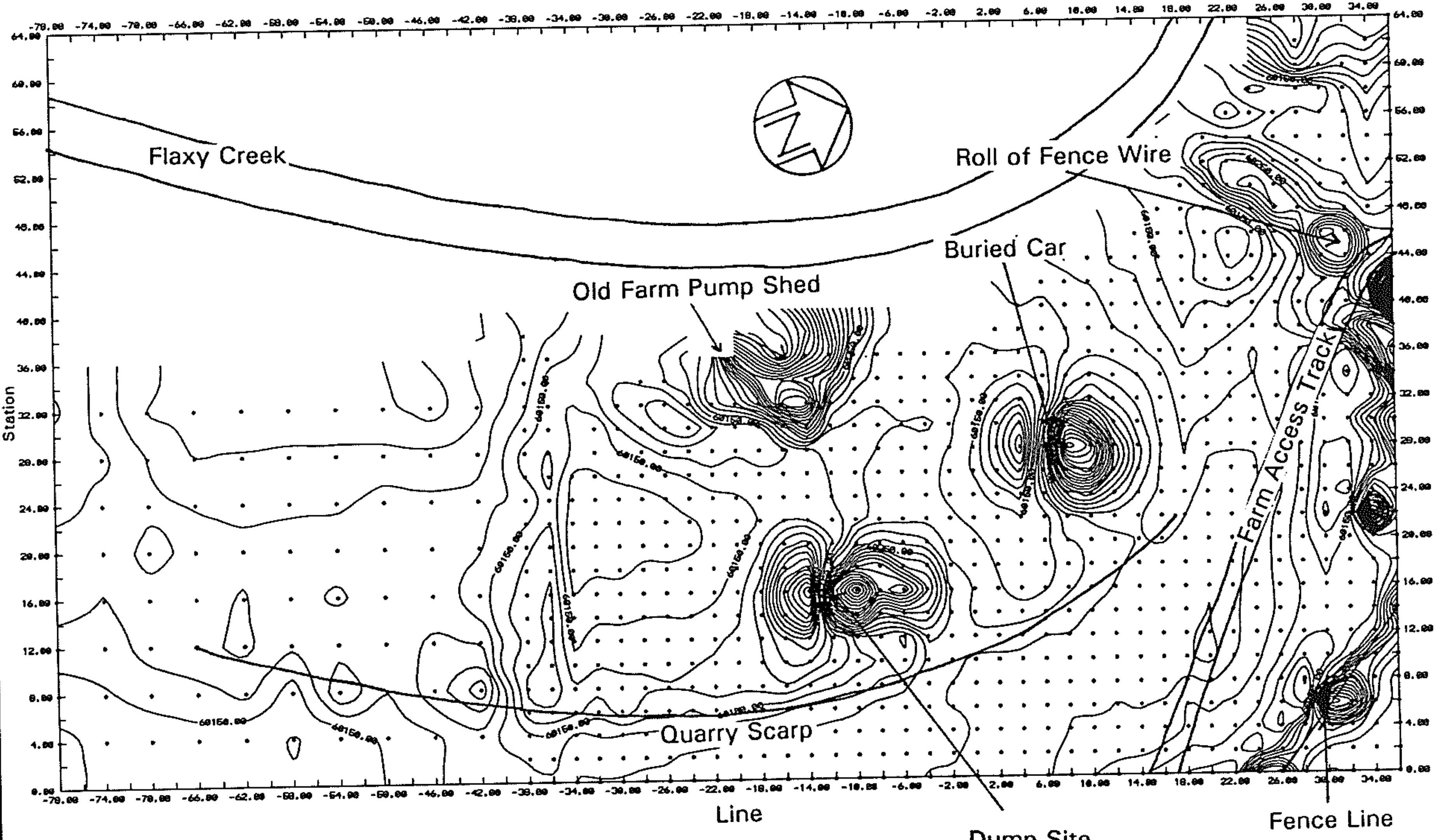
Exploratory excavation of the dump site commenced from three equally spaced locations approximately eight metres away from and working towards the centre of the geophysical anomaly. The excavation was carried out using an excavator to dig a trench about one metre deep. Highly jointed bedrock was intersected at a depth of approximately 1.5m below groundlevel.

Four 20 litre metal drums were found buried about one metre deep at the junction of the three trenches. The drums had been punctured during placement but were only slightly corroded and some labelling was still legible, including one drum which was labelled as Aldrin.

CONCLUSIONS

Magnetic and electromagnetic geophysical surveying techniques have been successfully utilised to locate buried hazardous chemical containers. The use of geophysical surveying has allowed relatively small targets to be located rapidly within a relatively large search area. Had invasive excavation been utilised only, the locating of the Flaxy Creek pesticide dump would not have been possible without substantially greater disturbance to the area. At Mokoreta it is highly unlikely that the pesticide dump would have been located at all.

The use of geophysical methods has resulted in significant savings in exploration time, hazardous excavation, reduced disturbance to the environment and ultimately represents significant financial savings.



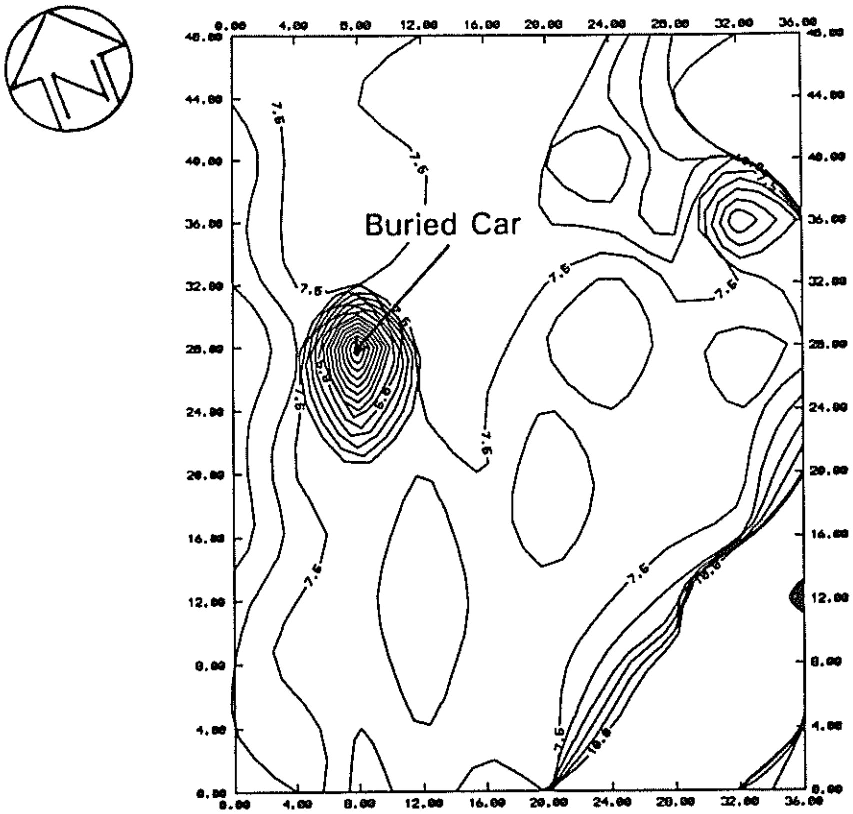
Legend: ———— Total Magnetic Field Contour (nT)
• Station

Note: Contour Intervals = 20 nT

JULY, 1993

MAGNETIC SURVEY - FLAXY CREEK SITE

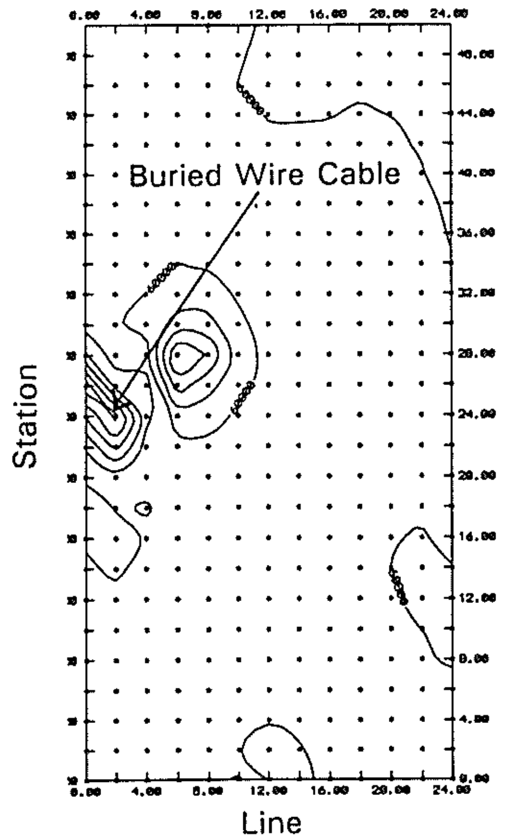
FIGURE 1



Legend: — 7.5 — Ground Conductivity Contours (ohm.m)
Note: Contour Interval 0.5 ohm.m

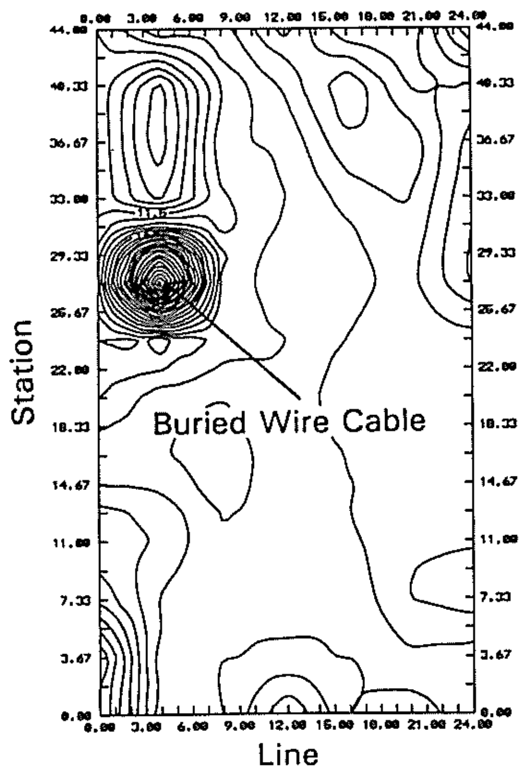
JULY,1993

EM SURVEY - FLAXY CREEK SITE FIGURE 2



Legend : —— Total Magnetic Field Contours (nT)

. Station



Legend : —— Ground Conductivity Contours (ohm.m)

. Station

Note : Contour Interval 0.5 ohm.m

JULY,1993

MAGNETIC AND EM SURVEY - MOKORETA SITE

FIGURE 3

REMEDIATION OF CONTAMINATED SOIL IN NEW ZEALAND

Dr M L Plested
Worley Consultants Limited
Auckland

SYNOPSIS

New Zealand industry and regulators are beginning to come to grips with the ramifications of the Resource Management Act and the Occupational Safety, Health, and Employment Act. One of the ramifications of these acts is that the potential adverse effects to human health and the environment represented by contaminated soils must be addressed. Once those effects have been shown to be, or could potentially be, present at a site, options for remedying or mitigating those effects must be investigated. This paper discusses what technologies are available in New Zealand to mitigate effects created by contaminated soils and how to go about determining which is best suited for a particular site. Technologies for controlling effects created by contaminated soils are not limited to treatment to reduce concentration or mobility of contaminants. Finally, technologies that may be needed in the future are discussed.

WHEN IS REMEDIATION NECESSARY?

At present New Zealand contaminated site owners and occupiers have two ways of determining the need for remediation on their site.

The first is the use of numerical criteria such as the much publicised Dutch environmental criteria. However, the use of such criteria can be difficult. If used without care, numerical guidelines can result in a decision to remediate based on contaminant levels that are either too conservative for a given site, or not conservative enough. This arises from the fact that the guideline contaminant levels were set for a receptor and exposure pathway combination which may not exist at the site. The Dutch criteria, for example, were developed to be protective of ground water because of that country's reliance on this resource.

The contaminant concentrations required to protect ground water quality are generally much lower than that required to avoid effects through say skin contact or dust inhalation. New Zealand populations rely on ground water resources in only a few regions. Therefore, where ground water protection is not required to protect an existing or potential future use, the Dutch criteria may be too restrictive.

The blind use of guideline criteria can therefore result in an unnecessarily high cost for remediation and can also result in a decision to do nothing because the site owner/occupier can't afford the price of environmental responsibility. An even more disastrous possibility is that the contaminant level is not protective enough.

The second method is a site specific risk assessment. This process is recommended in the Australian and New Zealand Environmental Criteria Committee *Guidelines for the Assessment and Management of Contaminated Sites* (ANZECC, 1992). The site specific risk assessment process allows the site owner/occupier the opportunity to identify and address the actual effects presented by their site.

To understand when remediation is necessary, and identify the full range of remediation options available in New Zealand requires an understanding of how unacceptable human or environmental effects are identified on a contaminated site. This is the fundamental objective of the risk assessment approach.

For an effect to occur, three elements must be present at a site. Each of these is discussed below.

Contaminants

Contaminants must not only be present, but must be present at concentrations that can potentially produce an adverse effect.

Exposure Pathway

The concentrations required to produce an adverse effect are different depending on the exposure pathways present or possible at the site.

The potential for an exposure pathway to exist depends on several factors including :

Site use	Surrounding land use
Topography	Soil type and geology,
Drainage	Hydrogeology

Physical and chemical properties of the contaminant

For example, consider a site with near surface soil contaminated by a carcinogenic compound with low volatility and high adsorption to soils. The exposure pathway of potential concern may be skin contact and inhalation of contaminated dust. This site will have a potentially lower allowable contaminant concentration thana site where the same contaminant is buried but at the same concentration. In the case of the buried contaminant, the exposure pathway of potential concern is not present.

Receptor

Assuming the contaminant and exposure pathway are present at a site, for a potentially adverse effect to occur there must also be either a human or environmental receptor. If the site is no longer in use and infrequently visited then there is minimal potential for an adverse effect because the exposure pathway is not complete.

When evaluating potential risks represented by a site, the future use of that site must also be considered. If the land use is to change, or could potentially change, as is the case for inner city industrial sites being redeveloped as residential, future exposure pathways must also be considered.

With respect to environmental receptors, the receptor may be a sensitive habitat, species, or element within a food chain. The data for assessing potential adverse effects on environmental receptors is even more scarce than that available for humans.

Having identified the complete current or potential future exposure pathways associated with a site, a risk assessment is then performed. The risk assessment uses available toxicological data on each chemical and characteristics of the exposure pathway and receptor to calculate the risk of an adverse effect. This risk is dependent on the means of exposure (inhalation, ingestion, or dermal contact), the period of exposure (time spent on site), and the type of receptor (child, adult, etc).

If the total risks represented by all contaminants on the site to a given receptor are above acceptable incremental risk, some form of remediation is necessary. With respect to carcinogenic effects in humans, the *Draft Health and Environmental Guidelines for Selected Timber Treatment Chemicals* (1993) defines an unacceptable effect as an increase in incremental risk to human health of 1 in 100,000.

RISK ASSESSMENT/RISK MANAGEMENT - A REMEDIATION TECHNOLOGY

By developing a clear understanding of the way risks are generated by contaminants on a site, a more flexible range of potential remediation technologies can be developed of a given site. A remediation technology that is protective of human health and the environment may not necessarily require removal or treatment of the contaminated soil.

Risk assessment technology can be used to manage the potential for effects to occur. A satisfactory remediation may take the form of controlling access to exposure pathways or controlling the presence of receptors to remove the potential for an effect to occur. This may be a more protective and cost effective solution than treatment of the soil to remove contaminants or removal of the soil from the site. Risk assessment and risk management achieve the primary objective of remediation which is to remove the potential for the adverse effect to occur, not necessarily to remove the contaminant.

By using a risk assessment to develop site specific contaminant levels, the site owner/occupier has the opportunity to select a remediation option that addresses the exposure pathways of concern, and volume of soil generating that concern. Risk assessment/risk management also gives the owner/occupier the knowledge and opportunity to either:

a. Control contaminant concentration
  - remediate or remove the soil

b. Eliminate exposure pathway
  - access controls (site use/access)
  - containment technologies

c. Remove receptors
  - relocate populations

When determining the appropriateness of exposure pathway and receptor controls for addressing current an potential future risk, both the owner/occupier and regulators must ensure that this is the best practical means of complying with the purpose of the Resource Management Act as defined in Part II Section 5(2).

SELECTION OF A SOIL REMEDIATION TECHNOLOGY

The process of developing and screening remediation options for a site has been developed into a reasonably well defined art by the United States Environmental Protection Agency. This process is also applicable to New Zealand and is broken down into eight general steps:

1 Develop remedial action objectives specifying the contaminants of concern, exposure pathways and preliminary remediation goals. The latter should be sufficiently flexible to allow a range of remediation options to be identified that can achieve the remediation objectives.

2 Develop a range of remediation options (containment, treatment, excavation etc) that could satisfy the remediation objectives.

3 Identify the volume of soil to which the remedial action objectives apply.

4 Identify and screen the technologies that are available under each remediation option and eliminate those that are not technically implementable at the site.

5 Select a technology from the range found to be technically implementable at the site for each remediation option. This step assumes the technology is available.

6 Assemble the selected representative technologies for each remediation option into the range of remediation options technically available for the site. The range of remediation options developed should encompass everything from a "no-action" option through to complete destruction and removal of the contaminants.

7 The initial range of options should then be screened on the basis of :

   i **Effectiveness** at remedying or mitigating the adverse effects associated with each exposure pathway and receptor of concern

   ii **Implementability** - both in terms of the stage of technical development of the technology (bench scale, laboratory, or full scale) and the administrative implementability. The latter takes into account regulatory restrictions and public acceptability of the remediation option

   iii **Cost** - typically done to an accuracy of +/-50% at this stage.

On the basis of this coarse screening process a reduced range of remediation options is identified. As full a range of options should be retained to provide a the site owner/occupier with a feel for the range of remediation options available and to demonstrate to the regulatory authorities that there has been no preselection of a remediation option. However, at this stage it may be possible to identify the most likely remediation option.

8 The final step is to carry out a detailed analysis of the short-listed range of remediation options. The criteria developed for carrying out the detailed analysis are:

   i **Overall protection of human health and the environment** - Explain how the remediation option will achieve this.

   ii **Compliance with all appropriate and applicable regulations.**

   iii **Long term effectiveness and permanence** - will the remedy still be effective once the remedial action objectives have been met.

   iv **Reduction in toxicity, mobility, and volume of contaminants through treatment** - evaluates the relative performance of alternatives that include treatment processes

   v **Short-term effectiveness** - assess the protection of human health and the environment during implementation and execution of the remediation option.

   vi **Implementability** - assesses the technical and administrative implementability of the remediation option

   vii **Cost**

   viii **Regulatory acceptance** - discusses the regulatory agencies preferences with respect to remediation

   ix **Public acceptance** -discusses the publics preferences with respect to remediation.

A preferred remediation option is then selected from the detailed analysis of options. The amount of detail required to complete this process will vary from site to site, industry to industry. Where a site is associated with an industrial clients with known and well documented contamination problems, the range of remediation options may have already been identified. In this case all that need be done is a detailed analysis the few remediation options known to be best suited to the site.

The process described above should be seen as a general guide for thinking through the process of selecting a remediation option, rather than a prescriptive process than must be rigorously followed. It also allows both the site owner/occupier and the regulators to evaluate options that involve risk management (capping, containment, access limitations) to control effects with those that use treatment.

SOIL REMEDIATION TECHNOLOGIES AVAILABLE IN NEW ZEALAND

The United States Environmental Protection Agency has developed a list of remediation technologies which are presented in the form of a screening matrix (Figure 1). Where known, technologies that are available in New Zealand have been indicated. Technologies that are proven, are affordable, and could be imported to New Zealand have also been indicated as potentially available.

In some cases remediation technologies are available in established industry within New Zealand. An example of this is cement kilns which are suitable for the thermal treatment of non-halogenated hydrocarbon contaminated soils. However, while significant amounts of work have been carried out overseas demonstrating that cement kilns can be used in this way, New Zealand regulators will still require convincing.

New Zealand currently has a fairly limited range of soil remediation options. These include:

No Action — The technologies available either won't work, are too costly, or create a greater risk to human health and the environment during remediation than doing nothing.

Institutional Controls — Use of site management plans, site use restrictions, monitoring, and other means to control the potential for exposure to contaminants.

Natural Attenuation — May be an option for some contaminants but requires long term monitoring.

Containment — Capping, cut-off walls, construction of an on-site landfill.

Off-site Disposal — Excavate and transport to suitable landfill.

Landfarming — Excavation and deposition in a thin layer to promote photodegradation, biodegradation, and volatilisation of contaminants. Suitable for volatile and to a lesser extent semi-volatile halogenated and non-halogenated hydrocarbons.

Solidification Stabilisation — Excavation and addition of lime, cement, and/or pozzolanic materials to stabilise contaminants prior to redisposal on-site or off-site.

Bioremediation — Inoculate soils with cultured 'super-bugs" or high concentrations of naturally occurring bio-organisms. Good for non-halogenated hydrocarbons - some limited success with halogenated hydrocarbons.

Some technologies could become available or may be being used on a few sites at present are listed below:

Vapour Extraction — Suitable for volatile contaminants on sites with higher permeability soils. Can be expensive if extracted gases have to be treated prior to discharge to the atmosphere.

Air Sparging — Air is injected into the soil to enhance biodegradation by existing naturally occurring bio-organisms. Simple equipment and only low air addition flow rates are required.

Thermal Treatment — Use of cement kilns for destruction of organic sludges, heavy resins, adhesives, and non-halogenated hydrocarbon contaminated soils.

Despite this range of technologies, there are still some contaminants that cannot be **treated** should the need arise. For example the most effective technology for destruction of halogenated hydrocarbons such as dioxin, pentachlorophenols (PCPs) and polychlorinated biphenyls (PCBs) is high temperature thermal destruction. This is a very expensive process because of the need to ensure no dioxin or furan formation in the exhaust gases.

In such cases, the technology screening process presented in this paper will identify the technological difficulties and cost implications associated with this form of treatment and a more affordable, risk management approach identified as the preferred remediation option. This may be an interim measure subject to periodic review and reevaluation as technologies develop in the future.

CONCLUSION

Risk assessment is a valid and defensible tool for determining the most appropriate and practical remediation option for contaminated sites. With New Zealand's limited financial resources and selection of remediation technologies, there will be occasions where the cost of contaminant destruction or the risks to human health and the environment to achieve destruction, outweigh the benefits. In these cases, risk management through exposure pathway control or receptor relocation may be the most practical and protective remediation option.

Because technologies, knowledge, land use requirements, and economic factors are all subject to change with time, remediation efforts should be subject to periodic review. This is especially important where remediation has been achieved by controlling an exposure pathway rather than by contaminant removal or destruction. Where contaminants remain on site, long term monitoring should be part of the remediation. This will ensure the review process has sufficient data to determine the continuing protectiveness of the remediation that was carried out.

REFERENCES

ANZECC, 1992 *Guidelines for the Assessment and Management of Contaminated Sites*

MFE, 1993 *Draft Health and Environmental Guidelines for Selected Timber Treatment Chemicals*

FIGURE 1

SOIL REMEDIATION TECHNOLOGIES

> **NOTE:**
> There are factors that may limit the applicability and effectiveness of any of the technologies and processes listed below. Source USEPA Remediation Technology Screening Matrix.

	Status - (F)ull-scale or (P)ilot scale (in USA)	Contaminants / Pollutants Treated [a]		Overall Cost	Capital (Cap) or O&M Intensive?	Commercial Availability in New Zealand
Soil, Sediment and Sludge						
In Situ Biological Processes						
Biodegradation	F	**3, 4, 5**	1, 2, 6	●	O&M	A
Bioventing	F	**3, 4, 5**	1, 2, 6	■	Neither	PA
In Situ Physical / Chemical Processes						
Soil Vapor Extraction (SVE)	F	**1, 3, 5**		■	O&M	PA
Soil Flushing	P	**1, 3, 7**	2, 4 - 6	O	O&M	PA
* Solidification / Stabilisation	F	**7**	2, 4, 6	■	Cap	NA
Pneumatic Fracturing (enhancement)	P		1 - 7	■	Neither	NA
In Situ Thermal Processes						
Vitrification	P	**7**	1 - 6	♦	Both	NA
Thermally Enhanced SVE	F	**2, 4, 6**	1, 3, 5	●	Both	PA
Ex Situ Biological Processes (assuming excavation)						
Slurry Phase Biological Treatment	F	**3, 5**	1, 2, 4, 6	●	Both	PA
Controlled Solid Phase Bio Treatment	F	**3, 5**	1, 2, 4, 6	■	Neither	A
* Landfarming	F	**3, 5**	1, 2, 4, 6	■	Neither	A
Ex Situ Physical / Chemical Processes (assuming excavation)						
Soil Washing	F	**2, 4, 5, 7**	1, 3, 6	●	Both	PA
* Solidfication / Stabilisation	F	**7**	2, 4, 6	■	Cap	A
Dehalogenation (Civcolate)	F	**2, 6**	1	♦	Both	NA
Dehalogenation (BCD)	F	**2, 6**	1	O	O	NA
Solvent Extraction (chemical extraction)	F	**2, 4, 6**	1, 3, 5	♦	Both	NA
Chemical Reduction / Oxidation	F	**7**	2 - 6	●	Neither	PA
Soil Vapor Extraction (SVE)	F	**1, 3**		■	Neither	PA
Ex Situ Thermal Processes (assuming excavation)						
Low Temperature Thermal Desorption	F	**1, 3, 5**	2, 4, 6	■	Both	NA
High Temperature Thermal Desorption	F	**2, 4, 6**	1, 3, 5	●	Both	NA
Vitrification	F	**7**	1 - 6	♦	Both	NA
* Incineration (low temperature)	F	**3, 4, 5**	1, 3, 5	●	Both	PA
* Incineration (high temperature)	F	**2, 4, 6**	1, 3, 5	♦	Both	NA
Pyrolysis	P	**2, 4, 6**	1, 3, 5	♦	Both	NA
Other Processes						
* Natural Attenuation	NA	**3, 4, 5**	1, 2, 6	■	Neither	A
Excavation and Off-site Disposal	NA		1 - 7	♦	Neither	A

[a] The listing of contaminant groups is intended as a general reference only. A technology may treat only selected compounds within the contaminant groups listed. Further investigation is necessary to determine applicability to specific contaminants.

* Conventional technologies / processes

Contaminant Codes
1 Halogenated volatile organics
2 Halogenated semi volatile organics
3 Non-halogenated volatile organics
4 Non-halogenated semi volatile organics
5 Fuel hydrocarbons
6 Pesticides
7 Inorganics
Target contaminants are listed first and in **bold type**

Rating Codes
■ Better
● Average
♦ Worse
O Inadequate information
NA Not available
A Available
PA Potentially available

BCD - Base catalysed dechlorination

REMEDIATION OF AVIATION FUEL SPILL IN GROUNDWATER AQUIFER, VANUATU

Diarmid Jamieson and **Mohammed Hassan**
Woodward-Clyde (NZ) Ltd, **Auckland**
Woodward-Clyde (NZ) Ltd, **Auckland**

SYNOPSIS

Undetected leakage of a pressurised underground pipeline at a fuel storage facility in Port Vila resulted in the accidental release of up to 100,000 litres of Jet A1 aviation fuel into the adjacent subsoil and the local contamination of an important aquifer by free-phase and dissolved petroleum hydrocarbon product. A number of groundwater users are located in the vicinity of the leaking pipeline including the town wellfield which supplies Port Vila with its supply of potable water from the same aquifer. A site investigation programme was carried out to identify the extent and profile of the contaminated soil and groundwater and a groundwater extraction and recharge system was installed and operated to prevent migration of the contamination plume away from the source of the release and to recover accumulations of free-phase fuel from the groundwater surface. Modelling of the plume of dissolved fuel was also carried out to predict the likely pattern of migration away from the site and the potential impact on downstream receptors. The results of the modelling were used to guide the final clean-up strategy for the site.

INTRODUCTION

Oil company inventory records showed that a significant quantity of aviation fuel (JetA1/Kerosene) was lost from an underground hydrant pipeline at a bulk fuel storage site in Port Vila, Vanuatu. The fuel leakage was identified as having occurred at pressurised flanged connections of the underground pipeline which is buried approximately one metre below the ground surface. The date and exact amount of the loss is unknown, however, inventory records showed that discrepancies in the input and output fuel volumes of up to 100,000 litres had occurred over a period of two years.

Woodward-Clyde was initially engaged to conduct an environmental investigation into the effects of the product leakage and were subsequently retained to plan and carry out the remediation of the the site. The main objectives of the remediation strategy were as follows:

1. To recover the free-phase hydrocarbons (free product) floating on top of the groundwater table.

2. To identify the likely adverse effects on the environment and health risks to human beings from the effects of the spill.

BACKGROUND AND ENVIRONMENTAL SETTING

The spill source was located over an alluvial aquifer about 3 km from the centre of Port Vila on the Island of Efate. Efate is roughly circular in outline, with a prominent embayment in the southwest caused by subsidence mainly due to faulting. The island has a tropical climate with a warm humid summer from November to April and a cooler, drier winter between May and October. The average annual rainfall at Port Vila has been recorded as 2332mm and the mean annual atmospheric temperatures vary between 22 and 29 degrees centigrade.

The water table at the site lies at approximately seven metres below the ground surface. The depth to the water table varies by approximately 1.5 metres between the dry and the wet seasons. The alluvial aquifer consists of coarse gravels, boulders and sand. The hydraulic conductivity of the aquifer has been estimated to be between 10^{-4} and 10^{-5} $m.sec^{-1}$, based on published data and an assessment of the aquifer geology from field drilling. Recharge to the aquifer is by direct rainfall infiltration and groundwater flowing from the upper catchment area. A substantial portion of the recharge is by direct infiltration of rain. The water table is relatively quickly affected by rainstorm events due to the porous nature of the alluvial soils. Groundwater has a gradient of less than one percent with a flow direction beneath the site towards the southwest. Potential receptors of groundwater flow from the area of the spill locations include several private water wells, the Tagabe River and the sea, located approximately 2 km distant.

The supply of potable water for Port Vila is abstracted from groundwater at a wellfield located in the same aquifer at around 1 km toward the south-east. Two groundwater supply wells are also located between the spill location and the wellfield. An emergency water supply well, located at an Ice Cream factory site, lies between the Port Vila wellfield and the spill location. To the south of the spill and located along the Tagabe River is a shallow hand-dug well fitted with a hand pump for domestic use.

Jet A1/Kerosene type fuel contains hydrocarbons in the range of C_9 to C_{16} and is typically comprised of the following group of compounds, expressed in percentage by volume.

• cyclo alkanes	0.74
• chlorinated aliphatics	0.05
• methyl alkanes	3.32
• polycyclic aromatic hydrocarbons	0.63
• monocyclic aromatic hydrocarbons	31.85
• simple alkanes	53.7
• others	9.71

Typically Jet A1 fuel has negligible solubility in water and a flash point of 38°C; in addition, benzene, a known carcinogen which can pose a threat to human health, forms about 0.02 percent by volume of Jet-A1 fuel.

Laboratory analysis of samples of the fugitive aviation fuel obtained from the groundwater indicated ethylbenzene, xylenes, benzene derivatives and napthalene as the detectable compounds. The analysis did not detect any polycyclic aromatic hydrocarbons (PAH) in the sample.

FIELDWORK PROGRAMME

Investigation Phase

The investigation phase involved the drilling and construction of sixteen groundwater monitoring wells (01 to 16) during 1992 around the area of the spill location. The monitoring well locations are shown in Figure 1. The purpose of the investigation was:

- to identify the groundwater flow regime at the site;
- to establish the presence and concentrations of dissolved and free-phase hydrocarbon product in groundwater at the site.

The monitoring wells were drilled using a cable tool rig and completed with 50 mm diameter slotted PVC pipe wellscreen. During drilling, samples of subsoil were collected from selected depths in each borehole and sent for laboratory analysis for petroleum hydrocarbon concentrations. Water levels and free-phase product thicknesses were also measured. The results indicated that Wells 03 and 06 contained significant amounts of free-phase product and Wells 02, 09 and 10 contained trace amounts. Subsoils close to the groundwater table at these locations also contained elevated concentrations of petroleum hydrocarbons.

Figure 1 also indicates the inferred extent of the plume of free-phase aviation fuel over groundwater, and the direction and gradient of groundwater flow. Monitoring Wells 04 and 05 are located upgradient of the spill source; Well 07 is located to detect any product migration towards the Port Vila town wellfield.

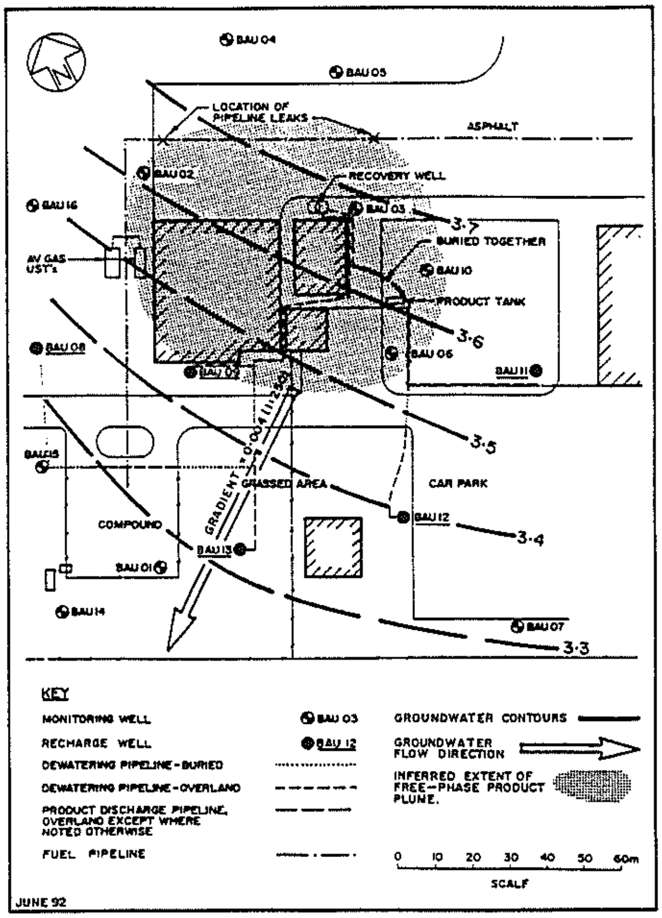


Figure 1 : MONITORING WELLS, RECOVERY WELL, DEWATERING DISCHARGE PIPELINES & GROUNDWATER CONTOURS PRIOR TO DEWATERING

Remediation Phase

A product recovery well was constructed at the location shown in Figure 1, where the free-phase product accumulation on the groundwater surface was considered to be the thickest and where access was possible.

The recovery well was constructed as a 12 m deep, 275 mm diameter borehole cased with slotted 250 mm PVC wellscreen. The product recovery system consists of the following components:

- dewatering pump with inlet at base of well to lower the groundwater table locally and to attract free-phase product flow towards the recovery well;
- groundwater recharge system to reinject the abstracted groundwater into downgradient perimeter wells and hence modify the local groundwater gradient back towards the recovery well;

- compressed air operated pump and floating skimmer system to remove accumulations of free-phase product from the recovery well; recovered product was pumped to an above-ground product storage tank along with some groundwater;
- an electrical and pneumatic control unit to operate the recovery system at selected regular intervals.

A schematic of the dewatering and free-phase product recovery system is shown in Figure 2.

The system has been operated and maintained by local personnel in Vanuatu for over 18 months. The total monthly and cumulative volumes of product recovered during this period are shown in Figure 3. The total volume of product recovered to the end of January 1994 was in excess of 20,200 litres. The free-phase product recovery system continues to operate during 1994.

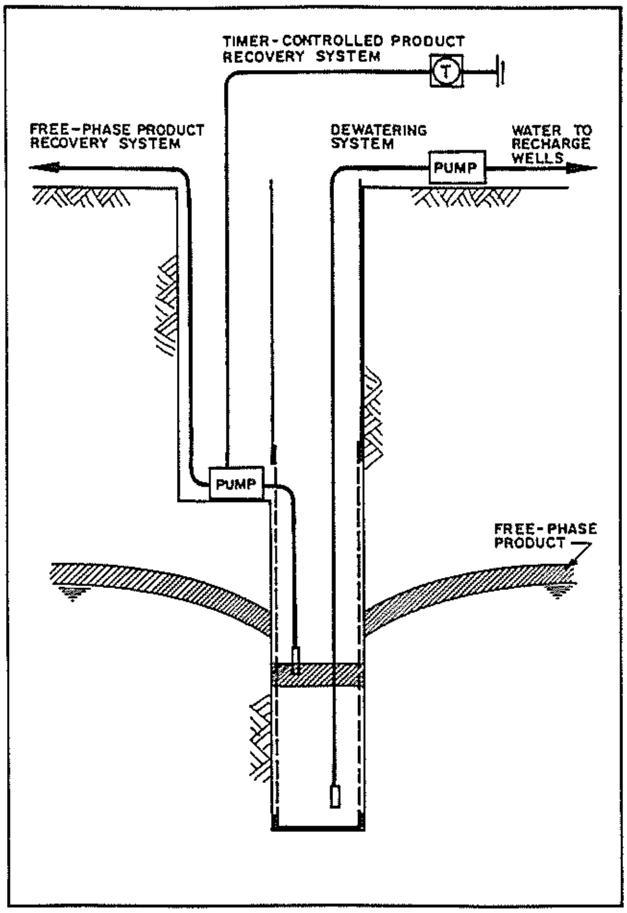


Figure 2 : SCHEMATIC OF DEWATERING/PRODUCT RECOVERY SYSTEM

Fuel Fate in Vadose Zone and Groundwater

The large seasonal variations in the level of the groundwater table at the spill site (up to 1.5 m) has the effect of smearing the lower vadose zone with hydrocarbon product within the plume. Hence the recoverable product is expected to be less than 50 percent of the total original spill volume. The remaining product which has smeared the lower vadose zone and is considered to be unrecoverable will remain in the soil and undergo natural biodegradation. The lighter fractions of the aviation fuel will volatilise into the soil air and slowly escape to the atmosphere. The volatilisation process is likely to be relatively slow due to the depth to the free product from the ground surface; however, the warm climate and relatively porous soil conditions are favourable for volatilisation to proceed. The remaining smeared product is expected to undergo aerobic biodegradation into carbon dioxide and water. Soluble fractions of the smeared aviation fuel will dissolve into the surrounding groundwater during the seasonal wetting cycle.

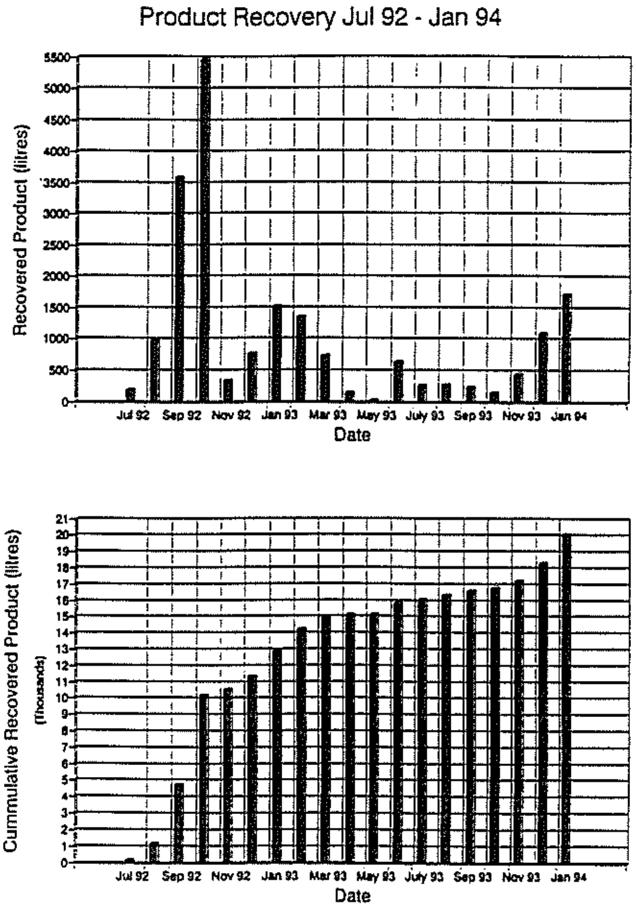


Figure 3 : PRODUCT RECOVERY RECORDS

Monitoring of Recovery System Performance

The recovery system monitoring includes the following:

- weekly measurement of the product and water levels in selected monitoring wells and the recovery well.
- monthly measurement of recovered product volumes.
- measurement of product and water levels in all the wells at selected times.
- determination of groundwater and recharge water quality at regular intervals.

Groundwater Quality Monitoring

A programme of monitoring the groundwater quality in selected monitoring wells and the recharge water quality has been carried out to measure any adverse effects on groundwater quality in the aquifer. Total Petroleum Hydrocarbon (TPH) and benzene, toluene, ethylbenzene and xylene (BTEX) have been used as parameters to indentify the presence of dissolved product in the groundwater. The results of the analytical programme are summarised in Table 1.

TABLE 1

SUMMARY OF DISSOLVED TPH/BTEX IN RECHARGE AND GROUNDWATER

Sampling Date	Location	TPH (ppm)	Benzene (ppb)	Toluene (ppb)	Ethylbenzene (ppb)	Xylene (ppb)
25-2-92	01	<0.5				
	04	0.5				
	04	0.6				
	05	<0.5				
	Ice Cream Factory	<0.5				
24-6-92	Recovery Well	0.3				
	Recharge Water	0.6				
	Field Blank	<0.2				
4-9-92	05	<0.1	<5	<5	<5	<5
	05	0.1				
	07	0.1				
	Recovery Well	0.1				
	Recharge Water	0.1				
24-8-93	07	<0.5				
	Field Blank	<0.5				
4-11-93	11	0.8				
	11	1.2				
	12	<0.5				
	Recharge Water	<0.5				
1-2-94	07		<1	<1	<1	<1
	09		<1	<1	<1	<1
	13		<1	<1	<1	<1
	14		<1	<1	<1	<1
	15		<1	<1	<1	<1

The groundwater monitoring results indicated that:

- elevated concentrations of dissolved TPH were initially present in the dewatering/recharge water; however the concentrations have reduced with time of operation of the system;

- elevated concentrations of dissolved TPH have been detected at some wells around the perimeter of the free-phase product plume; however recent analyses of groundwater from immediately down gradient of the free-phase product plume did not detect elevated concentrations of BTEX fractions.

In summary the fuel leakage appears to have had little significant impact on groundwater quality outside the immediate vicinity of the plume of free-phase product.

COMPUTER MODELLING

A three-dimensional analytical solute transport computer programme was used to model the likely dissolved plume migration rate and evaluate the resultant concentrations of dissolved petroleum hydrocarbons at down-gradient receptors in the event of the product recovery operations being terminated. Benzene was selected as the parameter of most environmental significance from Jet A1 fuel since the presence of detectable concentrations of PAH in the fuel has not been identified by the laboratory characterisation of the fuel. The contaminant plume was modelled in the solute programme as a slug of hydrocarbons at the top surface of the aquifer. The model was run with conservative input parameters, eg. all of the contaminant is soluble in water and present as dissolved benzene at 0.02 per cent by volume of the total spill volume. No account was taken of the reduction in spill volume due to the product recovery operations. The model was run assuming benzene as the contaminant for 2,5 and 10 years elapsed time. The modelling results are summarised in Table 2.

TABLE 2

SUMMARY OF GROUNDWATER MODELLING RESULTS

Spill Size (litres)	Dissolved Benzene (kg)	Darcy Velocity (m/d)	Elapsed Time (years)	Benzene Concentration in Groundwater 50 m (µg/l)	100 m (µg/l)	1200 m (µg/l)
50,000	700	8.6	2	9.2	4.6	0.4
50,000	700	8.6	5	3.5	1.8	0.1
50,000	700	8.6	10	1.8	0.9	0.1
100,000	1,400	8.6	2	17.6	8.8	0.7
100,000	1,400	8.6	5	7.1	3.6	0.3
100,000	1,400	8.6	10	3.5	1.8	0.1

The results of the computer modelling of the contaminant plume indicate that elevated concentrations of benzene may occur in groundwater directly down-groundwater gradient of the spill location. However the benzene concentrations predicted are relatively low and continuing physical attenuation would occur with time. In the event of termination of the current groundwater dewatering and free-phase product recovery operations at the site, the benzene concentrations in groundwater at a distance of 100 m or greater from the spill location are not expected to exceed 10 µg/l (the ANZECC drinking water quality guideline).

CONCLUSION

The results of the groundwater investigation, monitoring and modelling results has indicated that no significant environmental or health risks are present at this site as a result of the fuel leakage from the hydrant pipeline. The current programme to recover the available free phase hydrocarbon product from the surface of the groundwater table will continue, thus reducing the size of the contaminant source and the potential for adverse environmental effects. Contamination of the drinking water wells closest to the site is not expected to occur.

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RELOCATION OF CROMWELL AND CLYDE REFUSE DISPOSAL SITES

A P Stiles
Works Consultancy Services Ltd
Wellington

SYNOPSIS

The refuse disposal sites at Cromwell and Clyde townships in central Otago were due to be inundated when Lake Dunstan was formed following completion of the Clyde Dam. There was concern that such inundation could result in contamination of the lake water and investigations of the sites were carried out to assess the potential for contamination. The investigations revealed unacceptable levels of dumping of agricultural chemicals, industrial tars and oils at both sites. It was decided to remove all material dumped at the Cromwell site since 1982 and the total contents of the Clyde site, to locations remote from the effects of Lake Dunstan. A number of potential sites were identified and investigated, together with potential sources of low permeability liner material. The design option selected was to place the relocated refuse in excavations with a low permeability liner, install leachate and gas collection systems and cover the refuse with a low permeability capping layer and landscaped spoil. A monitoring programme was designed to confirm the integrity of the disposal sites.

INTRODUCTION

The Clutha Valley Development is a major hydroelectric development situated in central Otago, in the South Island of New Zealand and which is currently approaching completion. The main feature of the development is the construction of the Clyde Dam across the Clutha River, just upstream of the township of Clyde. Following completion of the dam, Lake Dunstan has been formed extending 18 km up the Cromwell gorge to the Cromwell township. From Cromwell, which is the confluence of the Clutha and Kawarau Rivers, the lake extends about 10 km up the Kawarau River and a further 8 km beyond Cromwell up the Clutha River, see Figure 1.

The existing refuse disposal site at Cromwell was located on the north (left) bank of the Kawarau River about 0.8 km upstream of its confluence of the Kawarau with the Clutha River. The refuse disposal site had been in use since the 1880s and covered an area of about 2.5 ha, with a frontage of approximately 400 m parallel to the bank of the Kawarau River and extending back between about 30 m and 50 m from the river edge. The base of the landfill was at about RL 178.5 m with the top at up to about RL 185.5 m. Such a level meant that, following filling of Lake Dunstan to it's design operating level of RL 194.5 m, the refuse site would be totally submerged and lie about 50 m to 70 m offshore.

The existing Clyde township refuse disposal site, which was considerably smaller and more recent than the one at Cromwell, was situated about 200 m upstream of the Clyde Dam on the east (left) bank of the Clutha River. The base of the landfill was at about RL 194 m and the top at RL 199 m. In a similar manner to the Cromwell site, it was apparent that the Clyde landfill would also be affected by the completion of Lake Dunstan. While in this instance the landfill would not be totally inundated the base of the site would be submerged, with the remainder of the refuse being affected due to rising groundwater levels.

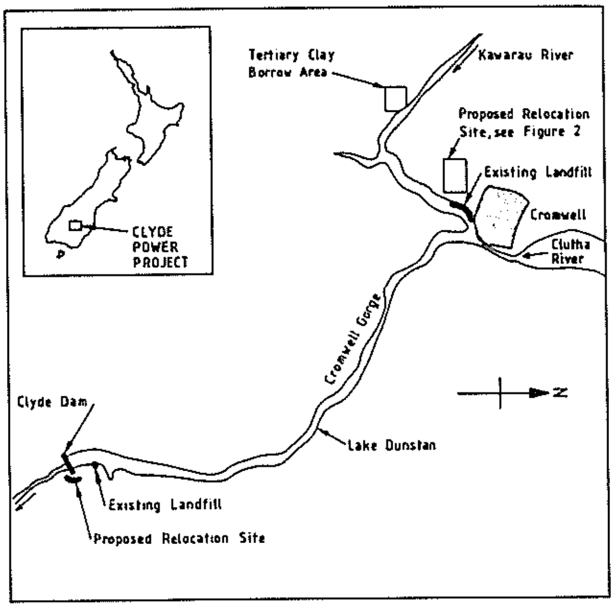

Figure 1 : Plan showing location of sites

Preliminary inspections indicated that dumping at both landfills was largely uncontrolled and that in addition to normal refuse, agricultural chemicals, industrial tars and oils could be identified on the working tip faces. It was clear that following inundation of the two sites by Lake

Dunstan there was a risk of contamination of the lake waters by toxic chemicals and oils leaching from the buried materials. The Clyde Power Project, (CPP), therefore commissioned Works Consultancy Services Ltd in 1990 to investigate the two sites, with a brief to assess the nature and extent of any possible contamination of the lake waters and to present possible options and recommendations for treatment of the landfill sites.

INVESTIGATION OF THE EXISTING REFUSE SITES

The investigation of the existing refuse sites comprised walkover surveys of both sites together with a programme of machine excavated test pits at the Cromwell site. The purpose of the investigations was to qualitatively determine if significant chemicals were present but not to quantify the relative amounts. In this instance, "significant" was defined as pesticides, chloro-organics, toxic metals, Polycyclic Aromatic Hydrocarbons (PAH) and dioxins. These substances were chosen based upon the probable consumables and products of the known industries in the Cromwell-Clyde area, principally fruit orchards, other agricultural activities and to a lesser extent waste from light industry. It was considered that if any of these "significant" substances were identified as being present then further investigations for other materials were unnecessary, as the significant substances themselves would give rise to sufficiently major potential health and environmental concerns as to warrant treatment of the sites.

Cromwell :

At Cromwell, 27 test pits were excavated up to 7 m depth and samples, both solid and leachate/liquid, taken from relevant excavations for possible analysis. An empirical approach was adopted in selecting samples with indicators of containment at an excavation location including smashed plastic containers, cardboard boxes with plastic liners, white drums, unknown coloured powders, oily seepages etc.

The results were conclusive, confirming the earlier visual inspections, with various agricultural chemical and industrial wastes found scattered throughout the refuse. Specific conclusions from the study included :

- there was no sign of decay or decomposition of domestic or industrial refuse which could be identified as having been placed after about 1982, (based on legibility of newsprint)
- over 20 assorted agricultural chemicals were identified. Individual quantities were generally small but distributed widely across the site.
- oils and industrial tars were present in small quantities
- analysis of leachate samples were indicative of "young" landfill in the stages of transformation from the aerobic to the anaerobic phase

The overall conclusion of the investigations was that agricultural chemicals appear to have been randomly (and illegally) dumped at the landfill since 1982 and there had been little biodegradation in the landfill since that date.

Clyde :

Investigations at the smaller Clyde landfill site were based on a walkover inspection which confirmed, in a similar manner to the Cromwell site, the presence of dumped agricultural chemicals and industrial waste.

ASSESSMENT OF CONTAMINATION

The conclusions from the investigations were that agricultural chemicals and industrial waste had been dumped at the sites in an uncontrolled manner and that while containers were generally of small size, intimate mixing of the organic compounds with the domestic refuse, which would have accelerated decomposition, had not been achieved.

The central Otago area has the lowest annual rainfall in New Zealand and this had a strong influence on the decompositional history of the existing refuse. The excavations at the Cromwell site showed the upper portion of material to be virtually dry and with little sign of decomposition corresponding to an initial aerobic "young" landfill phase. Such a phase is usually of short duration but the relative dryness of the two landfills appears to have held back decomposition processes. Agricultural chemicals and oils etc are therefore still present at Cromwell, virtually as dumped in refuse placed from 1982 onwards. At Clyde such contaminants were visible on the surface.

It is likely that at both sites wetting caused by inundation from Lake Dunstan will cause a distinct change in the degradation processes and they are likely to move to a "young" anaerobic phase. This will result in formation of leachate with mobilisation and interaction of the compounds and ultimately their release into the waters of Lake Dunstan.

The possible degree and consequences of such contamination were difficult to predict given the random distribution of unknown concentrations of chemicals through the refuse and the unpredictable release of contaminants as containers leaked or collapsed. Such contamination would have a detrimental effect on the eco-systems of the lake and on recreational users of Lake Dunstan. In addition, it was considered that there would be an unacceptable risk of contamination of the potable water supplies to the townships, which will be drawn from the lake. This was considered to be particularly important for Clyde, where the water intake was only about 500 m downstream of the existing refuse site.

SELECTION OF REMEDIAL OPTIONS

It was decided that remedial measures to prevent the possibility of such contamination would be required for both landfill sites and a number of options were initially considered namely

- grading the refuse sites and capping with spoil material. Impermeable materials were not readily available although could be obtained from weathered Tertiary sediments along the banks of the Kawarau river. Although there was a ready supply of permeable material adjacent to both sites, such materials would not prevent migration of leachate contaminants through the cap into the lake water. However, regardless of the material used as a capping layer, contaminants could still migrate through the permeable materials surrounding the sites and there was a risk of a build up of methane possibly rupturing the cap and releasing contaminants

- grading the refuse sites, capping with an impermeable layer and surrounding with an impermeable cut-off such as a slurry wall or grout curtain. While a recognised technique, particular in the USA, in this instance there were concerns about the ability to form an adequate cut off and the problems associated with gas rupturing the impermeable layer.

- "reworking" the refuse materials with bulldozers or similar plant to thoroughly mix the dumped chemical etc with the domestic refuse. However the dryness of the sites and hence the slowness of any degradation processes did not make this a practical proposition within the available timescale.

- relocation of the water supply intakes for Cromwell and Clyde could be considered although potential hazards to lake ecosystems and recreational users would remain

- removal of all or part of the contents of the two landfills and their relocation in properly engineered depositories away from the effects of Lake Dunstan. Relocation of landfill contents to a better engineered or environmentally less sensitive area is a recognised technique overseas and has been carried out previously in New Zealand albeit for highly concentrated and toxic chemical factory wastes.

After considering the engineering, environmental and timescale issues it was decided that the preferred option was to relocate all or part of the landfill contents. The selected treatment options for the two landfills were therefore identified as :

Cromwell :

relocate all material dumped at the site from approximately 1982 onwards. Investigations had shown that material prior to this date was sufficiently biodegraded to be considered to not present a problem with future contamination. It was also recognised that the problem of dumping of agricultural chemicals peaked in the years after 1982 as the orchards were cleared from the Cromwell Gorge as part of the preparations for impounding of Lake Dunstan. The areas of post-1982 filling had been approximately defined by the investigations but would be confirmed during the final clearance

Clyde :

relocate the entire contents of the relatively small refuse disposal site

SELECTION AND INVESTIGATION OF DISPOSAL SITES

Following initial discussions with the CPP, three potential relocation sites were identified to the south of Cromwell township, all within 1 km of the existing disposal site, see Figures 1 and 2. Similarly a potential relocation site had been identified within 0.5 km of the existing Clyde township refuse disposal site, see Figure 1. Site selection processes and investigations were therefore concentrated on determining the suitability or otherwise of these identified sites.

In order to select a site suitable for long term refuse disposal a large number of criteria must be considered. Geotechnical criteria include :

- identification of ground conditions beneath and in the vicinity of the site
- stability of any excavations both in the short and long term
- permeability of the surrounding in-situ materials
- suitability of materials from any excavations for use as liner material or landscaping spoil
- seismic aspects, such as the identification of any active faults which could affect the facility, together with the assessment of liquefaction potential
- effects on and of the existing regional groundwater and the final groundwater regime following the completion of Lake Dunstan
- identification of nearby readily available sources of lining and/or capping materials

Cromwell :

The three sites at Cromwell, referred to as the Camp site, the ex-Isles site and the Plantation site all lie close together on the flat lying terrace to the south and west of the present disposal site and between 200 m and 800 m from the Kawarau River, and at a level of about RL 210 m to 213 m, see Figure 2. The Central Otago District Council were proposing to construct a replacement township landfill in the Plantation site.

Published information suggested that the proposed sites would be underlain by Pleistocene outwash gravels up to 40 m thick, which in turn overlie Tertiary sandstones and mudstones. The gravels are thought to occur in a number of buried channels cut in the Tertiary sediment bedrock. Scattered outcrops of Palaeozoic schist occur along the west bank of the Kawarau River while Tertiary sediments are exposed in the southern side of the Kawarau River valley adjacent to the town of Bannockburn to the east of Cromwell, see Figure 1.

The existing level of the Kawarau River adjacent to the proposed site was RL 165 m and the limited groundwater data suggested groundwater levels beneath the proposed sites to be at about RL 180 m, ie about 30 m below ground level, with a gentle gradient towards the river. The nominal operating level of Lake Dunstan is RL 194.5 m and with time, regional groundwater levels in the immediate vicinity of the lake would rise to that level. Studies carried out by the Clyde Power Project suggested that the rise in groundwater to a final long-term level would be extremely slow and it is estimated that 100 years after lake filling the groundwater level at the site would be RL 198 m during normal flow and RL 199 m during a 100 year flood. For the purpose of design of the relocation facility it was therefore assumed that post-lake filling levels would be at least 12 m below ground level in the vicinity of the proposed relocation sites.

In order to confirm near-surface ground conditions a number of test pits were excavated to up to 7 m depth at the three proposed sites. All showed a similar sequence of cross-bedded coarse granular deposits. The ex-Isles site encountered gravelly sands whilst elsewhere the test pits showed predominantly sandy gravels. It was considered that, although the limited test pit programme showed broadly similar ground conditions across the site, the possibility of local variation being encountered in the disposal site excavations could not be discounted.

None of the test pits encountered groundwater nor even evidence of a seasonal capillary fringe with the deepest pit penetrating to RL 205.5 m. One of the main constraints on the location of any landfill site is the potential for contamination of groundwater resource. Investigations within the immediate vicinity of the proposed sites revealed one Water Right for groundwater extraction and two unregistered users with water extraction bores. Any design of the disposal site would therefore have to be engineered to minimise the risk of contamination of the local groundwater supply. In view of the likely development of a long term groundwater table with a gentle gradient towards Lake Dunstan, then there exists the possibility of contamination of the lake and ultimately the Cromwell township water supply, in addition to the immediate groundwater and extraction bores.

In terms of their relative potential for aquifer contamination all three sites were in approximately the same topographical location and at approximately the same elevation. The only differences were that the Plantation Site was more remote from Lake Dunstan hence maximising any seepage paths and the ex-Isles site was underlain by lower permeability material relative to the other two sites.

Refuse disposal sites should generally not be located adjacent to geological fault zones, particularly if such faults are still considered active. A study of available literature identified only one known active fault in the vicinity of the proposed sites. The fault, known as the Pisa Fault runs north-south to the west of Cromwell township and passes within 3 km of the Plantation Site. While other faults have been identified in the schist basement in the Cromwell area they are not known to have been active in Recent times, ie last 10,000 years. The considerable depth to groundwater and the generally coarse nature of the granular deposits suggests that any seismic event would not cause liquefaction of the near surface materials.

A low permeability liner and/or capping material would be required for the relocation site. Investigations were therefore also carried out to identify and assess the properties and quantities of readily available suitable materials. It was understood that oxidation ponds recently constructed by the Cromwell Borough Council had been lined with weathered Tertiary clays taken from a borrow area to the north of Bannockburn. The borrow area lies on a northerly facing bluff to the immediate south of the Kawarau River, see Figure 1. The material was identified in exposures at the former borrow area as a very weak light grey slightly sandy mudstone which had weathered to a sandy clay.

In addition to geotechnical considerations, a large number of other aspects need to be considered in site selection including :

- distance from the existing disposal site, in this instance less than 1 km.
- distance from airfields due to the possible attraction

of scavenging birds to the site.

- location of haul roads and the use of public roads
- proximity to existing developments
- impact on local residents and road users
- effects on wildlife habitats.
- visual intrusion during and following removal operations
- planned long term use for the site and surrounding area
- flood zones and erosion potential
- development and operation costs

In order to assess the suitability of the three proposed sites they were evaluated according to the criteria outlined above and on the basis of the geotechnical investigations. The evaluation gave relative ratings to each of the various criteria, with the basis for the ratings being those given "Guidelines for the Selection of Landfill Sites for Municipal Waste Disposal and Co-disposal (Works Consultancy Services, 1986) appropriately modified for this particular situation. The primary rating related to the risk of contamination of groundwaters and four cases were evaluated for each site, ie for the town water supply and for each of the three identified water abstraction bores. All three sites rated as acceptable for co-disposal of potential harmful wastes, such as identified at the current refuse site, provided the disposal site is adequately designed and constructed.

Clyde :

The single potential disposal site identified near the present Clyde landfill was investigated and assessed in a similar manner as for the sites at Cromwell discussed above. The proposed relocation site was south of the existing site in a backfilled haul road cut into the schist bedrock above and immediately downstream of the left abutment of the Clyde dam. The level of the backfill was between about RL 214 m and RL 220 m with the backfill up to about 10 m deep. It was proposed that currently placed backfill be partially removed, an engineered disposal site constructed and the removed backfill replaced to form a landscaped cover. Few other possible options were available in the immediate area and investigations concentrated on assessing the suitability of that site.

Published literature, confirmed by a comprehensive series of vertical and inclined boreholes carried out as part of the investigation works for the Clyde Dam, shows the area of the proposed disposal site to be underlain by Palaeozoic schists of the Haast Schist group. The boreholes and site inspections confirmed bedrock to comprise slightly weathered schist with a generally easterly foliation dip.

The investigations associated with the Clyde dam show the major River Channel Fault to be running north-south along the Clutha River through the schist basement rocks. Associated with this fault are a number of subsidiary steeply dipping faults, several of which intersect the line of the backfilled haul road. Whilst the River Channel Fault is considered to be potentially active, the subsidiary faults are not thought to have moved since the late Quaternary and may be considered inactive.

Test pits excavated in the backfill materials showed them to comprise dense silty sandy gravel and cobbles with frequent boulders. These corresponded to site records which indicated that the haul road had been backfilled with river gravel deposits. The back (east) face of the backfilled haul road is formed of slightly weathered schist, sloping at about 50° to 60° towards the river.

Investigation boreholes for the dam showed the present groundwater table to have a gentle gradient towards the river with an inferred level at the proposed site of RL 150 m, ie 55 m below the general base of the haul road. Following filling of Lake Dunstan, groundwater levels below the proposed site are predicted to gradually rise to the general lake level of RL 194.5 m, ie at least 10 m below the base of the haul road. The groundwater will therefore be within the schist bedrock and there will exist the potential for any leachates formed from the proposed site to pass into the groundwater and then potentially into the lake. The site will be located above and immediately downstream of the dam abutment which also contains the intake for the new Clyde township water supply. Whilst contamination of such an intake must potentially represent the most serious risk associated with the site, this potential is mitigated by a fracture zone acting as a cut-off and by the likelihood that the gradient of a future water table will be to the south.

The overall conclusion, based on the available information and limited investigations, was that there were generally no geotechnical factors which precluded the use of the site for a long term disposal facility. The only proviso was considered to relate to the proximity of the active River Channel Fault. However in view of the fault being at least 300 m from the site and all other identified faults being assessed as inactive, then the location of the site relative to the identified faults, whilst not ideal, was considered to be acceptable.

The suitability of the site was assessed in a similar manner as for the Cromwell sites discussed above with the most significant rating being for the potential contamination of the Clyde Township water supply intake. The evaluation of the site indicated that it was, by generally accepted standards, unsuited to co-disposal, but acceptable for municipal refuse disposal. It was therefore decided that the site could be used for the relocation of the contents of the existing Clyde refuse disposal site with the proviso that all excavated material would have to be inspected prior to relocation and any significant quantities of suspicious or obviously toxic materials removed, stored separately and taken to a more suitable site, such as the Cromwell relocation site.

DESIGN CONCEPTS

The general concept for the design and construction of both disposal sites was :

- excavate the site to the required level
- compact the existing granular materials
- place a low permeability liner, granular drainage layer and leachate sumps
- place and compact the relocated refuse
- install gas collection system
- place a low permeability top liner
- place landscaped spoil as required

An essential part of the design, requiring a significant geotechnical input, was the design of the low permeability liner. Following discussions with the Client three locally available liner materials were identified namely :

- crushed schist material from a source just downstream of Cromwell and which had been used as a low permeability upstream blanket for the Clyde Dam
- limited quantities of bentonite, surplus from the dam construction and which was stockpiled at Cromwell
- weathered Tertiary clays from the borrow area to the north of Bannockburn

In addition, consideration was given to using spray-on and HDPE geomembrane type effectively impermeable liners, although there was a clear preference to use the cheaper, locally available natural materials if possible.

Unless a totally impermeable liner such as HDPE or similar is used, some flow of leachates can occur through a natural, ie clay, landfill liner. The rate of flow will depend upon the permeability of the placed material, the thickness of the layer and the imposed head. The effects of leachate flows can be minimised however by ensuring that the clay liner has a high cation exchange and leachate attenuation capacity thus enabling the liner to remove many of the potentially harmful constituents from the leachate as it passed through.

In order to assess the relative properties and suitability of the three identified locally available natural liner materials a programme of laboratory testing was carried out. Particle size distribution analysis, laboratory permeameter testing and in-situ permeability testing had already been carried out on the crushed schist material by the Client. Those test results were supplemented by chemical testing to determine the material's ability to attenuate undesirable metal ions, ie cation exchange capacity, and its tendency to disperse in water. In an attempt to improve the physical and chemical properties of the crushed schist, a similar series of tests were carried out on samples of the material mixed with varying proportions of bentonite. A series of in-situ permeability tests were also carried out in a borrow pit in the Tertiary clays north of Bannockburn. These results were supplemented by laboratory permeameter testing and chemical testing.

The testing programme suggested that a design value of the vertical coefficient of permeability K_v of 8×10^{-8} m/sec for the crushed schist was appropriate for design. Addition of up to 5 % bentonite gave no obvious decrease in values of K_v. For the Tertiary clay materials a design value of K_v for placed liner material of 3.3×10^{-9} m/sec was considered appropriate. The chemical analyses suggested that the cation exchange capacity of the crushed schist material was very low and that, in the absence of organic matter, the soil would have a very limited ability to retain organic pesticides, metal ions and solvents. Addition of bentonite did not significantly improve the chemical characteristics of the crushed schist although it was considered that the bentonite would assist dispersion if the system was kept wet. The test results from the Tertiary clays showed a higher cation exchange capacity than the crushed schist material making it a superior lining in terms of retaining undesirable metal ions.

In order to assist in the design process for the low permeability liner materials, a risk analysis of the various options was carried out. The principal hazards associated with the relocation of the refuse sites were that leachate could contaminate the groundwater, Lake Dunstan and ultimately potable water supplies.

Clearly a number of designs can be considered each of which will have a particular level of overall risk and also cost for the various postulated hazards. At one extreme, placing the refuse in an unlined excavation minimises costs but maximises the risk of contamination. Conversely, totally sealing the refuse in an impermeable HDPE liner minimises the risk of contamination but at a high cost. In practise, a compromise option would be considered acceptable. For the risk analysis five proposed options were considered for six potential identified hazards which it was considered could lead to excessive levels of contaminants in potable water. It was not practical to quantify the levels of risk and they were therefore categorised subjectively using terms indicative of relative levels of risk. The terms used, in order of increasing risk were very remote, remote, negligible, insignificant, minor, significant. Table I sets out a subjective assessment of the level of risk of each of the hazards occuring.

	No Liner (Option A)	1.0 m crushed schist (Option B)	1.0 m Tertiary clay (Option C)	Interlayered crushed schist and Tertiary clay (Option D)	HDPE or similar liner (Option E)
Risk of surface or groundwater infiltration	Significant	Remote	Remote	Remote	Very Remote
Risk of excessive quantities of undesirable materials being present	Significant	Significant	Significant	Significant	Significant
Risk of leachate penetrating liner	Significant (no liner)	Remote	Remote	Remote	Very Remote
Risk of undesirable constituents not being removed from leachate that penetrates liner	Significant (no liner)	Significant	Negligible	Insignificant	Very Remote (assume impermeable)
Risk of groundwater movement towards potable sources	Minor	Minor	Minor	Minor	Minor
Risk of inadequate dilution not being provided at water resource	Insignificant	Insignificant	Insignificant	Insignificant	Insignificant

Table I : Comparisons of individual risk factors of excessive levels of contaminants in potable water

The results of the risk analysis suggested that Option D, to construct the liner as a combination of crushed schist material and imported Tertiary clays, was the preferred option and presented an acceptable level of risk at a moderate cost. The general concept of the design was to combine the low cost, but poor chemical properties, of the readily available crushed schist with the good chemical properties, but high cost, of the imported Tertiary clays in order to produce a liner acceptable on technical, economic and risk grounds. The relative proportions of the two materials and overall thickness of the liner was derived by considering the likely time period required for leachates to pass through the liner under the assumed leachate heads that might form. After consideration, two and three layer options, with overall base liner thicknesses of 1.0 m, were proposed, see Figure 5.

CONSTRUCTION RECOMMENDATIONS

The lining materials were to be placed in thin layers and compacted to at least 95 % New Zealand Standard Compaction in a strictly controlled manner. As it was considered that the greatest risk of leachate seepage was through the base of the excavation, the use in the liner construction of the expensive imported Tertiary clays was discontinued 1.5 m above the floor of the disposal area, reducing the overall liner thickness to 0.7 m. Similarly, the less onerous operating conditions for the capping layer enabled the liner for this section to be reduced to 0.3 m of crushed schist.

A granular leachate collection layer, 0.3 m thick, was to be placed over the low permeability liner at the base of the excavation and extended up the sides of the excavation. The granular layer was to be led to leachate collection sumps where any leachate that did form could be recycled through the landfill via the gas collection pipes and layers. This approach was considered acceptable in view of the small amounts of leachate that were expected to form in the dry, sealed refuse.

Although it was considered that gas production was likely to be minimal it was thought prudent to design for its possible production. After the bulk of the refuse was placed a collection system comprising slotted PVC pipe in a 300 mm thick granular gas collection layer was to be installed. The granular layer and PVC pipes were to be led to the leachate collection access shafts and vented to the air.

Cromwell :

A general design cross-section through the relocation site at Cromwell is shown as Figure 3. The final design of the facility was for up to about 100,000 m^3 of refuse with a plan area of about 150 m by 200 m. The general design level for the base of the excavation was RL 205 m, but with the formation graded at not less than 1/50 crossfall resulting in "valleys" leading to the leachate collection sumps. Following completion of the facility the relocation site was to be capped by a low mound of well compacted crushed schist material and landscaped spoil, constructed with a surface gradient of a least 1/200 towards the site perimeter. Surface water drainage channels to minimise the risk of surface water infiltration were to be installed. It was also recommended that the completed landscaped mound be fully vegetated to minimise erosion and infiltration and that a "biotic barrier" of cobbles and boulders be placed below the topsoil, in order to prevent possible breaching of the impermeable cap by burrowing animals.

Clyde :

The scale of the disposal facility required at Clyde was very much smaller than that for Cromwell and was designed for up to 11,000 m^3 of refuse. The same general design principles were used for the Clyde site, although some modifications were required to cater for the much steeper excavation sides. Figure 4 shows typical design cross-sections through the disposal site. The site was approximately 100 m long, up to 70 m wide and up to 10 m deep. A single leachate sump was used situated at the deepest point towards the southern end of the site. The gas collection pipes were arranged in a "herring-bone" pattern leading to the sump. Along the majority of the length of the disposal site, the backfill material was fully excavated to enable the liner material to be placed over the bedrock. In order to minimise the risk of damage to the liner it was recommended that the bedrock be covered by a layer of either graded backfill material or crushed schist prior to formation of the liner. Following completion, excavation spoil was landscaped with a general fall away from the existing eastern rock face of the haul road. Surface water drainage, vegetation and a biotic barrier were recommended in a similar manner as for the Cromwell site.

Monitoring :

Monitoring of the disposal sites following construction, was considered to be an essential part of the design process. At the Cromwell site it was recommended that up to four boreholes be installed to monitor groundwater levels and quality between the disposal site and Lake Dunstan. At Clyde two boreholes were established to monitor whether leachate contamination was migrating from the site, either to the new Clyde township water supply intake just upstream from the dam, or alternatively through the rock mass of the dam abutment towards the township and any potential bore supplies.

It was recommended that the boreholes be sampled at regular intervals and the samples analysed for background water quality over a wide range of organic and inorganic parameters. The sampling frequency will depend upon the observed rate of leachate formation. As noted above leachate formation is expected to be minimal and to decrease with time and hence sampling frequencies can also be decreased accordingly. Gas production will also be monitored from the leachate collection access shafts but is expected to show a similar trend to leachate production.

CONSTRUCTION ASPECTS

Works Consultancy Services Ltd were not directly involved in the construction of the relocation facilities which were supervised directly by the Client. Both facilities were completed without incident to the general design concepts and procedures outlined above. The remaining refuse at the Cromwell site was graded and covered by a blanket of spoil material prior to commencement of lake filling. Both the original sites have now been inundated by Lake Dunstan and monitoring results to date confirm that both the original sites and the relocation facilities are operating satisfactorily.

ACKNOWLEDGEMENTS

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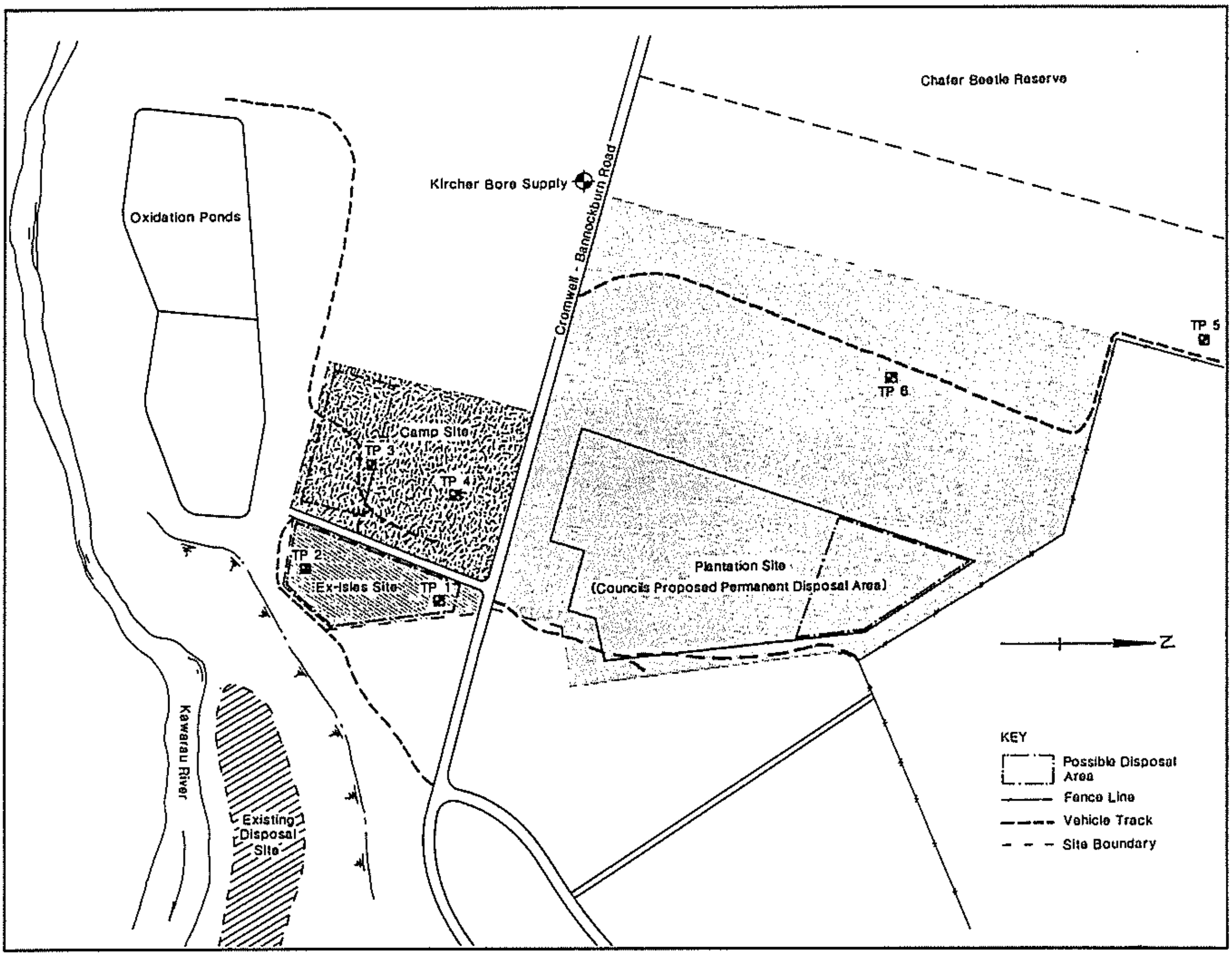


Figure 2 : Sketch plan showing relative locations of option sites, existing disposal site and test pits, Cromwell

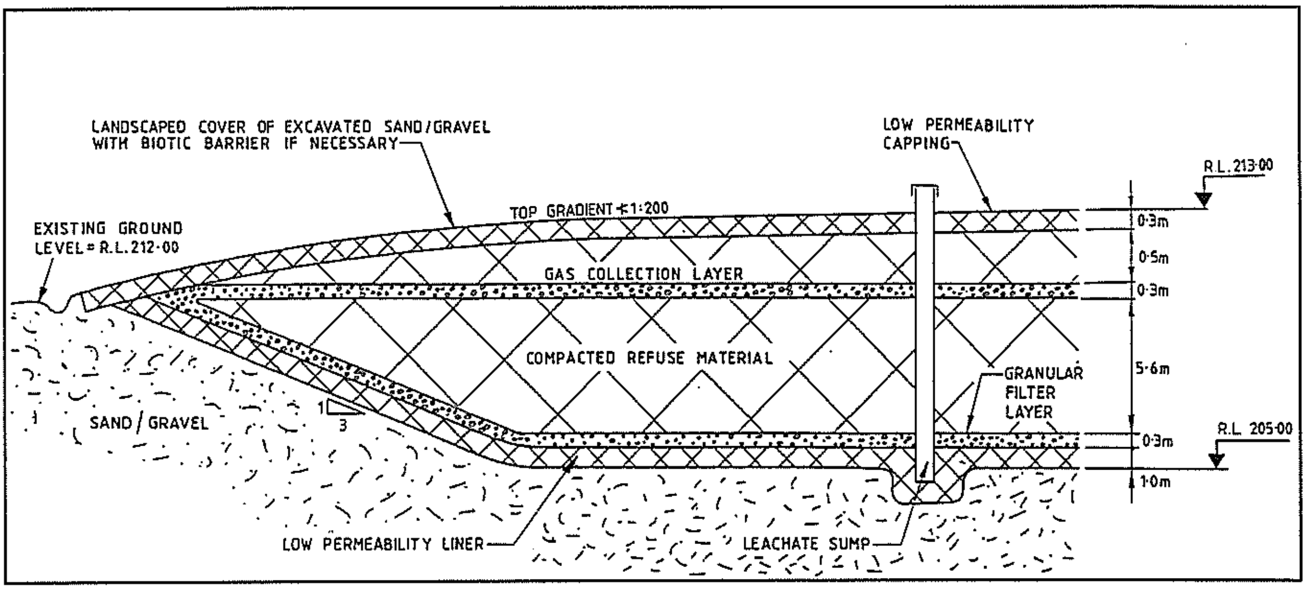
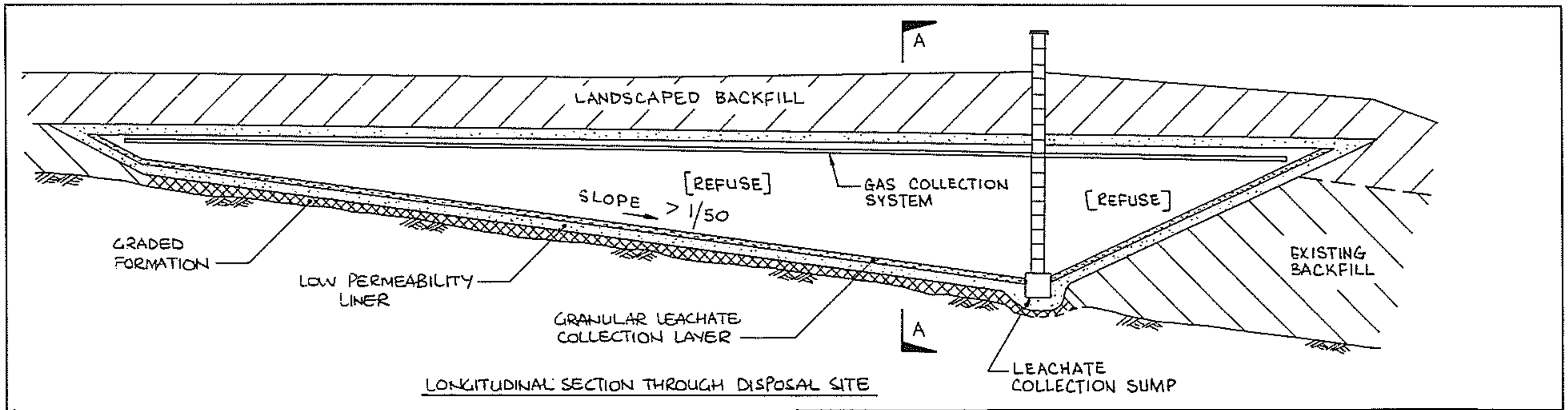


Figure 3 : Typical cross section through Cromwell disposal site



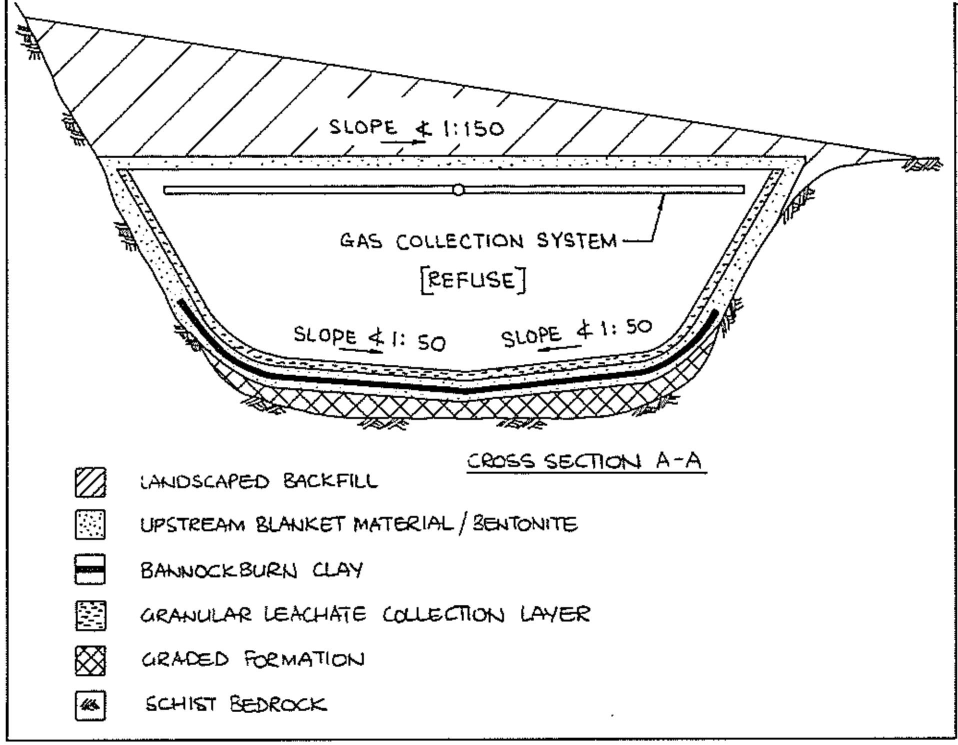


Figure 4 : Typical cross sections through Clyde disposal site

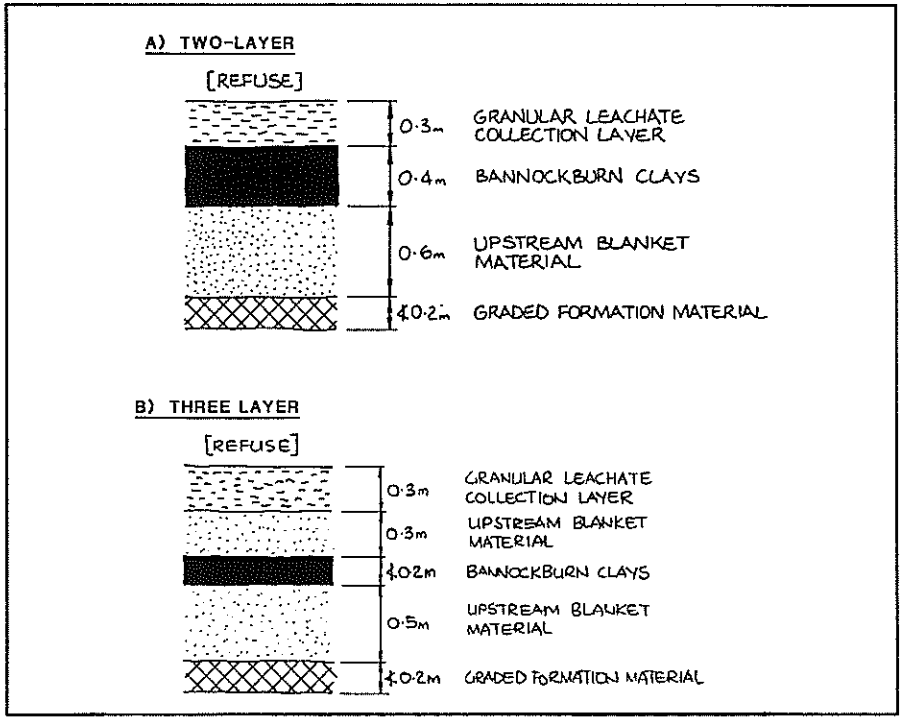


Figure 5 : Typical cross sections through base of landfills

THAMES LANDFILL - SITE AUDIT

Peter Higgs
Thames-Coromandel District Council

SYNOPSIS

This paper presents the results of investigations of a Site Audit carried out at the Thames Landfill in 1992. The purpose of the audit was to assess the effects that the existing landfilling operations were having on the environment. This information was intended to serve as a basis for determining the actions and procedures necessary to meet obligations under the Resource Management Act associated with closure of the landfill by 31 March 1993 and/or for possible additional consent applications necessary to continue filling operations after that date.

The paper highlights that the final decision on the appropriateness of the use of the site as a disposal facility has been made on non-technical issues.

1. INTRODUCTION

This paper outlines the investigations and results of a site audit of the Thames (tip) landfill.

The site was, for most of its life, an uncontrolled tipping operation and can therefore not be realistically described as a sanitary landfill. During its later years (1980 onwards) the District Council paid more attention to the method of operation and management of the site.

2. BACKGROUND AND HISTORY

In 1983 the area was bunded from the sea, a rudimentary leachate collection system was installed, stormwater control systems were constructed (including a sedimentation pond), and the method of operation was improved (e.g. covering and compaction).

Initially the leachate system was by way of retention on site with disposal to the Firth of Thames during intense rainfall as a leachate/stormwater mix, or by evaporation at other times. The discharge was authorised by a water right from the Hauraki Catchment Board (now Environment Waikato).

In the late 1980s, the leachate collected was pumped to the community sewerage scheme and only in extreme rainfall events would overflow of leachate be discharged with stormwater to the Firth, albeit in reduced concentrations.

The District Council was aware that the site was inappropriate as a long term solution for refuse disposal for Thames and was investigating options for a centralised landfill for the district. During the 1980's some seven tip sites were closed (because they were of unsuitable standard), within the district and sites at Coromandel, Colville and Whitianga were established which were capable of meeting planning and water right consents. The Council also developed a strategy which addressed all aspects of solid waste management in an integrated manner. Part of this strategy was the rehabilitation of existing and old or disused sites.

Although the closure of the Thames tip was as a result of a Planning Tribunal Decision, the Council had already accepted that the life of the site was limited and had indicated such in the review of the District Scheme (now Plan). In addition the Council had budgeted to carry out an environmental audit of the site together with rehabilitation for a suitable end use for the land.

3. SOCIAL IMPACTS

3.1 General

This project demonstrated that the physical impacts, i.e. geotechnical and ecological, are not necessarily the determining factors in establishing the suitability of a site for a particular use - in this case refuse disposal.

Community values and expectations were the determinants which decided that the site should be closed and are the main determinants for the rehabilitation and end use of the site.

3.2 Specific Issues

3.2.1 Reclamation

The site is reclaimed seabed. Council had the authority under an Empowering Act to reclaim seabed for some distance into the Firth of Thames. A number of "cleanfill" reclamations have been constructed which have provided Thames with useful land - e.g. housing subdivision, shopping complex, playing fields.

Initially, it appears that, the tip reclamation would provide land for housing and also provide a "cheap" facility for the disposal of refuse. However in later years it was realised that an end use for housing would be difficult due to the geotechnical aspects e.g. settlement and gas, and that the ongoing maintenance and rehabilitation of the site after it closed was not going to be "cheap".

There was also a growing awareness, both nationally and locally, that reclamation was not a desirable activity in the coastal environment and it should not be viewed as the easy solution.

The District Council took these wider issues into account when it limited the area available for reclamation in its District Scheme Review.

3.2.2 Coastal Environment

The Firth of Thames is an important fish nursery and is used extensively by a large number of birds. A study by Department of Conservation (1990) estimated some 40,000 migratory birds utilise the area.

Indeed the Firth of Thames is listed as a wetland of international importance under the Ramsar Convention.

Again these issues were taken into account when determining the fate of the site.

3.2.3 Maori Issues

The Thames area has many issues of cultural significance both Maori and post pakeha settlement (gold mining, kauri). While no urapa, waahi tapu, nor specific areas of significance have been identified at the site, there are several in adjacent areas.

The seabed is under claim to the Waitangi Tribunal and titles exist for this land.

These issues were also significant in the consideration of the site.

3.2.4 Community Issues

While the site was in a convenient location for public access, there was growing concern within the community of the appropriateness of the landfill tip locality. The site is adjacent to housing and the area along the foreshore is used more often now for recreation. A foreshore walkway linking the Thames Wharf is being developed and a bird hide has been constructed nearby. The area also abuts a Heritage area (Grahamstown) and is a part of the Coromandel Heritage Trail.

Similarly these issues were taken into account.

3.2.5 Tourism Issues

Tourism is the major industry for the district, both national and international. The Coromandel has a reputation for its scenic and environmental values. The clean green image is an important aspect for the industry. In the marketing of any product it is essential that the customer perception of quality is maintained. Although technical evidence might show that an operation such as the Thames Landfill has little environmental impact, the perception of "tipping rubbish into the sea" is incongruous with the image of a clean green environment that visitors expect to experience when they come to the area.

This was an important issue to be considered.

3.2.6 Political Issues

The District Council has a mission statement:

"A District Council committed to good government by openly serving its communities through communicating, forward planning, advocating and promoting public services, amenities and sustainable development opportunities - all consistent with our unique environment."

Further the Refuse Management Objective in the Annual Plan is:

"To provide for the needs of the District and minimise waste for disposal of refuse in a safe, efficient manner which meets the highest environment and health standards."

These together with the issues outlined above lead to a political decision to continue with the closure of the site. The evidence provided by the site audit suggested that there was technical support for pursuing the option of extending the site as a disposal facility.

However the non-technical issues were considered to be outweighed by social and cultural issues.

4. ENVIRONMENTAL SITE AUDIT

4.1 The site audit was carried out in 1992 and comprised three reports:

Barrett Fuller and Partners - "Thames Landfill - Geotechnical Status Report".

Barrett Fuller and Partners - Thames Landfill - Gas Status Report.

Kingett Mitchell and Associates - Thames Landfill - Environmental Status Report.

The site layout (as at May 1992) is shown in Figure 1.

4.2 The results of the site investigations and analyses are summaried as follows:

4.3 Geotechnical Features

The key findings of the geotechnical and related engineering investigations at the landfill are presented below.

4.3.1 Geotechnical Conditions and Subsoils

(a) Two district zones of filling exist within the landfill; the north east corner of the site is filled with stiff clays and silts while the remainder of the area consists of varying depths of domestic refuse mixed with clay.

(b) Alluvium and coastal sediments underlie the landfill; they consist of soft to firm clays up to 20 metres depth with shear strengths generally less than 20 kPa and in some cases less than 20 kPa. Figures 2 and 3 illustrate the site materials found by drilling and excavation.

(c) There is no artificial lining between the refuse/fill and the natural sediments. These soils are, however, highly impermeable with approximately in-situ permeability values assessed at 10^{-9}m per sec.

(d) The bulk permeability of the refuse is high even though it may have locally impermeable zones.

4.3.2 Groundwater/Leachate

(a) The phreatic surface within the landfill is perched above the surrounding estuary and high tide level.

(b) Piezometers within the landfill exhibited little or no response to tidal fluctuations. The surface drain which discharged to the estuary is subject to tidal fluctuations and is probably an important regulator of internal groundwater levels. (This drain has since been sealed off from the estuary).

(c) No external evidence of any seepages through the perimeter bund was observed (except where the surface drain discharged to the estuary).

(d) It was concluded that there is little inter-connection between the groundwater/leachate within the landfill and the external marine groundwater regime, except via the stormwater drain (since sealed off).

4.3.3 Settlement

(a) Settlement of the landfill surfaces takes place due to:

Compaction and consolidation of the refuse/fill;
Decomposition of the refuse;
Consolidation of the subsoils.

(b) Expected settlements within the landfill are between 500mm and 1.25m over the next 20-40 years.

(c) Differential settlement will make it necessary to undertake regular maintenance of the rehabilitated landfill surface, in order to maintain the prescribed finished form until such time as the effects of settlement become minor.

4.3.4 Seismic Setting

(a) The site is situated close to the Hauraki Fault and Firth of Thames Fault neither of which have been the source of recent large earthquakes.

(b) It is estimated that seismic intensities greater than seven on the Modified Mercalli scale would be necessary before any damage to the landfill in the form of localised shallow failures or spreading would occur.

(c) The mean recurrence interval for such earthquakes lies in the 100 to 500 year return period.

(d) It is concluded that the potential for such damage, its limited extent and environmental consequences are acceptably low.

4.3.5 Perimeter Bunding

(a) The perimeter containment wall of the landfill is made up of a wide variety of natural and man-made materials. Rocks have been placed for wave protection along the western side. Along the southern portion of the landfill perimeter metal, concrete and other debris project directly onto the foreshore.

(b) The perimeter is unsightly and unsafe for public access at present.

(c) Any modifications to the existing perimeter should be carried out only after adequate investigation of the bund.

4.3.6 Storm Events

(a) The perimeter is armoured with rock on the western edge up to a level of between 3 and 4 metres above MSL.

(b) This armouring has been exposed to recent major storm events and is likely to perform well even in extreme events.

(c) The southern edge is relatively sheltered; it does not require conventional armouring.

4.4 Environmental Quality

The key findings of the environmental investigations are presented below.

4.4.1 Vegetation and Ecology

(a) The upper surface of the landfill supports an almost entirely introduced flora.

(b) The fauna associated with the "rocky" shores around the landfill was depauperate. A total of 12 species were found compared with 23 at a control location at Rocky Point to the north of the landfill. Type of substrate, age of placement and exposure were thought to contribute to the lower diversity around the landfill.

(c) There have been significant changes in the distribution of mangroves around the landfill over the last 30 years. The effects relate to the physical expansion of the landfill. Significant loss of mangroves has occurred in adjacent areas. Some recolonisation along the southern shore is occurring.

(d) The abundance and diversity of benthic invertebrates found on the intertidal mudflats around the landfill was similar to that found at locations away from the landfill.

4.4.2 Leachate

(a) The examination of bores, bore logs and information obtained from pits excavated in the landfill has shown that the landfill contains primarily domestic waste. There are no records of hazardous materials entering the landfill.

(b) Leachate has escaped from the perimeter of the landfill at times. These leaks have been addressed by bunding with clay material. Leachate and stormwater drain to an open channel that discharges via a detention pond at the south eastern corner of the landfill during intense rainfall. Under normal conditions leachate is discharged to the public sewer. (The drain has since been sealed off from the estuary).

(c) The key features of the landfill leachate quality include elevated concentrations of the trace elements arsenic, copper, nickel, lead and zinc. Concentrations are similar or lower to those reported for other domestic landfills in New Zealand. Leachate samples from within the landfill examined for organic compounds revealed both naturally occurring (e.g. fenchore) and those of petroleum (e.g. benzene) or industrial origin (e.g. dichlorobenzene).

(d) The chloride concentration of the leachate within the landfill does not suggest any evidence of tidal intrusion into the landfill.

4.4.3 Sediments Within and Adjacent to the Landfill

The key outcomes of the investigation are set out below.

(a) The sediment in the landfill stormwater channel contains elevated concentrations of zinc, copper, nickel and arsenic reflecting concentrations observed in leachate in the landfill.

(b) A single intertidal sediment sample collected adjacent to the landfill stormwater drain discharge point contained slightly elevated levels of zinc, copper, nickel and arsenic.

(c) Other single sample elevations were noted such as zinc and lead in sediment at Site 16 which was located adjacent to the old sewer outlet pipe. The cause was not identified.

(d) Elevated concentrations of zinc, copper and arsenic were measured in the Waiotahi Stream. It is not possible to partition the elevations in the concentration of these parameters to either the concentration of these parameters to either the landfill or catchment without further assessment.

(e) The control sites sampled north and south of the landfill contained low concentrations of persistent organochlorine compounds. Their concentrations were similar to those recorded in sediments from the Hauraki Gulf.

(f) The sediments in the landfill stormwater channel was found to contain elevated concentrations of PAH's, PCB's, DDT, chlordane and dieldrin compared with sediments adjacent to the landfill.

(g) The closest intertidal sediment sample collected adjacent to the stormwater drain contained elevated concentrations of PAH's and dieldrin compared with other intertidal sediments.

(h) In all cases the concentration of PAH's, PCB's, DDT, chlordane and dieldrin were at or below guidelines set by the Australian and NZ Environment and Conservation.

4.4.4 Landfill Gas

(a) Landfill gas is being produced at the landfill. The composition of the gas varies considerably across the site, from 60% methane and 40% carbon dioxide at the western end to only traces of these at the northern end of the site.

(b) Gas composition was found to vary sometimes considerably, between readings taken from the same well at different times.

(c) Variable levels of gas were detected in bores and drains outside the landfill perimeter, to the north and east of the site. This gas could be migrating from the landfill or (more likely) it could be related to older in-situ refuse deposits of previous reclamations.

(d) The levels of CO_2 and CH_4 measured in bores on the landfill perimeter were found to be higher than recommended levels for residential areas set by USEPA and UK Department of the Environment. However as there are no buildings or structures immediately adjacent to these wells where unacceptable levels of gas could accumulate, the present levels of the gases are not cause for immediate concern.

4.5 Conclusions of Site Audit

(a) The landfill can be regarded as a "closed" system containing two material zones, the major one being domestic refuse and the other being clays and silts, founded on natural low permeability soils.

(b) There is little interconnection with the marine environment.

(c) The landfill has had very little effect on the quality of sediments or fauna in the intertidal zone.

(d) Elevated concentrations of certain contaminants at a single site are unlikely to result in significant effects on intertidal biota.

5. PUBLIC CONSULTATION

5.1 Public Participation - Landfill Rehabilitation

As part of the rehabilitation of the tip site Council has embarked on an extensive public participation process. The end use of the site has not been pre-determined by Council but will be decided upon as a result of the public participation programme.

Initial discussions have been held with the Tangata Whenua to identify specific Maori issues. A wide range of individuals, community groups and agencies have been invited to participate in formulating a rehabilitation plan.

The first stage of this process will be a one and half day workshop. On a Friday evening all participants will be welcomed at the local marae where the workshop format will be explained and major issues identified. On the Saturday a workshop session will be held at the site (the Council Depot is at the site) with some 40 participants. The objective will be to formulate a draft plan by the end of the day.

Following the workshop the draft will be prepared in more detail and further investigations carried out where necessary. The draft will then be publicly notified for comment and submissions. It is anticipated that, as a result of the representation of various interest groups, there should be little change to the concept reached by consensus at the workshop.

Provided there are no significant objections at this stage, then the rehabilitation plan will be put in place and any necessary consents obtained.

6. CONCLUSIONS

This symposium is on the Geotechnical Aspects of Waste Management and this section is on the Rehabilitation of Contaminated Sites.

The Site Audit of the Thames Landfill (tip) has dealt with the physical impacts - geotechnical and ecological - and has concluded that there is unlikely to be significant effects on the environment as a result of the refuse disposal operations.

However I have attempted to demonstrate that the final decision on the appropriateness of the use of the site as a disposal facility has been made on non-technical issues.

This is not to say that, conversely, a site that is not technically suitable (or not able to be engineered to be suitable) should be deemed to be acceptable.

The importance of public consultation is also stressed so that the decision is based on an understanding by both the community and the decision makers of all the relevant issues. In addition the decision is more likely to be accepted by all if they have had input and the chance to meaningfully participate.

7. ACKNOWLEDGEMENT

The support and consent of the General Manager of the Thames-Coromandel District Council in enabling this paper to be presented is acknowledged.

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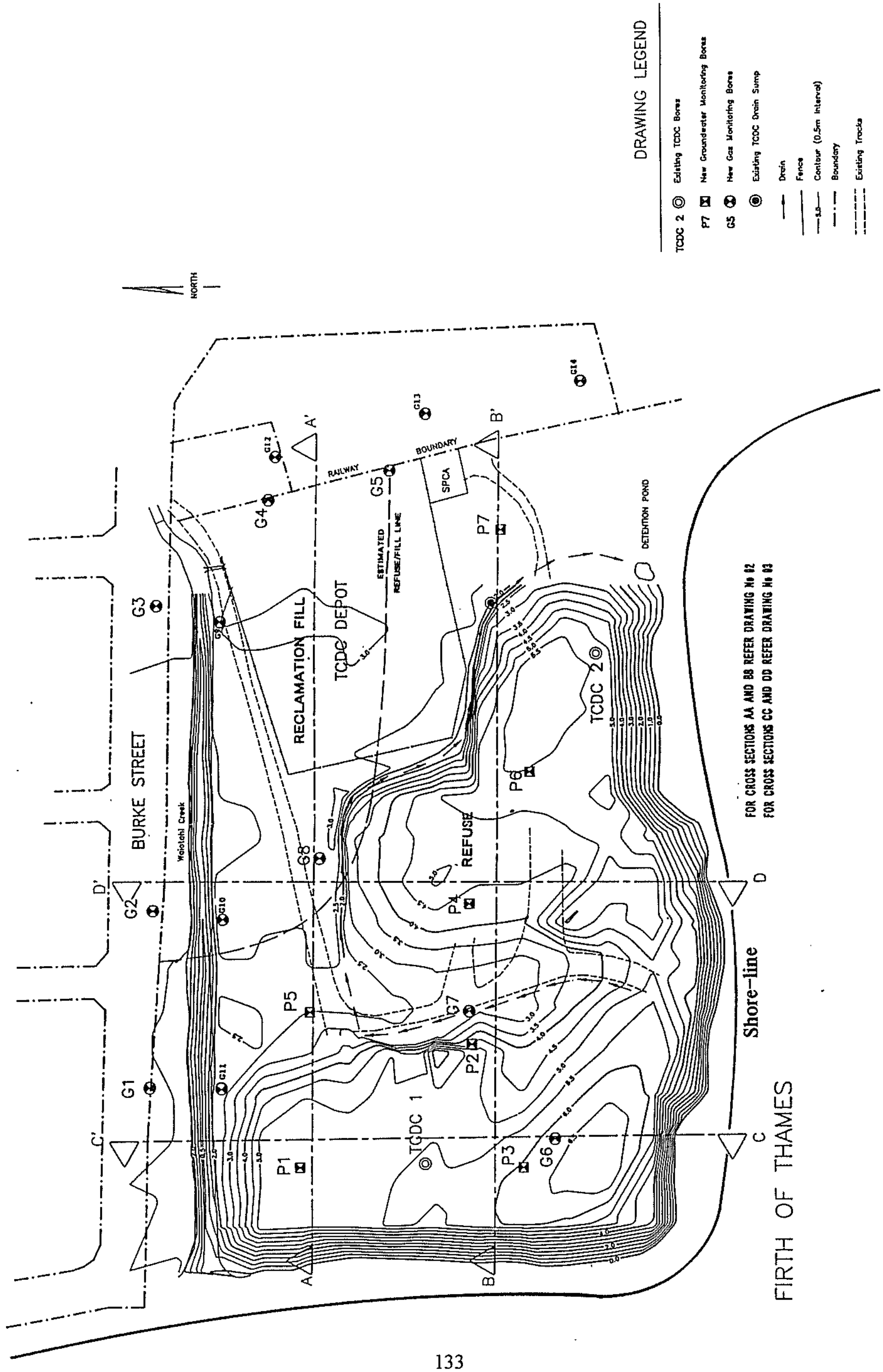
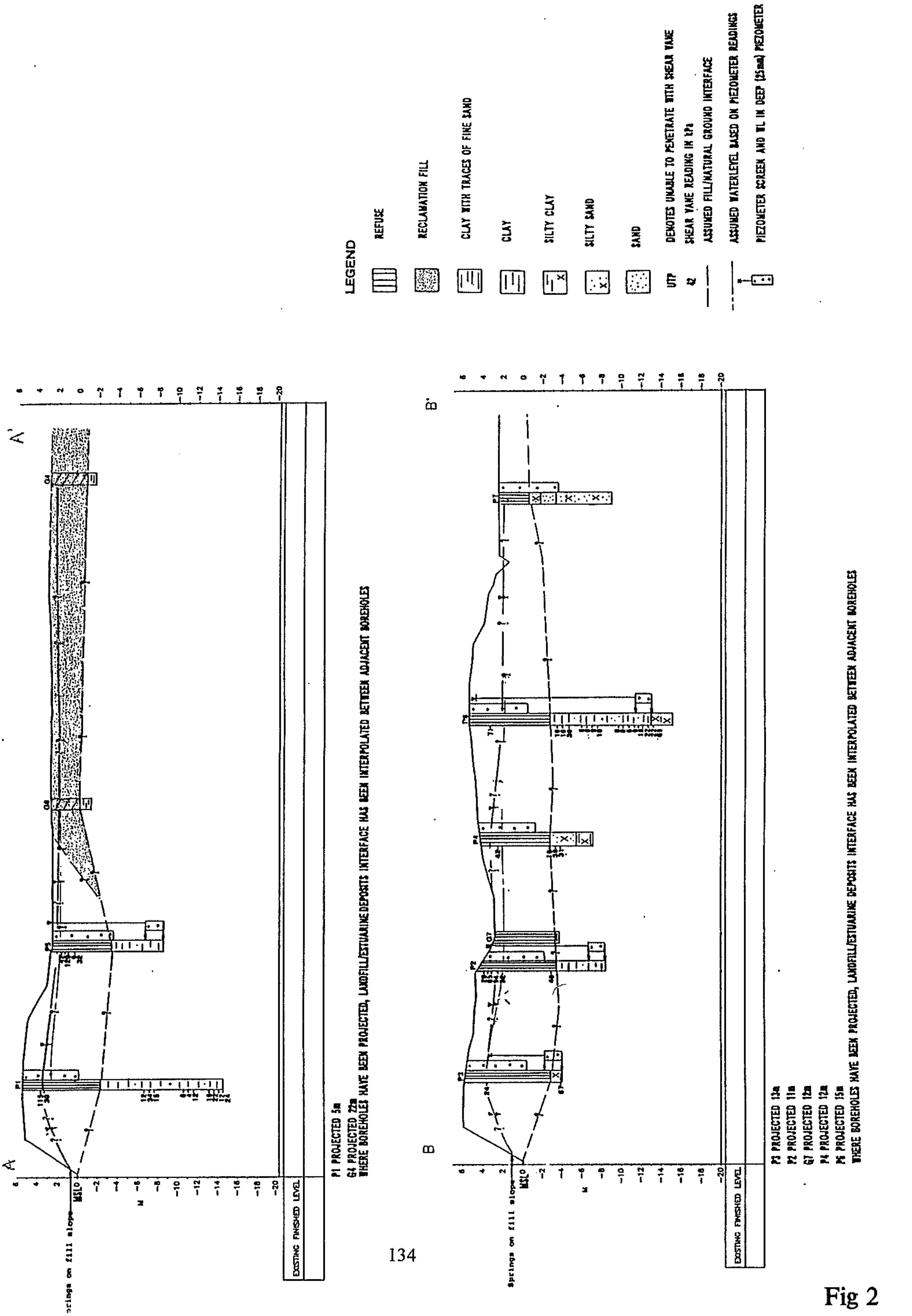
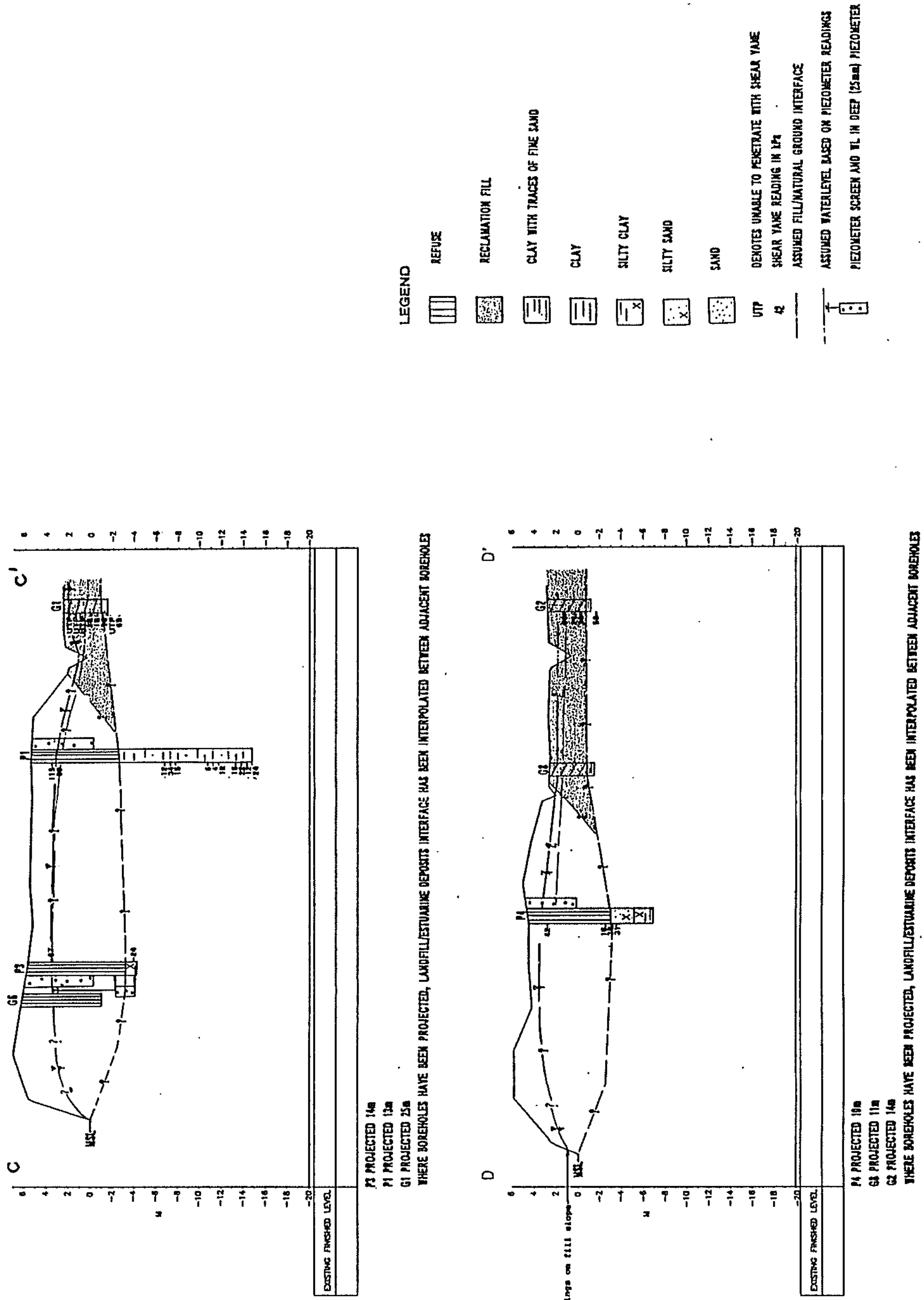

DRAWING LEGEND
TCDC 2 Existing TCDC Bores
P7 New Groundwater Monitoring Bores
G5 New Gas Monitoring Bores
Existing TCDC Drain Sump
Drain
Fence
Contour (0.5m Interval)
Boundary
Existing Tracks
NORTH
BURKE STREET
Waiotahi Creek
RECLAMATION FILL
TCDC DEPOT
ESTIMATED
REFUSE/FILL LINE
RAILWAY
BOUNDARY
SPCA
DETENTION POND
REFUSE
TCDC 1
TCDC 2
Shore-line
FIRTH OF THAMES
FOR CROSS SECTIONS AA AND BB REFER DRAWING No 02
FOR CROSS SECTIONS CC AND DD REFER DRAWING No 03

Fig 1



LEGEND
REFUSE
RECLAMATION FILL
CLAY WITH TRACES OF FINE SAND
CLAY
SILTY CLAY
SILTY SAND
SAND
UTP DENOTES UNABLE TO PENETRATE WITH SHEAR VANE
42 SHEAR VANE READING IN kPa
ASSUMED FILL/NATURAL GROUND INTERFACE
ASSUMED WATERLEVEL BASED ON PIEZOMETER READINGS
PIEZOMETER SCREEN AND WL IN DEEP (25mm) PIEZOMETER
A
A'
P1
P5
G4
EXISTING FINISHED LEVEL
Springs on fill slope
MSLD
P1 PROJECTED 5m
G4 PROJECTED 22m
WHERE BOREHOLES HAVE BEEN PROJECTED, LANDFILL/ESTUARINE DEPOSITS INTERFACE HAS BEEN INTERPOLATED BETWEEN ADJACENT BOREHOLES
B
B'
P3
P2
G7
P4
P6
P7
P3 PROJECTED 13m
P2 PROJECTED 11m
G7 PROJECTED 12m
P4 PROJECTED 12m
P6 PROJECTED 15m
WHERE BOREHOLES HAVE BEEN PROJECTED, LANDFILL/ESTUARINE DEPOSITS INTERFACE HAS BEEN INTERPOLATED BETWEEN ADJACENT BOREHOLES

Fig 2



C
C'
MSL
G6
P3
P1
G1
UTP
EXISTING FINISHED LEVEL
P3 PROJECTED 14m
P1 PROJECTED 13m
G1 PROJECTED 25m
WHERE BOREHOLES HAVE BEEN PROJECTED, LANDFILL/ESTUARINE DEPOSITS INTERFACE HAS BEEN INTERPOLATED BETWEEN ADJACENT BOREHOLES
D
D'
Springs on fill slope
MSL
P4
G8
G2
EXISTING FINISHED LEVEL
P4 PROJECTED 19m
G8 PROJECTED 11m
G2 PROJECTED 14m
WHERE BOREHOLES HAVE BEEN PROJECTED, LANDFILL/ESTUARINE DEPOSITS INTERFACE HAS BEEN INTERPOLATED BETWEEN ADJACENT BOREHOLES
LEGEND
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ASSUMED WATERLEVEL BASED ON PIEZOMETER READINGS
PIEZOMETER SCREEN AND WL IN DEEP (25mm) PIEZOMETER

Fig 3

SOUTHDOWN FREIGHT TERMINAL PAVEMENT DESIGN

R S Ryan BE, FIPENZ, MCIT
NZ Rail Limited
Wellington

SYNOPSIS

This paper was originally presented to an internal NZ Railway Engineering Conference in October 1977.

It outlines the research work undertaken to develop a heavy duty pavement design where the subgrade was a sanitary landfill. The paper also touches on some aspects of the construction work that followed.

Southdown Freight Terminal is NZ Rail's principal Auckland Terminal. The pavement is used by large forklifts for the transfer of freight between Road and Rail vehicles.

It was opened in 1973 and has been gradually extended as freight operations demands grew.

Over the 20 years, the pavement has performed exceptionally well. A small number of local "distress areas" have required attention during that period. A more significant failure has occurred in one location only. This is where the installation of underground services, subsequent to the original work has traversed a Roadway. Reconsolidation of bed for the services and backfill to inadequate standard are deemed to be the cause of failure.

A part from the above no other failures have occurred. Settlement have been minimal and surface discontinuities non existent.

NEW ZEALAND GOVERNMENT RAILWAYS
DISTRICT ENGINEERS' CONFERENCE :
OCTOBER 1977

A HEAVY DUTY PAVEMENT ON A SANITARY LANDFILL
OR
AN AIRFIELD ON A RUBBISH DUMP!

INTRODUCTION

Heavy mobile plant being used in Container terminals and Railway Goods Yards impose loadings on the pavement structure similar to those of modern Jet Airliners* on Airfields.

Recognition of this fact, and the lack of documented evidence of rubbish fill behaviour under dynamic loading conditions, lead to a series of experiments being performed at Southdown in 1972, to evolve a suitable pavement design for the freight terminal.

The 100 acre Southdown site, adjacent to the foreshore of the Manukau Harbour, was typical of the surrounding rugged basaltic lava flow remnants from Mt Smart. Although essentially level in contour, the large surface irregularities caused by the lava flows had been filled by tipping of 500,000 cubic yards of material over a period of 20 years to the current time, by the One Tree Hill Borough Council.

The landfill was being constructed from domestic and trade waste consisting of garbage, fibrous organics (wood, paper, synthetics), inorganics (car bodies, steel scrap, tyres, concrete and timber) plus a host of other materials including vegetation and large tree stumps. In general it was compacted with a large bulldozer in two or three foot lifts each of which was covered with soil, earth, clay or road construction wastes to minimise odour, rats, insects and windblown debris problems.

The resulting plateau was generally level, and gave no hint of the original ground form or what lay beneath the surface.

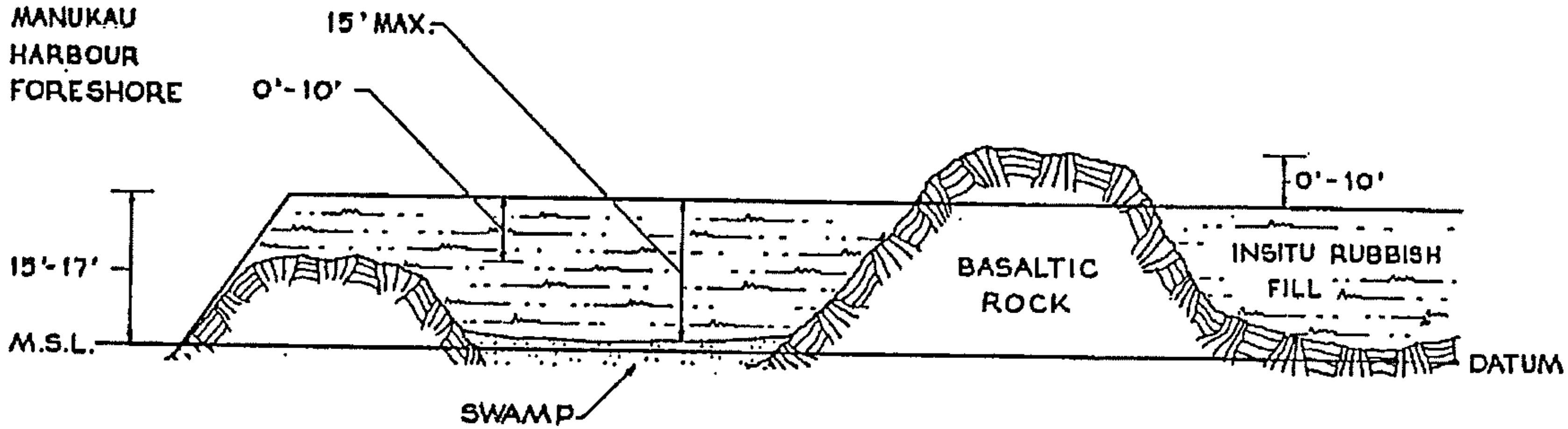
Figure 1 diagrammatically represents a typical cross section through the terminal site.

The large volume of rubbish fill precluded the option of "getting rid of it" as unsuitable, so the following account recalls some of the experimental and theoretical work that was carried out to arrive at specification requirements for its treatment to make it more acceptable as a fill material. Some comments are also included on the contract and subsequent pavement performance.

EARLY INVESTIGATIONS

The initial site investigations, consisted simply of digging inspection pits with a Beaver hydraulic excavator to assess, the condition and consistency of the fill material.

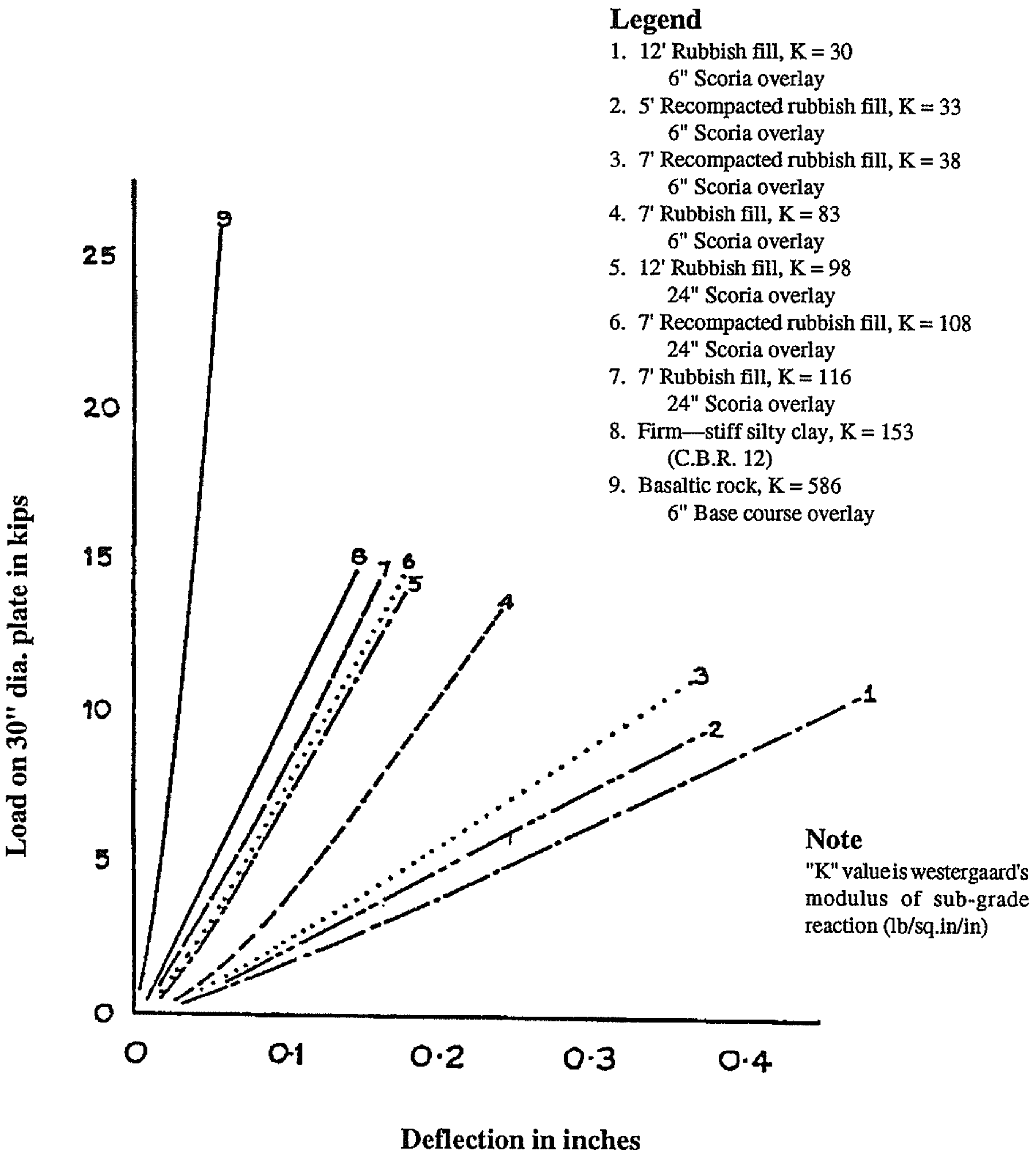
* *DC 8's and Boeing 707's at that time*



Diagrammatic sketch of typical cross-section through terminal site

SOUTHDOWN FREIGHT TERMINAL

Figure 1



Load Deflection Graphs for Various Materials

SOUTHDOWN FREIGHT TERMINAL

Figure 2

We found:

(a) it stunk!!;

as predicted from literature, and concluded that aerobic decomposition was generally complete but anaerobic decomposition was proceeding (a very slow process).

(b) extremely variable consistency;

although nominally layered, there could for example, be a truck load of car tyres intermingled with timber, overlying timber demolition material, while adjacent to it, the remnants of pure domestic trash. Density tests, based on 2 to 3 cubic yards samples revealed insitu values of 1100 to 1700 lb/cu.yd.

can you imagine trying to assess the performance of a 4 feet deep layer of "used Coffee Bean grounds" (ex Nestles) extending over 1 acre?

(c) irrespective of age;

only the putrescence had decomposed and the original material remained essentially in the form in which it had been dumped.

(d) it was "workable" and "compatible":

by putting the materials back in the holes from whence it came and in the process "pushing it down" with the excavator bucket, a 10% volume reduction was achieved. Although the small excavator and BTD.6 Drott could handle the material, it was obvious that large machinery would be required to be really effective in breaking out the sub-surface entanglements.

From these early investigations it was concluded that some homogenising and compacting operations would be required to the insitu rubbish material, to minimise differential and long term settlements.

PAVEMENT DESIGN PROCEDURES

The majority of pavement design procedures base assumptions on subgrades or formations of uniform consistency measuring the reaction to an applied load. In this situation, considering the subgrade material consistency and axle loads to be applied, the C.B.R. technique seemed inappropriate as the plunger size for this test is approximately $1^3/_4$" in diameter with an applied load of $1^1/_2$ tons. The Binkleman beam, a commonly used technique for design and rating load pavements seemed inappropriate also, bearing in mind the fourth power ratio of pavement wear caused by axle load in excess of standard design axle of 18,000 lb.

Pavement Wear by Fork Truck, per pass $= \left(\frac{120{,}000}{18{,}000}\right)^{3.85} = (6.66)^{3.85} = 2060$ times that of the Standard axle, per pass

An airfield rating system, known as the "Load Classification Number System" of Classifying Aerodromes and Aircraft, offered some potential for development as a design technique. This empirical rating system, developed in the UK from an extensive series of load tests on existing rigid and flexible pavements of varying thickness and on varying subgrades, relates the load necessary to produce failure of the pavement and the contact area over which the load is applied.

The applied loads for rating were therefore comparable in magnitude to the actual loadings the pavement was subjected to in operation and were applied through circular steel plates comparable in area to the "imprint" of the wheels operating on the pavement in the normal service conditions.

Eg: $\left(\text{Contact area} = \frac{\text{Wheel load}}{\text{Tyre pressure}}\right)$

The criterion established for "failure" recognised the fatigue characteristics of a pavement structure and was based on a "permanent set for a given number of load repetitions". Full details of the procedure and background are given in reference 1.

A second series of experiments were then conducted basically to build a trial pavement and measure its performance by plate bearing testing in accordance with the L.C.N.

TRIAL PAVEMENTS

Three test pads, each 60' long x 30 feet wide were selected for the construction of trial pavements.

The first: over 12 feet depth of insitu Rubbish fill.

The second: over 7 feet depth of insitu Rubbish fill.

The third: over solid rock on which initially a 5 feet depth of recompacted rubbish fill was built by compacting excavated rubbish material in 1 foot layers with a heavy steel vibrating roller*. This was subsequently built up to a final depth of 7 feet above the initial rock level.

The aim of these experiments was to establish the relationship (if any) between the load bearing capacity, depth and density.

*This compaction equipment was the type available to us at the time.

- Density

    800 cubic yards of insitu rubbish fill ("natural" density 1400 to 1800 lbs/cu.yd) was used to make the 7' depth recompacted test pad.

    A 27% reduction in volume was achieved while a 23% increase in density was recorded. It was considered that the correlation was reasonably consistent.

- Load Bearing Capacity

    Westergaard's modulus of Subgrade reaction was used as a comparative load bearing parameter. Full details of this modulus, its theory and establishment are contained in reference 2.

    It was generally concluded that in spite of the increase in density brought about by the recompaction of the fill, there was little increase in the subgrades support value. Refuse tip fills are extremely weak in both "natural" and recompacted states. Lines, 1, 2 and 3 of figure 2 bear this out while line 4 gives an indication of the variability in subgrade support value that can be experienced.

    Increase in subgrade support value could only be achieved by increasing the depth of the granular layer placed on top of the fill material. The consistency of Lines, 5, 6 and 7 in figure 2 indicated that the imposed loads are being supported within the overlay and that, as such, the underlying rubbish fill is being subjected principally to steady "overburden" loading. Settlement characteristics of the fill material therefore became a more important criterion than load bearing capacity of dynamic loads.

- Settlement Experiments

    Figure 3 illustrates Load Settlement curves derived from tests conducted on the site.

    (a) Recompacted Refuse Fill

    The recompacted refuse fill test pad was surcharged to approximately 500 lb/sq.ft with scoria and base course materials used as the trial pavement. Concrete weights were then added bringing the surcharge up to approximately 1000 lb/sq.ft and settlements noted. The loading portion of line (1) figure 3 therefore represents the settlement caused by increasing the surcharge from 500 lb/sq.ft to 1000 lb/sq.ft.

    (b) Insitu Fill (line 2, figure 3 refers)

    Loose earth and clay surcharge was placed over an area of insitu fill approximately 60' x 25' with average depth of rubbish to 10'. Settlements were recorded at four points on the overburden/rubbish interface. Line 2 figure 3 represents the average of these readings.

    It was generally concluded that the increase in density caused by the recompaction, greatly reduced long term settlement characteristics of the refuse material.

    Irregularities in the settlement curves are due to wetting and drying of the surcharge material caused by weather conditions.

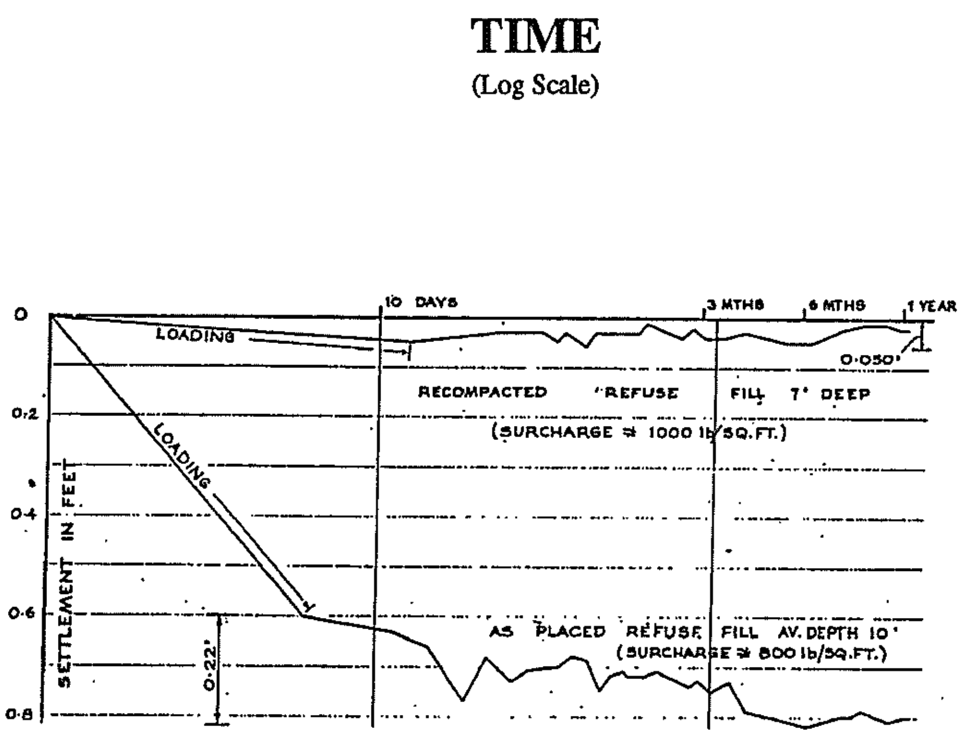
ADAPTION OF L.C.N. SYSTEM OF PAVEMENT RATING INTO A DESIGN PROCEDURE

The Field testing for the L.C.N. rating procedure is a time consuming, repetitive loading system for full depth pavements. Each test takes in the order of 5-6 hours to complete. The analysis of the loading and settlement observation from the procedure gives a "safe load for a given Load Contact area". Although for rating purposes this test is done at the pavement surface, at Southdown, L.C.N. tests were conducted at various levels as the trial pavement was built up in layers. Using the assumption that the stresses from a load on any contact area disperses through the underlaying material in a 45° frustum, predictions could be made from results at any layer, the additional depth of material above that layer that would be required to ensure the safe load at the test layer was not exceeded.

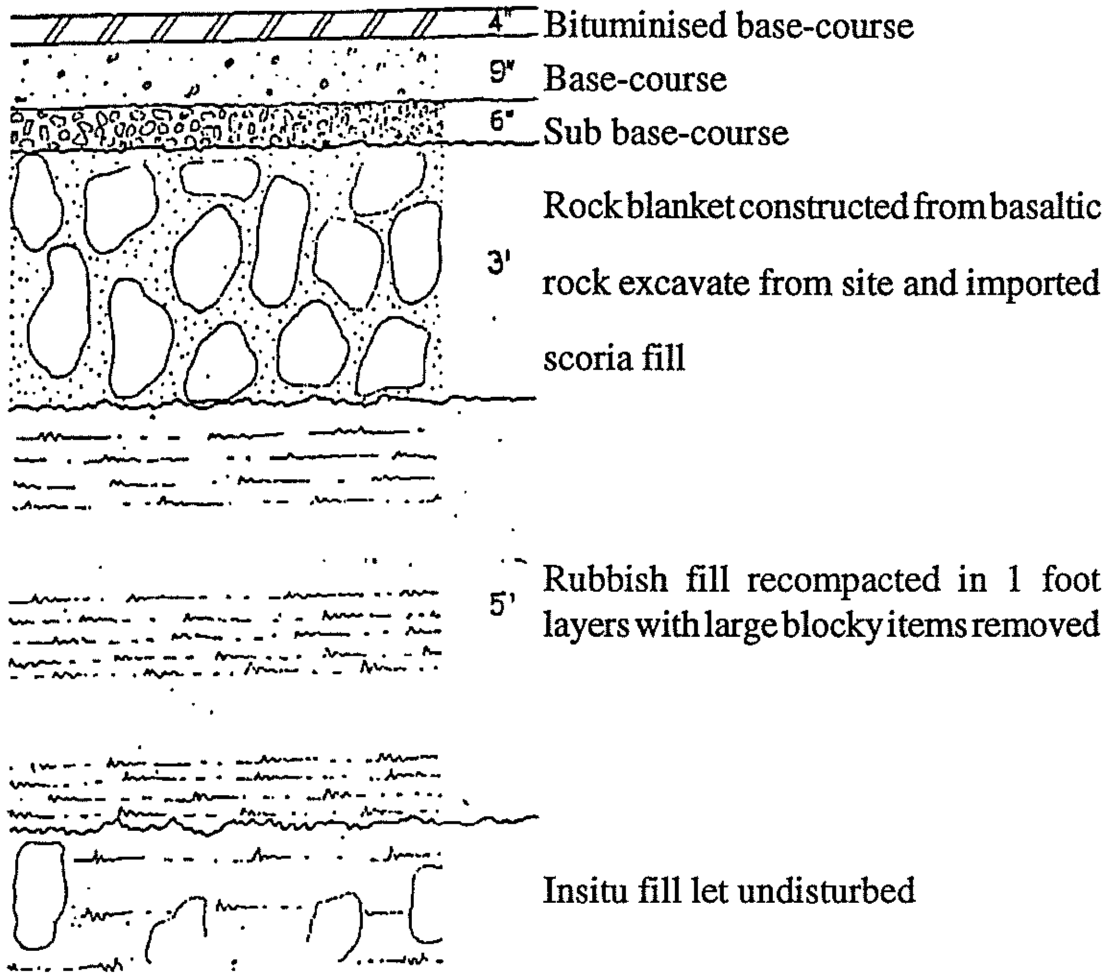
The set of Design Charts in NZR Way and Works Branch Engineering Circular Memorandum 1972/24 for varying axle loads and subgrade strengths were compiled on this basis using the appropriate and intricate mathematical processes explained fully in reference 1. A copy of a typical design chart has been included as Appendix 1. Four "K" value tests were conducted (duration $^1/_2$ hour each) around each L.C.N. test. These values were noted essentially for use as "consistency" parameters, in the full scale pavement construction.

DEVELOPMENT OF SPECIFICATION REQUIREMENTS

The conclusions drawn from the experimental work lead to the following requirements being included in the specification for the contractual works. Figure 5 illustrates the requirements diagrammatically.



Load–Settlement Graphs
SOUTHDOWN FREIGHT TERMINAL
Figure 3



Typical Cross-section through Rubbish Fill and Pavement
SOUTHDOWN FREIGHT TERMINAL
Figure 4

(a) Treatment of Rubbish Fill

Where the pavement structure overlay rubbish fill, all rubbish, to a maximum depth of 5 feet below the rubbish/pavement interface, would be excavated "homogenised", and recompacted as "optimum" moisture content with heavy rollers in 1 foot layers. Large blocky items (old stoves, fridges, tree stumps and the like) were to be excluded from these layers. Rubbish below that 5' maximum depth would be left undisturbed. The recompaction was to improve density, reduce ultimate settlement while the exclusion of blocky items would eliminate discontinuities appearing at the surface created by differential settlement.

In effect a raft was crated, so that any settlements as a whole would be dished rather than sharp discontinuities.

(b) Rock Blanket

The L.C.N. Tests indicated that a granular depth of 48 to 54 inches was required above the rubbish fill to adequately withstand the Fork truck loadings. A "rock blanket" 36" deep, constructed from basaltic material excavated from the site and mixed with imported scoria form the base layer of the pavement and a substantial "raft" over the rubbish fill.

(c) Pavement Layers

Above the rock blanket, sub-base, base course and asphalted layers were placed in accordance with normal N.R.B. specifications, except that the thickness of asphalted concrete was in accordance with recommended airfield practices for specific wheel loads. (Ref. 1, p.342)

INFORMATION FOR TENDERERS

The reworking of some 250,000 cubic yards of rubbish material as a contract requirement was something new for the "industry" in the Auckland area. To assist them in compiling contract prices, prospective tenderers were invited to the site during the last week of the tendering period to observe the Department undertaking further exploratory and site investigation work.

A D7 dozer and sheeps-foot roller was employed *"pushin it! shovin it! spreadin it! compactin it, and all those sort of things; a neat little Hymac was standin there a-digging it, all in the presence of the Engineer!!!"*

Feed back from all the tenderers indicated that the "Field-Day" had been a valuable aid to them in assessing pricing. Workability, variability of the rubbish content and moisture content, the principal factors in establishing the unit rate, had been clearly demonstrated.

QUALITY CONTROL CHECKS IN THE CONTRACT

Quality control checks written into the contract for the compaction of the rubbish fill and rock blanket were based on achieving a certain minimum "K" value. These values had been arrived at by way of experiment at the trial pavement site.

The use of this parameter as a contract quality control measure was not satisfactory however, because of the time required to perform each test ($^1/_2$ hour min) and the vast area (60 acres for the Rock Blanket 15 acres of the full pavement structure) involved.

In hindsight, a minimum density requirement for the recompacted rubbish fill or a proof rolling technique (as we later developed with the co-operation of the contractor) would have been more satisfactory quality control measure.

The moral of this story is then:

> "Don't expect experimental test techniques to be adequate or practical construction quality control measures."

Moisture content was an important factor in achieving maximum density. The contract documents called for the "rubbish fill to be compacted at the optimum moisture content" — written somewhat with tongue in cheek — for how does one determine the moisture content of a heterogeneous material with elements such as old tyres, car bodies, timber off-cuts, household refuse, trade wastes and the like? The Contractors operators soon became very proficient and developed a good "Sense of feel" mixing wet and dry materials to produce well compacted reworked rubbish.

Too wet: excessive heaving

Too dry: excessive "springing"

Just right: minimum "spring"

PAVEMENT PERFORMANCE

After three years in operation, the full depth pavement was subjected to a series of five complete L.C.N. rating tests in an attempt to correlate as built "Rated Capacity" with that "Predicted Rating" prior to construction (design stage). Three tests were conducted on the pavement where no distress was evident. The analysis of results has shown that a higher pavement strength has been achieved than initially expected.

Using the UK Air Ministry criteria for evaluation of airport pavements to permit a 55,000 lb capacity forklift to operate at maximum capacity for an unlimited number loading cycles on the pavement, a L.C.N. value of the pavement at Southdown would need to be in the vicinity of 80. The most pessimistic prediction in the test results rated the pavement at 98 with the average value being around 115. Appendix 2 lists the UK criteria for aircraft pavement use using the LCN method.

Two tests were conducted where surface cracking of the pavement was evident. Fortunately, this was a small area approximately 50 feet long by 15 feet wide. During construction, an area of clay, previously undetected was discovered when cutting the rock down to grade. Although undercut and backfilled at the time, it was now evident that insufficient drainage was provided to prevent the top layer of the clay "pugging" under the cyclic pavement loading. From test bores and analysis of the L.C.N. plate bearing tests, it was concluded that the surface cracks in the asphalted top layer of the pavement developed as a result of excessive strains generated in the "pug layer" (or film) **BUT** these strains having been taken up during the loading cycle the pavement structure then continued to perform in the manner predicted by the design method.

SETTLEMENT OF RUBBISH FILL UNDER PAVEMENT ROCK BLANKET

Of the 100 acres Southdown site, approximately 60 acres were developed to the stage of completing the rock blanket. Of the 60 acres only about 20 acres have the full depth pavement constructed on them at this stage.
Although no monitoring of the rubbish fill areas for settlement has been undertaken since completion of the contract, observations has lead to the conclusion that there is negligible settlement under the pavement or any of the rock blanket areas.

CONCLUSIONS

The pavement design for Southdown developed from the blending of simple theory, with experimental results and common sense. Its performance to date has been satisfactory and subsequent testing indicates that it should continue to give maintenance free performance for a considerable time.

Since embarking upon this research project in 1972 with its completion in 1973, considerable literature has been published on other design methods and rating of heavy duty pavements. My readings of these, and the practical experience gained at Southdown make me realise that, because of the number of variables involved, qualities of materials used and the practicabilities of construction, great precision in pavement design in unjustified. Engineering assessment and judgment is still of prime consideration in this sphere.

ACKNOWLEDGEMENTS

The successful completion of the experimental and contract work would never have been achieved without the able assistance and encouragement of many people but particularly those mentioned below who held the position recorded at that time:

Peter Vink, Structural Engineer; whose analytical mind never failed to produce a suggestion for yet another series of tests to track down some unexplainable result.

Ken Schmidt, Graduate Engineer; whose diligent resolve was tested to the limit attempting to blend experimental results into quasi-theoretical predictions.

John Keen, Draughtsman; whose practical philosophy of "it looks right" was of great encouragement when the figures didn't add up.

Derrick Mead, Draughting Cadet; whose opportune dry comments belied his quiet and conscientious manner.

The other great "behind the scenes" resource was the Railway Library Staff who chased relentlessly those obscure references that I thought may throw a glimmer of light onto the characteristics and behaviour of those piles of rubbish.

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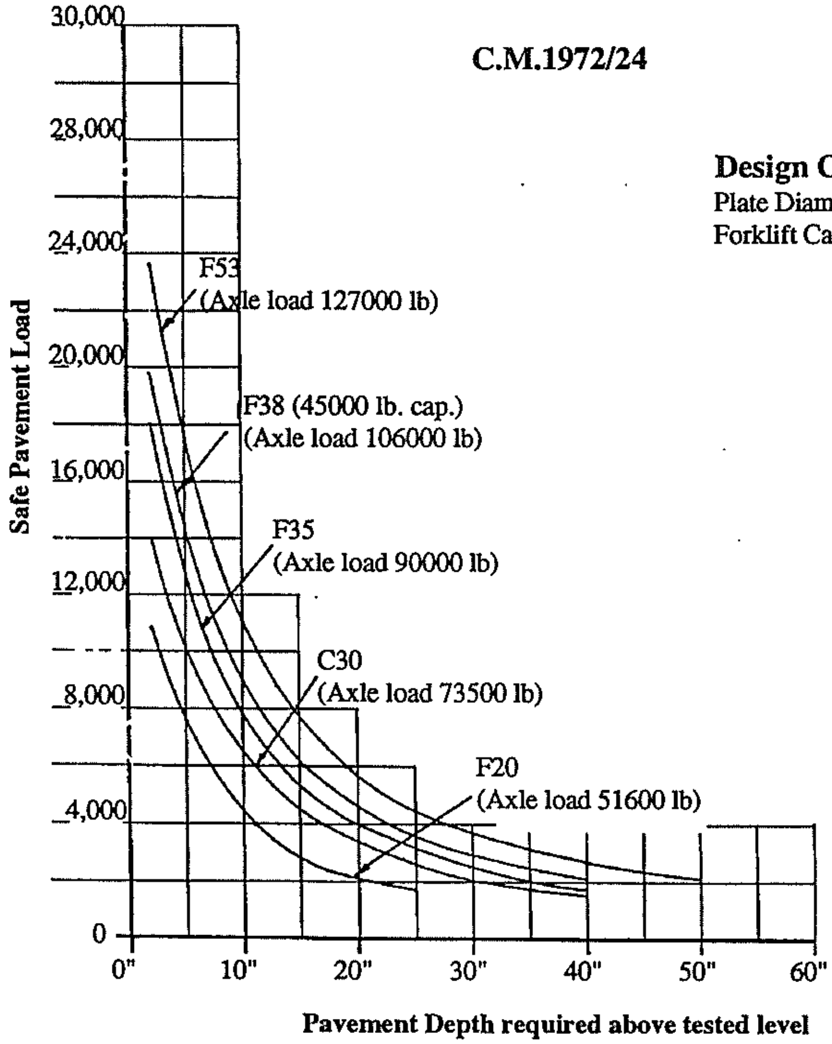
APPENDIX 1 : DISTRICT ENGINEERS' CONFERENCE (1977)



Design Chart No. 8
Plate Diameter: 18"
Forklift Capacity: 100%

APPENDIX 2 : DISTRICT ENGINEERS' CONFERENCE (1977)

SOUTHDOWN FREIGHT TERMINAL PAVEMENT
The United Kingdom criteria for aircraft pavements are:

(i)	if	LCNv < LCNp	1.1	the pavement is suitable for unlimited use by the particular vehicle.
(ii)	if	LCNv = LCNp	1.1 to 1.25	then 3000 movements of the vehicle can be confidently expected. Above this number of movements heavier maintenance commitments can be expected and some minor failures.
(iii)	if	LCNv = LCNp	1.25 to 1.5	then up to 300 movements (spread in time) may be planned. Some local failures of the pavement may be expected.
(iv)	if	LCNv = LCNp	1.5 to 2.0	only limited use of the pavement by the vehicle should be permitted.
(v)	if	LCNv = LCNp	2.0	the pavement should be used only in emergencies by the particular vehicle.

Where: LCNv = Calculated value for aircraft (or vehicle in NZR case).
LCNp = Assessed value for pavement performance from field tests.

SOME ASPECTS OF LAND TREATMENT OF CONTAMINATED SOILS

G T Proffitt
Pattle Delamore Partners Limited
Wellington

K G Delamore
Pattle Delamore Partners Limited
Auckland

ABSTRACT

Aerobic bioremediation of soils by land treatment is an accepted and economical way of dealing with a wide range of organic contaminants. This paper presents some background to the bioremediation process, some practical aspects of land treatment and then presents some examples of the land treatment of hydrocarbon contaminated soils in the context of the earlier discussion.

INTRODUCTION

Aerobic bioremediation of soils is an accepted and economical means of dealing with a wide range of organic contaminants. The process involves encouraging indigenous or introduced microorganisms to multiply, using the organic contaminants as their source of carbon and energy, while in the process converting the contaminants into carbon dioxide and water. For the microorganisms to flourish they need, apart from the carbon source, appropriate quantities of oxygen, water and nutrients and physical conditions appropriate to their survival.

Land treatment is the simplest form of bioremediation, being, in its most basic form, insitu aeration of the contaminated soil by cultivating with agricultural equipment, while ensuring that the moisture content is appropriate. It is a low technology, low energy, easily implemented and therefore low cost solution. In its more complex form, land treatment involves careful investigation of the soil and its contaminants, including laboratory and pilot scale remediation trials, and then designing and constructing a treatment cell which may be fully contained and have such things as:

- forced, negative pressure aeration
- odour and volatiles control
- automatic irrigation with nutrients added
- leachate control and recycling
- frequent progress monitoring

In between there is a wide range of economical solutions appropriate to New Zealand conditions and its regulatory environment.

This paper presents some background to the process of bioremediation, some practical aspects of the land treatment process and then some results of dealing with hydrocarbon contaminated soils.

BACKGROUND

Simply going out and stirring up some contaminated soil seems a very attractive way of dealing with a problem. However, some understanding of the processes involved in bioremediation and land treatment may be useful in deciding whether such an approach is going to be effective in a particular situation.

Land treatment is actually a combination of several physical, chemical and biological processes which are more or less important depending on the type and range of contaminants present, the physical and chemical characteristics of the soil and the design and operation of the treatment process. The processes that are involved are:

- photolysis
- chemical oxidation
- sorption (adsorption / desorption / dissolution / diffusion)
- volatilisation
- biological oxidation

While degradation by ultraviolet light (photolysis) may be important for some compounds and chemical reactions can play an important role for others, the most significant mechanisms are sorption, volatilisation and biological oxidation (Matthews and Pope, 1992, Smith et al, 1990). The latter three interrelated mechanisms are shown schematically in Figure 1.

Figure 1 models the soil as a collection of porous, water stable aggregates, 1 to 250 microns in diameter, one of which is shown. Soil water can exist in micropores between the aggregates, as bulk soil water, or within the micropores as pore water. In this model, the organic contaminants can exist in three states – bound (adsorbed) to the soil, in aqueous solution, either in the micropores or the bulk soil water, or in a free phase state. As biodegradation is mainly a water based process, the contaminant (C_s) must get from the adsorbed and free phase states into the aqueous state (C_l) and then to a position where it is available to the microorganisms. The micropores are too small to allow the entry of the microorganisms therefore a process of adsorption/desorption, dissolution and diffusion is required to transport the contaminant to the soil aggregate surface. It is then available to be biodegraded by bacteria or volatilised from the aqueous phase into the gaseous phase (C_g).

Whether a compound is volatilised or biodegraded depends largely on its solubility and volatility. In general, volatilisation is a significant loss pathway for low molecular weight compounds. Depending on the regulatory environment, this may necessitate designing the land treatment operation to control and/or treat volatilised compounds.

Once in solution, many organic compounds are biodegradable provided suitable microorganisms are present and conditions are favourable to the organisms. Rates of degradation depend on the chemical structure of the particular compound. For example, rates are typically slower as the number of attached chlorine atoms increases and for high molecular weight aromatic compounds, but conversely, faster for long straight chain aliphatic compounds compared with shorter chains, (NCASI, 1990). However, the rate limiting process may not be that of biodegradation but of bioavailability through the mass-transfer processes of desorption/diffusion. For example, for multiring polycyclic aromatic hydrocarbons (PAHs), degradation is limited by PAHs having low solubility and being strongly adsorbed onto the soil matrix. Much research effort is being expended in trying to find ways of increasing the bioavailability of such compounds.

The actual process of biodegradation is very complex and not the subject of this paper. Typically, several organisms are involved, with both bacteria and fungi being active although most research has been carried out with bacteria (Matthew and Pope, 1992). Biooxidation may occur as a number of sequential steps for some compounds while other compounds may not be capable of fully satisfying the carbon requirements of the microorganisms and require co-metabolism in the presence of other carbon sources.

Certain bacterial species are prominent in biodegradation but they must be adapted to the particular chemical of concern. (Hohn and Loehr, 1992). In land treatment it is generally assumed that appropriate bacteria are already present and simply need encouragement to multiply. Addition of bacterial cultures is not recommended because such cultures are unlikely to be able to compete with the indigenous species (Matthews and Pope, 1992).

PRACTICAL APPLICATION

Having gone through the process of investigating a site, confirming that it is contaminated, identifying the contaminants and confirming that they are biodegradable, one is then faced with carrying out the task. Ideally, this can be carried out in-place, but frequently the soil must be excavated and taken to a specially prepared treatment cell or reactor bed. Depending on the circumstances, this may be a bunded, lined and drained cell, with forced aeration, automatic irrigation and drainage monitoring systems or simply a matter of spreading the soil in a thin layer over a drainage layer laid on the natural ground. Air is then introduced by regular tilling full depth with a rotary hoe. When remediation of this layer is complete, a second lift can be placed and so on. The complexity of the design will depend on many things, including the characteristics and concentrations of the contaminants, the soil characteristics, the site location, perhaps the climate and, of course, the regulatory environment. General requirements rather than specific design details will be explored in this paper.

Of greatest importance is the maintenance of optimum moisture conditions and sufficient aeration. In the soil environment the two are inversely related. Too much water may mean too little oxygen and vice versa. Ideally, there must be as much water as possible to provide dissolved contaminant to the bacteria, while not so much as to exclude the oxygen. As a guide, Matthews and Pope (1992) recommend the moisture content be between 40% and 70% of the field capacity, the field capacity being the moisture content at which the micropores are full of water but the macropores are drained. Short periods of either too dry or too wet can seriously inhibit bacterial activity.

Regular testing of the moisture content is necessary to confirm conditions but as a rule of thumb, if the soil looks moist (a gardener's eye helps) but the soil is not so wet as to make tilling difficult, then the moisture content is probably within the desired range.

Moisture control can be a problem, particularly as in our experience, budgetary constraints prevent a sophisticated approach and the site is remote from the office preventing frequent inspection. Reliance on a contractor becomes necessary. We have successfully used two approaches.

The first approach is to leave the cell uncovered, relying on rainfall plus the application of additional water with sprinklers if testing or the contractor's experienced eye indicates this is necessary. Problems arise if the contractor is not very diligent or the soils have poor water retaining properties and dry out rapidly. A further problem is that frequent rainfall is likely to make the soil too wet. This has several effects. Firstly, oxygen is excluded if the soil is saturated. Secondly, tilling may not be possible or is inadvisable and must be delayed until the cell is sufficiently dry. Thirdly, constantly changing conditions require constant adaptation by the bacteria to the detriment of the remediation.

The second approach is to cover the cell. We have adopted this approach where moisture retention was likely to be a problem and have found polyethylene silage covers to be cost effective. The covers maintain a more even moisture content, both spatially and temporally, by preventing surface drying and excluding rain, provided they are maintained in a good condition and attention is paid to surface drainage. Allowing rain to pond on the cover will inevitably lead to the creation of wet spots. These are generally not discovered until the cell is next due for tilling, at which time they cause problems with tractor bogging and compaction of the soil. The solution is then to leave the cover off for a while, running the risk of further rain or overdrying of the parts of the cell that were at the correct moisture content. Obviously, the best approach is to ensure adequate surface drainage from the outset and then maintain the covers in a good condition.

Covers have a number of advantages and disadvantages other than their use for moisture control. They are useful for controlling objectionable odours but on the other hand will prevent significant remediation by volatilisation. Surface photolysis will also not occur. There will be less interchange of oxygen at the soil surface but in our experience, aeration of the soil mass is adequate provided the cell is uncovered and tilled regularly, typically weekly or fortnightly.

In addition to oxygen and water, microorganisms require nutrients, principally organic carbon, nitrogen and phosphorus but trace amounts of other elements are also required. Testing for the major nutrients is recommended as part of the initial setting up of the treatment cell. Fertiliser can then be added with the aim of achieving a C:N:P ratio of 100:10:1 (Matthews and Pope, 1992). In our experience, this is seldom achieved in the often poor soils requiring remediation. Nitrogen is typically deficient but to increase to recommended levels would require the addition of such large quantities of fertiliser that microbiological activity is likely to suffer. An analogy is the burning off or wilting of plants in an over fertilised garden. Instead, we work on the assumption that the indigenous bacteria are adapted to the local conditions and just require a moderate boost, much as would be given to a garden or pasture.

This approach has been borne out by routine monitoring of microbiological activity using plate counts. This involves incubating samples in a nutrient medium and counting microorganism colonies. The test is not specific to any particular microorganisms but the assumption is that if the counts are high then conditions are satisfactory for the microorganisms active in bioremediation.

Getting the oxygen, water and soil nutrients to appropriate levels is not the end of the story, as biological activity is affected by other aspects of the soil. On many contaminated sites, a range of contaminants may be present that may inhibit bioremediation, for example some pesticides, high concentrations of metals or substances that raise or lower the soil pH. Microorganisms prefer conditions near neutral, around pH 6 – 8. This may be adjusted, if necessary, by the addition of appropriate materials.

Metals will not be transformed in the same sense as organics but valence states may be altered or chemical bonds broken so as to change the toxicity or mobility of the original compounds. A small laboratory trial is appropriate where there is doubt about whether a particular soil is remediable.

Appropriate remediation targets must be set taking into account such aspects as eventual site use, groundwater quality protection and regulatory requirements. Monitoring of the treatment performance against this target will be required. Care must be taken that sampling is representative.

Before moving on to some examples of bioremediation, it is worth commenting on some of the practical aspects of tilling the soil. A typical agricultural hoe cannot penetrate any deeper than 250mm. Soil should not be placed any deeper than the capacity of the hoe. When placing the next lift of soil at the completion of a lift, it is important to penetrate into the top of the previous lift, not only to limit the development of a hard, poorly drained layer, but also to mix soil that already contains high microorganism populations into the new lift.

Soils to be remediated are frequently the subsoils, often clayey and difficult to till. As for a garden, addition of bulking agents may improve workability. These may be in the form of organic material such a sawdust, straw or animal manures or inorganic such as sand. Organic bulking agents can be useful where organic carbon is deficient and animal manures also add nutrients, but such materials may also add a biological load which, in some circumstances, may detract from the remediation of the organic contaminants. Large quantities of organic material will obviously increase the total volume of soil to be disposed of and may make remediated soil more difficult to compact for on-site disposal.

If an improvement in workability is the only objective, then we have found the addition of fine sand to work well with clay soils.

Another soil problem that is often found is a high proportion of oversized material. This is frequently demolition rubble from when the site was cleared. Rotary hoes will not cope with material larger than about 75mm - 100mm.

For small jobs, hand picking may be appropriate but for larger jobs prescreening and/or the use of a harrow type cultivator have been employed. Prescreening has the advantage of aerating the soil and encouraging volatilisation.

Finally, a word about health and safety. The average contractor tends to regard a land treatment job as just another small scale earthworks operation. He may need reminding that the soil contains contaminants that may be hazardous to health and require appropriate precautions to be taken. Typically, a health and safety programme is developed prior to the site work. The programme may include worker protection such as clothing, goggles and dust mask and operating constraints such as noise suppression and odour emission controls.

EXAMPLES

Some examples of remediation by land treatment are useful in illustrating some of the concepts described above.

Figure 2 shows the reduction of petroleum hydrocarbons from two land treatment sites. Both sites involved leakage or spills of petroleum products into the ground causing moderate contamination. Both soils were fine grained silts and clays with a tendency to be difficult to work when wet and to bake hard if allowed to dry.

Simple treatment cells were set up for both jobs with weekly tilling being carried out for the first 4 weeks followed, if necessary, by fortnightly tilling until completion. One cell employed a cover to improve moisture control while the other remained uncovered.

Both sites showed a rapid drop off in total petroleum hydrocarbon (TPH) concentrations but the uncovered cell proceeded at twice the rate of the other. This is not thought to be related to biological activity but rather that this soil contained a higher proportion of volatile components which were able to escape from the uncovered cell. Subsequent soil lifts (not shown in Figure 2) containing less volatile hydrocarbons have shown slower rates of remediation more like those of the covered cell.

The second example, Figure 3, compares the remediation of polycyclic aromatic hydrocarbons (PAHs) contained in two soils from a gas works site. PAHs are a major constituent of coal tar. One of the soils, Soil 1, also contains metals and cyanide compounds. Both soils were treated under the some conditions and had similar nutrient levels and moisture contents. Soil 1 tended to be clayey while Soil 2 was sandy.

Two effects can be seen in Figure 3. Firstly, the effect of increasing complexity of the compound. The lightest compound, phenanthrene, is a three ring PAH with a molecular weight of 178. The heaviest, benzoperylene, has 6 benzene rings and a molecular weight of 276. The other two compounds have intermediate properties. Solubility decreases 4 orders of magnitude from phenanthrene to benzoperylene. Bioavailability can be expected to decrease accordingly. This is reflected in the results, with the lightest two compounds, regardless of soil type, showing a significant reduction in concentration over 4 weeks while the heaviest two compounds showing no degradation (the apparent slight increase can be put down to sampling differences).

Secondly, for the compounds where degradation did occur, significantly more occurred for Soil 2 than Soil 1. This is probably related to the different characteristics of the two soils. Adsorption on to the soil will be stronger with the clay of Soil 1 with the result that the contaminant can be expected to be less bioavailable compared with the sandy Soil 2. In addition, the metal and cyanide content of Soil 1 may be inhibiting microbiological activity. This is supported by Soil 1 having lower microbiological counts than Soil 2.

SUMMARY AND CONCLUSIONS

Bioremediation using the land treatment technique is an appropriate low cost technology for many organic contaminants. However, its success is not just determined by whether a particular compound can be biooxidised but by a complex interaction of physical, chemical and biological processes that determine whether the compound is bio-available. Contributing factors are the physical and chemical characteristics of both the soil and contaminant. When carrying out land treatment it is necessary to consider these factors and modify the approach taken accordingly.

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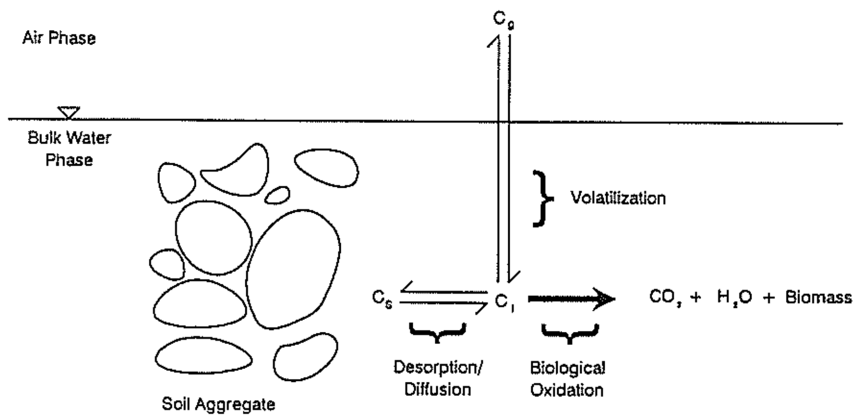

Figure 1 - ROLE OF DESORPTION/DIFFUSION IN BIODEGRADATION (After Smith, et al)

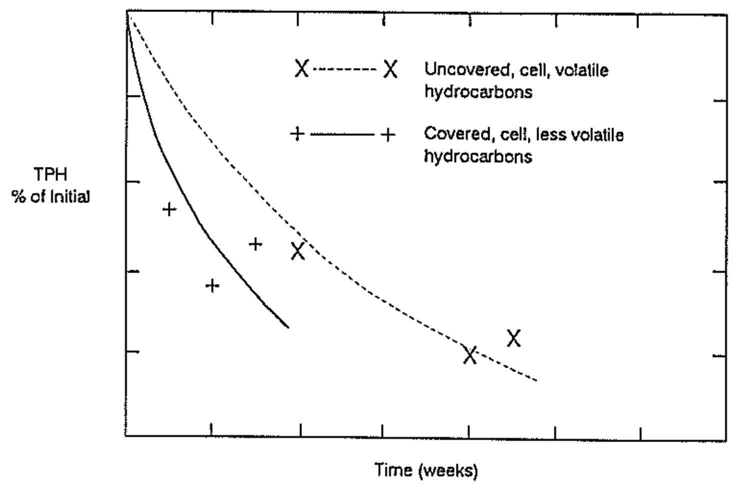

Figure 2 - PETROLEUM HYDROCARBON REMEDIATION COMPARISON

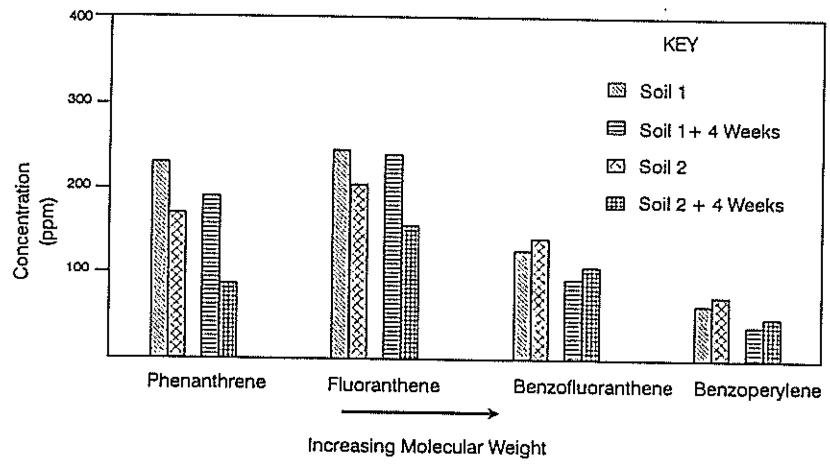

Figure 3 - COMPARISON OF DEGRADATION OF POLYCYCLIC AROMATIC HYDROCARBONS

GEOTECHNICAL AND ENVIRONMENTAL ASPECTS OF MINING AND QUARRYING

David H Bell
Senior Lecturer in Engineering Geology
University of Canterbury
Christchurch

SYNOPSIS

Mining and quarrying are amongst the earliest industries, with widespread recovery of both metals and non-metals predating Greek and Roman civilisations. Modern mining practices place considerable emphasis on environmental management of wastes and site restoration to an appropriate end-use, whilst geotechnical data input is necessary for mine and quarry planning, operation, and to ensure that environmental standards are met. In New Zealand mining and quarrying are principally controlled by the Crown Minerals Act 1991 and/or the Resource Management Act 1991, although high standards of geotechnical and environmental performance were achieved prior to the present legislation. Major metals currently mined are gold, silver and ironsands, whilst non-metallic products include sand and rock, limestone, clay, and coal. Future developments are likely to include expanded gold and coal production and continued demand for non-metals, with increasing geotechnical data input to meet the environmental standards required for the industry.

INTRODUCTION

The earliest use of both nonmetals and metals predates recorded civilisation, and the mining and quarrying industries are amongst the oldest of professions. Nonmetallic substances (such as flint, obsidian and soapstone) were initially used for weapons and utensils, whilst the extraction of clay for pottery, brickmaking and writing tablets was the first large-scale mineral industry. The oldest form of mining was practised by the early Egyptians for gems and decorative stones, and the use of natural stone for building purposes was widespread amongst the ancient civilisations of Egypt, the Indus Valley and Mesopotamia. Jensen & Bateman (1979) note that four metals (gold, copper, tin and iron) were recovered and used in pre-Dynastic times (ie before 3500 BC), and that the development of smelting techniques had resulted in widespread production of copper-tin bronzes by about 2500 BC. Thus most common nonmetals and metals (with the main exception of zinc) were both known and utilised prior to the Christian era, and the rise of Greek and later Roman civilisation was certainly facilitated by the availability of the various minerals and rocks and the technology to recover them.

Mineral recovery in ancient times involved both surface and underground extraction without the benefit of explosives or other technology readily available today. The Egyptians mined emeralds to a depth of 240m near the Red Sea, and the Greeks and Romans mined silver to similar depths using fire-setting and iron tools to form shafts and drives (Bieniawski, 1984). The Romans also became expert in driving tunnels for a variety of purposes other than mining (such as water supply and transportation), and a number of their aqueduct tunnels are still in use. The first systematic documentation of mining practice and geological controls of ore occurrence was provided by Agricola (1556), and modern concepts are reviewed by Peters (1978), Jensen & Bateman (1979), and Edwards & Atkinson (1986).

Modern mining practices have evolved through the ready availability of explosives and the use of materials such as tungsten carbide in high-speed drill steels. A century ago extensive use was made of timber supports for underground openings, and the mining practices then in use are detailed in Peele (1941). Since the mid-1940s the use of rockbolts for roof support has revolutionised the mining industry, and the ready availability of mechanised boring machines and automated systems for mineral extraction has resulted in significant productivity improvements (Bieniawski 1984). Long the province of the mining engineer, geomechanics aspects of mining such as ground support, pit slope stability and tailings dam design have increasingly involved specialist input from both geotechnical engineers and engineering geologists.

The waste products from mining and processing operations (such as mine overburden and mill tailings) were frequently disposed of in a manner that minimised extraction costs and did not consider ecosystem damage or pollution of land, air or water resources. During the past 25 or so years, however, there has been a dramatic growth in awareness of environmental matters, both in terms of public

TABLE 1

PRINCIPAL MINING AND EXTRACTION TECHNIQUES FOR SOLIDS

	Type of Mining	Extraction Technique		Deposit Examples
	PLACER	Hydraulic mining		Gold-bearing gravels
		Dredging	A. Suction	Beach sand heavy minerals
			B. Bucket-Line	Alluvial tin or gold
SURFACE		Mechanical	A. Dry Mining	Alluvial diamonds; gravel aggregates
			B. Wet Mining	Gold or heavy minerals
	OPEN-CUT	Open-pit mining		Disseminated gold or copper ores
		Opencast/Strip Mining		Coal extraction
		Quarrying		Non-metallic materials (eg limestone)
	UNDERGROUND	Naturally-supported stopes		Coal extraction by room-and-pillar method
SUBSURFACE		Artificially-supported stopes		Most metals eg Pb-Zn ores
		Caving methods		Most metals; longwall coal extraction
	SOLUTION	Melting		Sulphur recovery by Frasch process
		In Situ Leaching		Recovery of copper and uranium

perceptions and legislative requirements, and the short- and long-term impacts of mining have come under close scrutiny. The mining and extractive industries have in general responded very positively to these issues, and it is now standard practice for companies in Australia to prepare environmental management plans prior to commencing a major project (Vines & Ward, 1993). In New Zealand resource consents must be obtained as a condition of licensing, and strict controls exist to minimise any long-term environmental damage.

MINING METHODS

The principal surface and subsurface techniques for mining and extraction of solids are summarised in Table 1. Recovery methods for fluids such as oil and gas have not been included but are also clearly important in terms of environmental management. The adoption of a particular mining method is dependent on a variety of factors, with the economics of recovery (including waste disposal) being the ultimate determinant. Both geological and geotechnical factors strongly influence the economics of mining because the nature and geometry of the ore deposit and its overburden, together with the presence or absence of groundwater, are critical to optimum extraction. Current terminology distinguishes between resources, which are in situ mineral occurrences, and mineable ore reserves to which various economic, mining, metallurgical, environmental, social and governmental factors have been applied. In Australia and New Zealand mineral resources are reported as measured, indicated or inferred depending on the extent of geological control available, and ore reserves are classified as either proved or probable (AusIMM/AMIC, 1989).

Surface mining methods have been subdivided into placer and open-cut on the basis of the deposit origin and the consequential type of extraction technique

(Table 1). Placer deposits consist of unconsolidated materials in which denser minerals have been concentrated by a variety of fluvial or marine processes, and typically include occurrences of gold and tin (as cassiterite - SnO_2) in alluvial gravels as well as concentrations of economically important minerals such as zircon ($ZrSiO_4$) and rutile (TiO_2) in beach sands. Many placer deposits were formerly mined by hydraulic means, in which a high pressure jet of water was directed onto a face and the economically important mineral(s) recovered by a system of riffles or a similar gravity concentration method: because of the resultant damage to land and water resources from this extraction method, hydraulicking and sluicing is seldom permitted today (Shackleton, 1986). Dredging is a method of placer mining which uses either a suction-cutter system or a mechanised bucket-line (or bucket-ladder) for recovery, usually with onboard processing and continuous tailings stacking: New Zealand was in fact a pioneer in the design and construction of dredges, with the first in the world recovering gold from the Clutha River (Otago) in 1864 (Macdonald, 1983). Bucket-line dredges can work in water depths of 30-50m with throughputs up to $5x10^6$ m^3 per annum, and suction-cutter dredges typically operate in water depths up to 35m with capacities exceeding 2,000 m^3h^{-1}. With the availability of large capacity mechanical excavators, either tracked or rubber-tyred, use is now commonly made of conventional earthmoving equipment for both dry and wet mining operations (ie above and below the water table): as an example, extensive use is made of floating concentrating plants fed by mechanical excavators for gold recovery from alluvial deposits in various parts of the South Island.

Open-cut mining refers to surface operations in which an economic deposit at shallow depth is exposed by the stripping of overburden, and extraction typically proceeds by a series of benches with or without partial backfilling of the mined-out area by waste material. Depending on the geology, geometry and geotechnical characteristics of the waste and ore being mined, open-cut methods can be broadly subdivided into open-pit mining, opencast/strip mining and quarrying (Table 1). Open-pit mining is a term generally applied to the recovery of metalliferous ores, either from a conical or spiral-shaped pit or by a series of benches that advance laterally or pivotally (Shackleton, 1986): ore recovery typically involves waste stripping, drilling and blasting, loading onto trucks or conveyors, and transfer to the mill site for further processing. Opencast mining is a term applied to bedded deposits, in particular coal, and extraction can proceed either by benching (as in open-pit operations) or by strip-mining: in the latter case large volumes of overburden are removed by dragline shovel to expose the flat-lying coal in a narrow slot, with overburden replacement after extraction as part of the land restoration process (Thomas, 1978). Quarrying refers to the recovery of dimension stone or crushed rock for a variety of purposes, and worldwide this is economically more important than either open-pit or opencast mining (Peters, 1978).

Underground mining takes place when the waste-to-ore stripping ratio is too great for economic extraction by surface methods, and where the ore grades are sufficiently high to justify the added costs of recovery. As with surface mining, the geology, geometry and geotechnical properties of the deposit dictate subsurface mining methods: a broad distinction can be made between thin, laterally extensive deposits such as coal seams, and those with a much greater three-dimensional extent such as massive metalliferous orebodies. The use of underground mining methods requires access from the surface by adits or drives, by declines, or by vertical or inclined shafts: these are necessary for physical accessibility to the orebody, mine ventilation and drainage, and ore haulage to the surface. Underground mining methods can be classified by the type of ground support used (Table 1) into those with naturally-supported openings, those with artificially-supported openings, and caving techniques, although more than one type of extraction is often used in the same mine (Peters, 1978). The methods relying on natural support of openings include longhole open stoping and shrinkage stoping in strong metalliferous ores, and room-and-pillar mining of flat to gently dipping coal seams. Cut-and-fill mining of metalliferous ores, whereby dewatered mill tailings or other fine-grained materials are used to provide a working floor for overhand stoping between levels, is the most common method using artificial support: longwall mining of coal, in which cantilever hydraulic roof supports provide protection for a movable coal shearer and conveyor system, is a further example of artificial support of openings, although goaf collapse into mined-out areas is strictly a caving method. Block caving is a low-cost method of underground mining suited to large bodies of weak ore and wall rock, and in which broken ore is recovered from a series of draw points above a haulage level: as the name implies, caving methods involve eventual subsidence of the fractured overburden following ore extraction.

GEOTECHNICAL CONSIDERATIONS

Although similar geomechanics (ie soil and rock) parameters may be required for both civil engineering and mining engineering purposes, there are two very important differences between the disciplines (Peters, 1978). The first is that ore has to be mined where it is found: unlike civil engineering projects, where to some extent alignments or sites can be varied to optimise foundation conditions or minimise geotechnical hazards, in mining the only variables are the access

TABLE 2

GEOTECHNICAL CONSIDERATIONS IN MINING

	Category	Geotechnical Input
INFRASTRUCTURE REQUIREMENTS	A. Transportation	Corridor investigations; design; construction
	B. Water Supply	Hydrogeological evaluation and installation
	C. Other	As required
MINING FACTORS	A. Material Properties	Determination of intact rock & soil properties (strength; durability; etc)
	B. Mass Characteristics	Identification and evaluation of defect sets, esp. for rock mass stability and/or land subsidence studies
	C. Groundwater	Assessment of pressures, quantities and hydrogeological modelling
WASTE MANAGEMENT	A. Overburden Disposal	Dump locations; physical durability; chemical decomposition
	B. Ore Processing	Temporary storage of liquids and/or solids, some toxic
	C. Tailings Disposal	Design and construction of tailings dams; seepage monitoring systems
	D. Water Treatment	Design and construction of facilities, including monitoring
MINESITE REHABILITATION	A. Waste Reclamation	Techniques for waste restoration such as revegetation
	B. Mine Excavation	Long-term stability and/or end-use of mined-out area
	C. Environmental Monitoring	Long-term performance of rehabilitation measures (eg water quality monitoring)

method, the extraction method, and the sequencing of operations. The second is that to maximise ore extraction in mining it is often necessary to operate with only marginal factors of safety: in contrast with the "permanence" of conservatively designed civil engineering structures, stress fields are constantly changing in an operating mine and working faces are most certainly only "temporary". In mining and quarrying, therefore, optimal extraction involves a balance between costs and safety, and unscheduled mine closure due to ground failure can have serious long-term economic consequences as well as disastrous short-term effects.

Geotechnical considerations in mining are summarised in Table 2, and have been grouped under infrastructure requirements, mining factors, waste management concerns, and minesite rehabilitation. Geotechnical data input is here considered to involve the disciplines of rock and soil mechanics, engineering geology, and hydrogeology: studies range from "conventional" geomechanics site investigations to the specialist design and construction of, for example, tailings dams and associated groundwater monitoring systems. The infrastructure requirements for transportation (such as road, rail or conveyor systems) typically involve conventional investigations for the design of cut slopes and fill embankments, and data input includes engineering geological mapping, rock and/or soil sampling and testing, and geomechanics design. On the other hand provision of an adequate water supply, either by developing a local well-field or by pumping from an existing source, may require significant hydrogeological input as to groundwater quantity and quality but little geomechanics data. Other infrastructure requirements, such as power supply, may also involve only limited site-specific investigations for alignment or foundation design purposes.

Following Sullivan (1993), mining factors have been grouped under material properties, mass characteristics, and groundwater effects (Table 2). Intact material properties of importance in mining practice include strength, elasticity, plasticity, shrink/swell potential and durability: as in civil engineering the significance of these parameters differs for the engineering soils, soft (= weak) rocks or hard rocks (as defined by Bell & Pettinga, 1984) that form the ore and waste materials. As in any geomechanics study it is the mass characteristics which ultimately determine stability in either surface or subsurface mining: defect control of failures in both soft and hard rock is thoroughly documented, with parameters such as orientation, continuity, spacing and infilling of particular importance (Hoek & Bray, 1981; Brady & Brown, 1985). The effects of groundwater on both open-cut and underground mining are similarly well documented, with geotechnical problems including decreased wall, floor or roof stability, increased costs due to a need for wet blasting, and materials handling difficulties (Misich et al, 1993): mine dewatering is therefore a common requirement, with poor hydrogeologic understanding and delayed installation being two of the principal causes of water-induced problems in mining (Morgan, 1993). Geotechnical requirements for mining are briefly reviewed by Sullivan (1993), whilst Bridges (1993) details options for assessing ground conditions for mining at the exploration stage making particular reference to available software packages for core logging and rock defect analysis.

Waste management in operating mines is broadly concerned with the storage and disposal of uneconomic materials such as overburden and mill tailings, and with the impoundment and treatment of water (and other liquids) used in ore processing. Overburden from surface mining may be disposed of by backfilling worked-out areas, or by the creation of dump sites beyond the limits of the deposit: in either case some form of restoration, such as revegetation, will be required as a condition of licencing, whilst the long-term geotechnical performance of the waste material is important in terms of its strength, durability and stability. The stability of mine tailings dams is of particular importance because of past failures involving substantial loss of life (Genevois & Tecca, 1993), and because of the potential for significant environmental damage to land and water ecosystems in the event of dam failure: there is wide recognition of the need for legislation to ensure the safe design, operation and rehabilitation of tailings dams, and ICOLD (1989) has developed a set of guidelines which has been adopted in Western Australia and elsewhere (Jones et al, 1993).

Fell et al (1992) provide an overview of the nature and properties of mine and industrial tailings, and of current practices in the design, construction and monitoring of tailings dams: seepage and potential contamination from mine wastes is specifically discussed by Fell et al (1993), and they conclude that prediction and control involves a study of the geotechnical and geochemical properties of the mine tailings and waste rock, and also of the site hydrology. Storage of ore treatment chemicals and contaminated tailings is a priority in waste management at mine sites, whilst the potential for acid drainage generation from mining wastes or residues has received particular attention (Fell et al, 1992; Kwong, 1993): in gold ore treatment cyanide is widely used, and given its potential toxicity close controls are essential on both its short-term storage and longer-term disposal into a stable, non-leaching structure (Jones, 1993).

Minesite rehabilitation is concerned with long-term management of an area once mining operations have ceased. Whilst many of the concerns are more correctly termed "environmental", it is clear that geotechnical data input is still required at this stage. Waste reclamation describes techniques for the long-term stabilisation and/or remediation of waste dumps and mill tailings: although an extreme example, the problem is illustrated by the Ranger uranium mine in the Northern Territory (Australia) where more than 100 x 10^6 t of tailings, subeconomic ore and waste rock will require secure containment for a minimum period of 1,000 years from the cessation of mining in about 2012 (Willgoose & Riley, 1993). The end-use of the mine or quarry site may also involve geotechnical works to ensure stability and/or the minimisation of effluent production, especially if development of the site for building or related purposes is proposed (Singh, 1993). Finally, environmental monitoring of rehabilitated mining areas may be required to ensure that design objectives are met: as pointed out by Mattiazzo (1993), mining is only a temporary land-use and the opportunity exists to create a wide range of after-uses for such sites.

ENVIRONMENTAL MANAGEMENT

Mining can be broadly considered in four separate but sequential stages (Table 3), the first being exploration for potentially economic deposits (or prospects); the second involving mine feasibility studies from which the final capital, operating and site restoration costs can be estimated; the third requiring infrastructure, mining and waste management expenditure to permit mine operation if the project is economically viable; and the fourth involving mine site rehabilitation to meet any licencing conditions or other objectives. Both Vines & Ward (1993) and Dutton (1993) stress the importance of planning to minimise environmental impacts at each

TABLE 3

ENVIRONMENTAL ASPECTS OF EXPLORATION AND MINING

Stage	Geotechnical Objectives	Potential Environmental Impacts
EXPLORATION	Broad assessment of site geology in terms of geotechnical implications; Preliminary hydrogeological studies	Low impact operations including access tracks, costeaning, drilling and field camps
FEASIBILITY	Detailed evaluation of geotechnical and hydrogeological factors in mine feasibility; subsequent design if warranted	Additional site-specific investigations of low impact; assessment of environmental constraints to mine development, including end-use of site
OPERATION	Design/construction/monitoring of mining methods and waste management, with modifications where appropriate	Significant visual impact, with potential effects on land, air and water; close monitoring to meet or better licence standards
REHABILITATION	Remediation of waste materials and tailings dams; effluent control measures and monitoring where necessary	Return site to approved end-use, with long-term monitoring to ensure specific standards are met

stage of project development, and Warren (1989) notes that for every 1000 exploration prospects considered only about 100 are evaluated on the ground and only about 10 are drilled or explored in detail. Vines & Ward (1993) further note that before a major mining project commences in Australia it is now normal practice to prepare an "environmental management plan" which 1) incorporates baseline data (for example, on soils, landforms, flora, fauna, hydrology and heritage or conservation values); 2) clearly defines short-term objectives for environmental management whilst the mining or quarrying operations continue, including monitoring and audit requirements; and 3) identifies long-term objectives and measures for minesite rehabilitation, as well as contingency plans. Most mining and exploration companies have strict environmental policies and standards which as a minimum meet regulatory requirements, and the implementation of which extends to all employees and contractors: the approach adopted by the Broken Hill Proprietary Company Limited (BHP) is typical, with "...organisational structures...in place to ensure that environmental management is treated as an issue whose importance ranks equally with health and safety, production, quality and costs" (Purnell & Davis, 1993, p108).

Modern exploration programmes are usually multi-staged, with initial work utilising remote sensing methods (satellite or aerial photography and airborne geophysics) in combination with ground geochemical techniques (for example, stream sediment surveys): ground follow-up will typically involve one or more of soil or rock geochemistry, geophysics, surface trenching or costeaning, and drilling of selected targets. The ground phases of exploration may thus involve construction of access tracks, survey and seismic lines, drill platforms and field camp sites (Vines & Ward, 1993): the impacts of such activities can be significantly reduced by careful planning and appropriate field practices, and it is becoming common practice for areas disturbed during exploration to be rehabilitated by limited earthworks and/or revegetation (Purnell & Davis, 1993; White, 1993). Feasibility studies would normally involve infill drilling for grade confirmation, site investigations for locating treatment plants and waste disposal areas, baseline environmental management studies, and trial excavations (including possibly adits or shafts) to provide bulk samples for mineral processing. Again, site restoration can be realistically carried out if the decision is made not to proceed to mining or quarrying: the extent of ground disturbance, especially if the feasibility investigations have been carefully managed, is still much less than would result from full-scale operations.

Dutton (1993) identifies four environmental impacts associated with quarry operations, these being 1) visual impairment by the working site and associated facilities; 2) nuisances such as dust and noise; 3) physical and/or chemical pollution of water, both surface and underground; and 4) habitat loss by removal of vegetation and/or soil erosion. During the operating stage of a mine or quarry, especially one close to an existing urban area or a site of special sensitivity, environmental management procedures will be required that minimise nuisance and visual impact, and at the same time ensure that atmospheric or water pollution is reduced to acceptable levels. Clearly the measures adopted will be site-specific, and will at least meet the regulatory standards set as a condition of the mining licence or any subsequently agreed variant: again as an example, BHP has a range of operational environmental practices for the monitoring, control and minimisation of solid, liquid and atmospheric discharges that include improved production procedures to reduce waste generation as well as close liaison with affected communities (Purnell & Davis, 1993). At the Olympic Dam operations of Western Mining Corporation Limited (WMC) in South Australia, where products include copper, gold, silver and uranium, an extremely comprehensive monitoring programme is carried out: parameters recorded on a regular basis include vegetation, weeds, fauna, vermin, sand movement, salinity, hydrogeology, air emissions, mound spring ecology, and radiation levels (White, 1993). Methods of waste water treatment for the removal of heavy metals from mine and plant effluent are reviewed by Miedecke (1989), and he concludes that in most cases satisfactory discharge quality can be achieved aeration and/or sand filtration: the increasing use is also noted of wetlands for the treatment of acid mine drainage, in which the retention of metals is attributed to chemical and biological processes such as cation exchange, adsorption and uptake in growing vegetation (Miedecke, 1989).

Rehabilitation requirements for mines and quarries vary widely depending on the nature of the operations and the site itself, regulatory standards, and community expectations: Dutton (1993), for example, identifies after-uses for worked-out quarries that include outdoor recreation areas, agricultural land, pasture or timber production, housing or light industrial purposes, and water supply storage. In the mineral sands industry in Australia, where some 90% of the world's titanium and zircon sources are produced, environmental concerns have been identified for more than 40 years and integrated mining and rehabilitation practised: these procedures include landform reconstruction by tailings placement, careful management of topsoil, re-establishment of native species, and monitoring of vegetation growth (Brooks & Nicholls, 1993). In the Darling Ranges of Western Australia bauxite has been mined since 1963 by Alcoa of Australia Limited, and the objective of mine rehabilitation is to establish a stable, self-regenerating jarrah forest ecosystem: procedures for earthworks, erosion and drainage control, planting, fertilising and rehabilitation monitoring have been worked out jointly with the Western Australia Department of Conservation and Land Management (Ward et al, 1993). In mid-1992 an economic decision was taken to close the Goldsworthy iron ore mine site and associated infrastructure in the Pilbara area of northern Western Australia, and a closure protocol was established by the owner (BHP) as there was no existing legislative standard for such a process: the principal rehabilitation objective for the project was "...to create a stable, sustainable site compatible with the surrounding topography and habitats", and the procedures adopted included removal of all materials from the site, reprofiling of waste dumps, and revegetation to ensure long-term stability and regeneration of natural species (Smith, 1993).

MINING AND QUARRYING IN NEW ZEALAND

The recovery of greenstone (= jade or nephrite) by the Maoris for weapons, tools and ornamentation predates European settlement in New Zealand: it was the discovery in 1852 of gold in the Coromandel, however, followed by rushes to Collingwood (1857), Otago (1861) and the West Coast (1864), that contributed to the rapid development of the country (Williams, 1974). In the period to about 1960 some $26x10^6$ oz (= 815 t) of gold were produced from both lode and alluvial sources, and major production came from the Hauraki and Otago Goldfields: as previously noted the bucket-line (or bucket-ladder) dredge was first used for alluvial gold mining in New Zealand, whilst the cyanide process for gold recovery was also pioneered in this country (Williams, 1974). In the last 10 or so years renewed alluvial gold mining has taken place in the South Island, whilst large-scale hard-rock operations have recommenced at Martha Hill (Waihi) and Golden Cross in the Coromandel, and at Macraes in Otago: in calendar 1991 6.76t of gold were produced in New Zealand, as well as 11.37t of silver (Table 4). In addition to gold (and associated silver), iron ore mining took place sporadically from a hard-rock source at Onekaka (Nelson), but it was not until about 1960 that the technical and economic feasibility of recovering iron from titaniferous sands was finally established: major production of steel now occurs at Glenbrook from titaniferous ironsands and there is also a significant export market, with calendar 1991 production totalling 2.26Mt of ironsand concentrates (Table 4). Minor production of other metals has also occurred in New Zealand, these including antimony, arsenic, copper, chromium, mercury, tin and tungsten (Williams, 1974): currently none of these metallic ores is being mined.

As in many developed countries, the value of non-

TABLE 4

MINERAL PRODUCTION IN NEW ZEALAND FOR CALENDAR 1991[1]

	Commodity	Quantity (Mt)	Value (NZ$M)
METALS	Gold	6.76×10^{-6}	136.5
	Silver	11.37×10^{-6}	2.5
	Ironsands	2.26	24.9
	TOTAL	**2.26**	**163.9**
NON-METALS	Clay	0.14	11.8
	Limestone & Dolomite	3.13	27.9
	Sand & Rock	19.68	163.4
	Others	0.18	5.0
	TOTAL	**23.13**	**208.1**
COAL	Coal	**2.68**	**156.3**

Note: 1) Data from New Zealand Mining December 1992.

metals produced in New Zealand for both export and domestic consumption in New Zealand exceeds that of metals: in calendar 1991 the production of non-metals was valued at NZ$208M, coal at NZ$156M, and metals at NZ$164M (Table 4). Of the non-metals produced, approximately 80% by both quantity and value were sand, rock and/or gravel for a variety of purposes including armour stone, building materials, and roading aggregates: the other major products were limestone and dolomite used for industry, agriculture and cement manufacture (Table 4). Other non-metals mined or quarried in calendar 1991 included clays for bricks, tiles and pottery, perlite, pumice, serpentine, silica sand for glass and foundry purposes, and sulphur: in the past other non-metallic minerals recovered in New Zealand included asbestos, bentonite, magnesite, and mica (Williams, 1974). Significant production of coal (2.68Mt in 1991) and also of peat continues, principally for export sales, whilst both oil and natural gas are recovered and processed from Taranaki and contribute substantially to New Zealand's energy requirements.

Two Acts of Parliament effectively control the licensing of exploration and mining in New Zealand, these being the Crown Minerals Act 1991 and the Resource Management Act 1991. The Crown Minerals Act applies only to those minerals owned by the Crown (ie gold, silver, uranium and petroleum), and provides for the issuing of permits to prospect, explore or mine and the securing of land access arrangements. The term "prospecting" is used in the Act to refer to any activity undertaken to identify land likely to contain exploitable minerals, including geological, geochemical and geophysical surveys as well as hand sampling and aerial surveys. On the other hand "exploration" relates to investigations to identify mineral deposits and evaluating the feasibility of mining, including drilling and bulk sampling: the carrying out of geotechnical investigations for the purposes of design or environmental management would similarly be termed "exploration". In a general sense the use of "prospecting" and "exploration" in the Crown Minerals Act 1991 is similar to that given in Table 3 for "exploration" and "feasibility", although the distinction in the Act is somewhat arbitrary and does not reflect current practices in the minerals industry: in particular, machine drilling to obtain core samples would normally be considered at an early stage in the evaluation of some geophysical and/or geochemical anomalies. The term "mining" under the Act refers to any method used for the economic recovery of a resource, whilst the term "mining operation" means any operation in connection with mining, exploring or prospecting: the term "operation" in Table 3 is thus equivalent to the term "mining" in the Act.

In the case where minerals are not owned by the Crown, mining or quarrying proceeds by means of resource consents issued by District and/or Regional

Councils following satisfactory negotiation with the land and the mineral owners. The resource consents will cover such matters as water and land use, dust emission, noise control and discharges of solids and liquids: applications are to be submitted with supporting documentation in the form of an Environmental Assessment or Report. Because each region or district in the country can decide what are permitted and conditional uses under the Act, it is likely that consents for mining or quarrying will be more difficult in some locations such as coastal land or areas with high conservation values (Clarke & Whitlock, 1993). The need for wide consultation with affected and interested parties has increased under current legislation, although it is unlikely that well researched and documented proposals for mines and quarries will be rejected: the time frame necessary for the granting of consents has, however, quite clearly been extended, with obvious economic ramifications for mineral developers.

ENVIRONMENTAL MANAGEMENT PRACTICES FOR MINING IN NEW ZEALAND

As in most countries, mining and quarrying activities in New Zealand until about the 1950s paid little attention to land restoration or later usage of the worked-out areas. Dredge tailings were typically dumped from the stacker without regard to reinstatement for agricultural or forestry use, whilst mine dumps and mill tailings were abandoned without deliberate attempts at stabilisation or revegetation. In parts of the country with high rainfall and natural regeneration, worked-out areas often became overgrown as the mining machinery rusted and the mine buildings deteriorated. The underlying philosophy was that mining was an end-use in itself, and that economic progress was paramount: the abandonment of a mine or quarry site was seen as the end of the operation, and not as a stage in changing land use. Although the Resource Management Act 1991 embodies a new philosophy of sustainable management of resources (excluding minerals), the concept of responsible environmental management was already well established in New Zealand and in earlier legislation.

In the alluvial gold mining industry it had become common practice for land disturbed during mining to be restored to a capability or productivity equivalent to that which previously existed, and to plan for that restoration work prior to the commencement of mining. According to Parker & Tinnelly (1989), effective rehabilitation planning required a description and assessment of existing site conditions; choices regarding final land use; establishment of the final landform; water management and erosion control; soil reconstruction and revegetation; and management of the rehabilitated land. These measures for rehabilitation are well illustrated by the Grey River Dredge, which operates on low-grade gold-bearing gravels in the Grey River valley, and which was originally designed for a 750 m^3h^{-1} bucket-ladder and a separate 1500 m^3h^{-1} suction cutter system (Daniel & Breese, 1989). Although reconfigured to operate as a bucket-latter dredge, the procedures for land restoration remain unchanged: topsoil is stripped from the proposed pond area, and tailings after processing are stacked and levelled, fines are spread before re-topsoiling, and pasture or forestry established as a future land use.

The Waihi Gold Mining Company Limited operates an open-pit gold-silver mine and ore treatment plant at Waihi in the Coromandel, with up to 3.5 Mty^{-1} of waste rock and 0.85 Mty^{-1} of ore being produced. The mine site is within the town area, whilst waste is disposed of adjacent to horticultural and dairying land on its outskirts: strict environmental management procedures have been implemented since 1982, and continue as part of the mine operating strategy (Brodie & Mason, 1993). At Waihi environmental management involves 1) monitoring and minimisation of dust, noise and vibration; 2) waste rock disposal and progressive rehabilitation; 3) mill tailings disposal; 4) treatment, safe disposal and monitoring of mine waters and cyanide process waters; 5) monitoring of groundwater quality and levels; and 6) community liaison. The disposal of waste rock provides a typical example of environmental management practice at Waihi, and significant geotechnical input has been required in the design and construction of these facilities, as well as for long-term monitoring of disposal procedures and embankment performance. After crushing to minus 300 mm in the pit, the waste rock is in part used for construction of the tailings dam, and unoxidised waste (which could generate acid mine drainage) is encapsulated in the dam wall: Brodie & Mason (1993) describe other aspects of operational practice at Waihi, and restoration procedures for tailings and waste rock are detailed by Gregg & Stewart (1991) and Lapwood (1991). At the Macraes gold mine in Otago permits for dams for water impoundment and tailings retention contained special conditions for design, construction, management and monitoring (O'Leary, 1992): the tailings dam is a zoned embankment with complex drainage measures to ensure collection of seepage and appropriate disposal or reuse, and analyses indicate seismic stability under the design basis earthquake (DBE) of 0.15g. It is clear from this extremely brief review of both Waihi and Macraes operations that significant geotechnical data input is required for environmental management at major operating gold mines in New Zealand, and that the strict environmental standards adopted are an important part of the overall economics of such projects.

Quarrying operations in New Zealand are similarly required to meet appropriate standards with regard to operation, waste disposal and restoration. Given the fact that most non-metallic minerals are chemically inert, operational requirements are typically concerned with dust, noise, vibration and waste containment: water quality issues relate more to physical (ie sediment load) than to chemical composition, and long-term uses for quarry sites after cessation of operations is now an important consideration. It is necessary for significant geotechnical input at all stages of quarry management, and not least to address stability and related concerns after abandonment: for quarries located within urban areas there is often pressure to allow residential development within or immediately adjacent to quarries, and past practices have not always been adequate in this regard. In Christchurch the former Tramways Quarry at McCormacks Bay, which produced armour stone and rock fill from volcanic lavas, had been abandoned for more than 20 years when Scheme Plan approval was given about 1985 for subdivision of the site without requiring any investigation of batter stability or related geotechnical concerns (McDowell, 1989). Subsequent engineering geological investigations revealed extensive blast damage to the batter faces, spoil tips below batters and a boulder retaining wall that had been placed on old quarry fill: as a consequence individual section owners were faced with average costs close to $10,000 each for remedial measures (such as scaling and underpinning) to ensure long-term slope stability to a standard suitable for residential development.
Only limited underground mining takes place in New Zealand, this being primarily for coal in the Waikato Coalfield and for gold and silver at the Golden Cross Mine in the Coromandel. Environmental concerns with underground operations include ground subsidence over unsupported workings, generation of acid mine drainage, and mill operations and associated waste and water disposal: the question of land-use after mining is not usually a concern in such cases. Kelsey (1987) describes ground subsidence affecting approximately 7 ha of land in Huntly township following extraction of coal from the Huntly East Mine: maximum settlement exceeded 800 mm, and resultant damage of about $0.45M occurred to a hostel complex. A dewatering consolidation model was interpreted for the site, with void migration following mine roof collapse leading to drainage and depressurisation of aquifers in the unconsolidated pumiceous sediments forming the Tauranga Group overburden sequence. At the Golden Cross Mine, which involves both open-pit and underground mining operations, a comprehensive water management system is in operation involving diversion drains for runoff; silt ponds to collect turbid water; and a treatment plant for contaminated water from the operations (Crampton, 1993). Comprehensive waste management practices have also been implemented, and a range of environmental factors are monitored including water quality, waste geochemistry, structure stability, fisheries and wildlife, and rehabilitation: significant geotechnical input is clearly required for this project, which again reflects the high environmental standards that are achievable when mining in sensitive areas.

CONCLUSIONS

1. Mining techniques can be broadly grouped into placer, open-cut and underground methods depending on the geological nature and location of the orebody: quarrying is regarded as a form of open-cut mining that is applied to the recovery of non-metallic minerals and rocks.

2. The principal geotechnical considerations in mining and quarrying practice are infrastructure requirements, specific mining factors, waste and water management, and minesite rehabilitation: the mining factors include material properties, mass characteristics, and the influences of groundwater in either surface or subsurface operations.

3. Mining and quarrying involve sequential stages which can be termed exploration, feasibility, operation, and rehabilitation: each has associated environmental impacts and constraints, and integrated planning is an essential requirement to meet present-day standards.

4. New Zealand produces significant quantities of gold, silver and ironsands, and was formerly a major producer of gold: non-metallic minerals products include sand and rock, limestone, and clay, as well as coal and peat.

5. Current mining and quarrying practices have to meet stringent standards of geotechnical and environmental performance in terms of the Crown Minerals Act 1991 and/or the Resource Management Act 1991: these practices include operating parameters, waste and water management, and site rehabilitation to an acceptable standard.

6. Given the demonstrated capability of the mining industry to meet strict environmental standards and to perform to acceptable economic levels, further large-scale developments in gold recovery are likely in New Zealand: continued quarrying operations to support industrial growth are also envisaged, and a balance between extraction and conservation of resources must be maintained.

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PIT SLOPE STABILITY

J St George
The University of Auckland
Auckland

SYNOPSIS

The stability of open pit mine slopes will be reviewed in respect to final slope position, end use of pit and long term stability. The factors influencing stability will be addressed including the effect of old workings. Problems associated with lake filling and the change in stability during filling will be investigated.

INTRODUCTION

The stability of slopes is an important factor in the design and operation of any open pit mine. The batter angles and height of both local and final slopes has a major effect on the overburden removal and therefore directly influences the economics of the project. Disruption from slope failures can cause delays in production and incur extra costs for clean up and stabilisation. Slope design criteria will also involve other factors such as bench heights, berm widths, machinery selection, overburden/topsoil dumping arrangements, placement of haul roads and siting of facilities.

From its early conception through to the final production shifts and onto rehabilitation, there is a need to assess the stability of all slopes within the boundary of a mining site. This assessment will depend on the stage of the project and the service requirements of the slope. During the life of a mine the actual excavated slopes and design requirements change quite radically. In the preliminary phases of the mine development approximate slope designs are made using the data obtained from site investigation and limited testing. Once production has started these designs are refined as more information becomes available. Because of the similar geological environment at a mine site, there may be direct transfer of past experience into current slope design. Stable slopes demonstrate successful design strategy under the external influences that the slope has been subjected to up to the present, future conditions could initiate failure. Unfortunately it is not possible to assess how well a stable slope is performing or how it reacts to changes in external conditions until they occur. Final slopes will always be sited with due consideration to stability within the constraints of maximising the mineral recovery in the lease boundary. On completion of the production phases the long-term stability of the final slopes must be designed to suit the intended use of the pit, most frequently the pit is either filled with water serving as a lake or left open. In some cases the latter option is only temporary as the old pit becomes a dump for overburden waste rock from an adjacent operation.

The main objective in mine slope design is to produce a workable slope geometry which is able to accommodate changing conditions and one suited to the production schedule. The performance of slopes can be monitored during production and, if necessary, alterations made to the design.

There are fundamental differences between slopes designed for permanent civil engineering ventures and mine operations. In the case of the former location and orientation are controlled by external influences while failure is generally considered unacceptable and has severe consequences. For the latter there is much greater flexibility; the orientation and sequence of extraction of working slopes can be adjusted to take into account slope stability. Some instability of the pit slopes, although undesirable, can be tolerated. This is because access for personnel to areas at risk can be restricted, equipment and facilities can be relocated, the exposure of the working slopes to failure is usually for a limited time and the consequences of failure are much easier to control on the mine site.

These factors are ideally suited to a reliability approach where the conventional factor of safety is augmented with a probability of failure which is based on the uncertainties inherent in the design. One advantage of using a probability measure is that alternative design strategies can be compared on a risk - benefit basis and the optimum one selected. It could therefore be cost effective to accept a slope failure with the associated site disruption and clean up costs offset against a more stable slope and greater stripping quantities.

A further instability problem particular to mine slopes is related to the presence of old workings. Many surface mines are excavated in localities with a history of underground mining and therefore abandoned workings are frequently encountered and when present within or beneath slopes can promote instability.

In this paper the types of failure occurring in open mines will be reviewed and this will be followed by an introduction to reliability methods applied to mine slope analysis. The particular problems related to the effect of old workings on slope stability will be discussed as well as the long term stability of final pit slopes in relation to their end use.

FAILURE MECHANISMS

Although all slope failures are three dimensional in form it is general to consider a two dimensional cross section through the slope as being representative of the failure mode for the purpose of analysis. For the rock materials generally encountered there are four broad categories of failure which have been described in texts on slope stability e.g. Hoek and Bray (1981) and are as follows:

- Planar — including biplanar, multiplanar and slab with sliding taking place on one or more essentially planar surfaces.
- Wedge — sliding on two intersecting surfaces 3-D effects included.
- Circular — sliding on quasi-circular surfaces in highly fractured or low strength material.
- Toppling — blocks of rock toppling or falling from the face.

Flow slides caused by liquefaction are not considered because they are only really applicable to very weak natural materials or dumped material.

Most actual failures are reasonably complex with a combination of failure modes and significant 3-D effects, which are often ignored in the design stages. This is because most 3-D effects produce constraining forces to any potential failure therefore their omission will be conservative.

There is little published work on documented case studies of slope failures in open pit mines which give insight into the mechanisms of failure, their controlling factors and their overall impact on operations. It is usually the large scale failures or those which have atypical conditions or major consequences that have been reported.

A survey of 240 slope failures was undertaken as part of an ongoing research programme set up between the Department of Mining Engineering, Nottingham University and the Opencast Executive of the then National Coal Board. For each case data was recorded on the rock types, failure mechanisms, slope geometry, water conditions and any contributing factors. The results of this survey have been discussed in detail by Stead (1984). A total of 171 case studies of rock slope instabilities were classified under the ten distinct failure mechanisms identified by Walton and Atkinson (1978) for rock slopes in surface coal mines. Figure 1 shows the proportion of the various mechanisms recorded as a percentage of the total. Only nine mechanisms are listed since no buckling failures were observed in the study. The collapse mode refers to failures associated with the collapse of old workings.

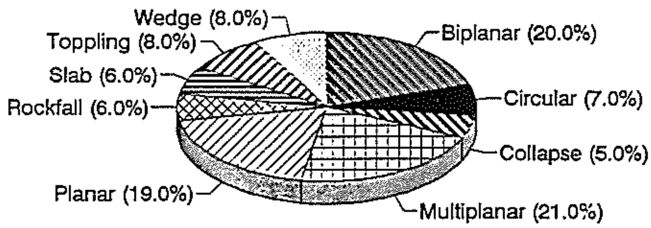

Figure 1 Rock Slope Failure Mechanisms

In order to have an indication of the seriousness of the event in terms of disruption or cost, the volume of failure was used as guide. It is recognised that this is not always correct since quite small failures which threaten important facilities can be very disruptive. Table 1 lists the median volumes of the rock slope failures for each failure mode together with the percentage greater than $50000m^3$. This figure is an arbitrary level above which failures were taken to be substantial. The median was considered to be more representative than the mean volume since the sample size in each group was small and the distribution of volumes highly skewed.

As can be seen from Table 1 there is a predominance of the planar type failures, 66% of all recorded rock failures are in this category with a fair percentage of those being major ($> 50000m^3$). All planar type failures are controlled by discontinuities in the rock mass such as bedding, joints, faults or intraformational shear zones. This emphasises how important it is to determine the structural geology of the pit walls in advance of mining.

Table 1 Rock Slope Failure Mechanisms

Mechanism	%	Median Volumes	% > 50000m^3
Planar	19	10.0	24
Biplanar	20	25.0	35
Multiplanar	21	14.4	18
Slab	6	65.8	57
Circular	7	12.6	9
Wedge	8	2.0	0
Toppling	8	1.5	0
Rockfall	6	1.0	0
Collapse of old workings	5	0.7	11

All volumes in 1000m^3

RELIABILITY APPROACH

In using a probabilistic methodology for slope stability it is important to note that some uncertainties in the design procedure will be subjective, that is they will be associated with a lack information or knowledge. The resolution of these necessitates the use of deductive reasoning and as such they cannot be included in the analysis.

For rock slope problems there is a natural dichotomy in the treatment of uncertainties. There are those uncertainties which are related to the structure of the failure at the level of the mechanism and those which arise from the variability in parameter estimation and the conditions affecting the stability.

The uncertainties due to geological structure are usually highly subjective since they originate from incomplete or a total absence of information. Exact locations, dips, dip directions are rarely known accurately and extrapolation and deductive reasoning are frequently used. Statistical descriptions of orientation data from surveys give some measure of the scatter of data points however they are not universally applicable and should only be used judiciously on well behaved poles. In practice it is very difficult to determine the areal extent of discontinuities from essentially two dimensional planes (exposures, excavations). Persistence may be measured in the field, yet this does not give a true three dimensional picture of the discontinuity surface and characterisation is most often left to subjective assessment. Also measurement errors are introduced from sampling and these are likely to include bias as well as randomness.

The uncertainties from parameter estimation are dominated by the variability in the shear strength and the groundwater conditions. It is well recognised that the shear strength of geological materials are not single-valued parameters but display natural heterogeneity as well as some spatial dependence which implies correlation between neighbouring points. This correlation will diminish as the distance between points becomes larger until the covariance is zero. The sampling and testing procedures used in defining strength parameters will introduce errors from a number of sources including sample disturbance, testing environment and scale effects.

Unfortunately most uncertainties concerned with determining the watertable or groundwater conditions within a slope are from a lack of information. The prediction of the water pressures within a slope from very limited data usually involves gross, order or magnitude assumptions about infiltration rates, position of aquifers, aquicludes, flow, permeabilities, and presence of perched aquifers or artesian conditions. To quantify these in a meaningful way is an extremely difficult task.

To incorporate all these uncertainties into a formal analysis is unrealistic; only those uncertainties which are objective and those which can be quantified, should be included. Initially the uncertainties associated with the structural controls on failure will produce various potential slope failure geometries which need to be checked to see if they are kinematically admissible. The sum of all the probabilities of occurrence of the kinematically admissible geometries (the kinematic probabilities) multiplied by the probability of sliding for each geometry will give the overall probability of failure for the slope.

In order to calculate the probability of failure for one geometry either the full distribution of failure conditions or the moments of that distribution need to be determined. Therefore all the uncertainties associated with the random input variables need to be combined in an appropriate probability model, taking account of modelling errors, to produce the distribution of failure conditions, or its moments.

The procedures for analyzing slope failures are the same as in ordinary deterministic analyses: limiting equilibrium, stress-deformation or plasticity models. Because of the generally sparse information available, the simplicity and wide acceptance in deterministic analyses of limiting equilibrium techniques, these are favoured for probabilistic analyses. Using the factor of safety F defined as the ratio of resisting forces (R) to mobilising forces (S), the probability of failure P_f is

$$P_f = P(F < 1.0) = P(R < S)$$

If the probability density functions (pdf) of R and S are f(r) and g(s) respectively, then the probability of failure with both R and S independent is

$$P_f = \int_{-\infty}^{\infty}[1 - G(x)]\, f(x)\, dx \qquad (1)$$

where G(.) is the cumulative distribution function (cdf) of mobilising forces. The intersection of the two probability density functions indicates the region of possible failure. Defining the safety margin S_m as R-S, it is clear that S_m will also be a random variable, with failure defined as $S_m < 0$. A measure of the degree of safety is given by the safety or reliability index which is the number of standard deviations the mean safety margin is from the failure condition $S_m = 0$.

If the pdfs of the input variables are known or assumed then the P_f can be determined directly by application of equation (1). The way in which the relevant input variables are combined to produce the distributions of f(r) and g(s) will depend on the analysis and which variables are considered random. Few analytical solutions are available as even the most simplistic equilibrium models with common distributions are intractable. Wu and Kraft (1970) formulated the problem using both F and S_m; the uncertainties of resisting and mobilizing moments were combined in terms of their distribution moments. A normal distribution was assumed and the P_f determined by integration.

An alternative approach is simulation by the Monte Carlo technique which has developed by a number of authors to calculate the probability of failure P_f. All the random input variables are sampled from their respective distribution functions, joint or univariate. Each set of simulated sample variables is used in the appropriate model to produce either a factor of safety or safety margin. This process is repeated a large number of times to yield the distribution of F or S_m. The Monte Carlo method has the advantage of being a very general technique and relatively easy to apply. The only drawback is the full distribution of all random parameters is required which generally is not available.

In the literature second moment analysis has been favoured because no assumptions are required regarding the distributions of the input variables. Two techniques are preferred.

First-order second moment approximation: where the factor of safety or safety margin is expressed as a function of the random variable input parameters and the first and second moments are determined from the partial derivatives of the function about the mean.

Point estimates: the first few moments of a random function can be approximated from the moments of a function of random variables. Expressions are derived for the moments as linear combinations of the powers of the point estimates of the function. Since normally only the first two or three moments of the random function are of interest, point estimates are calculated using the first two or three moments of the random variables. A restriction that the function be well behaved is usually adequate for F and S_m formulations.

With both these methods only the moments of the F or S_m results and an assumption as to their distribution is required before a P_f can be calculated.

The probabilistic approach has been advanced to three dimensions (Vanmarke 1977) for circular slip surfaces by considering the variance reduction along the lateral extent of the slope and the end restraining effects on the failure. This work has been extended by St George(1991) to include spatial variability and arbitrary failure surfaces.

Even with these advance probability analyses the calculated P_f has not yet reached a level where it represents a realistic measure of the actual conditions. It therefore remains as a comparative quantity which can be used for selecting between alternative slope design strategies. An example of this approach is reported by Bertuzzi (1992) for the slopes at the Macraes Gold Mine, Central Otago, NZ. In this study the kinematic probability of failure was calculated for different slope angles and orientations. These were then used directly to evaluate the size of berms required to catch potential failure volumes. Various options were compared on an economic basis by taking the initial cost of excavating the slope and adding to this the weighted cost of clean up of any potential failure. The clean up cost was calculated from the expected failure volume and were weighted by P_f. This allows for rational decision making which includes the uncertainties in the solution technique.

EFFECT OF OLD WORKINGS

Most current surface mining operations encounter old abandoned underground mine workings. This is because much exploration and development has been concentrated on historical mining sites. In some cases there are no plans of the old workings and they are located either by site investigation or uncovered during mining. A good example of this is the bell pits in the North of England which were worked several centuries before records were kept. Even with more recent workings and accurate plans, there is conjecture as to whether the plans have included the full extent of the mining to the completion of the operation.

The impact on stability will depend on the type of mining; essentially either coal or metal mining. In New Zealand all the recorded coal workings were mined by the room and pillar method up until present day workings. For the metal mines only the gold prospects are of interest and these were mostly mined by narrow vein stoping methods. Stability problems from abandoned old workings generally arise from collapse mechanisms in or beneath the slope.

In the gold mining areas the underground extraction in narrow veins is generally concentrated and therefore their impact on the stability is limited to localised slumping of old roadways and stopes. Alignment of working slopes perpendicular to the drives is usually practical since the old workings are located in the ore zone.

Old coal workings represent a much larger problem since the extraction is extensive and the slopes are formed on the same horizon as the old workings. The collapse of bedded roof strata in coal measures will occur sometime after the cessation of mining. The material filling in the void eventually chokes up the opening. The height of void migration can be between 2-10 times the worked thickness, although often limited to less than twice the room width. The arch formed is typically chevron shaped from the fractured bedded strata moving into the fill zone.

Walton and Taylor (1977) in an article on the stability effects of old coal workings recognised that their presence in or beneath slopes could promote or assist translational, toppling, span and slab sliding failures. Translational sliding can occur through strata directly above the fill by undercutting or by sliding along the downwarped strata over the void or fill. Passive wedge or biplanar failures have been observed along fractures in the rock created by mining and subsequent strata collapse, and extending through the fill material. The presence of heavily fractured rock above the void can promote toppling or rockfall once the slope is excavated to expose the collapsed zone. In relatively strong strata where there has been little collapse, the roof will span the opening between the pillars. Excavation of the slope creates stresses which can induce failures at the edge of the pillars which causes the slope to move downwards. In steeply dipping strata old workings can initiate slab sliding by removing toe support or even initiate buckling failures.

The presence of old workings in and beneath a slope can in some cases dewater the strata more effectively than in virgin ground. This will tend to increase the stability of the slope. The increased flow through old workings can create problems at the end of production if the pit is to be used for waste storage. Also there is always the possibility of breaching a natural dam while mining and causing an outrush of water which could trigger collapse of the workings and/or the slope.

STABILITY OF FINAL PIT SLOPES

At the end of the mining operations the slopes must be designed to perform adequately for the intended use and life of the pit. With the current legislation under the RMA these conditions will be set up before any mining licence is granted. The three options are:

- Left exposed; this requires the pit to be self draining or natural groundwater level below the lowest excavated bench

- Lake formed; the pit is allowed to fill with water

- Filled; this can be waste rock from an adjacent mine or waste from other sources e.g. industrial or domestic etc.

If the first option of leaving the face open is taken then the stability of the final pit walls must take into account the long term effects, particularly degradation, and the likely access to the area of personnel. Stability considerations would be similar to normal slopes excavated for civil projects, although rehabilitation would probably have commenced during mining. If the pit is being used as a dump for waste rock then the stability only has to match the operating conditions during the dumping. This will be mainly safety and operational aspects. The dumping of toxic waste requires the isolation of these materials from the natural groundwater and therefore any slope failures could disrupt the integrity of the groundwater shield. This will be exacerbated with the presence of old mine workings in the slope.

The lake filling option introduces an interesting factor into the stability. During operation the mine would be pumping water from the pit depressing the groundwater table in the area. After withdrawal of the pumps the groundwater re-establishes itself and the pit begins to fill with water until the lake achieves the same height as the natural water level. The presence of an increasing water height as the slope becomes submerged, will affect the stability. Cojean and Fleurisson (1992) investigated the effect on stability of failure surface geometry, slope height, strength values (c and ϕ) and water table position. For a single planar failure surface as the lake level rose there was a corresponding increase in F. This was quite small of the order of 2% for the slopes in the study. The circular and multiplanar convex failure surfaces displayed a reduction in F as the water level increased to a minimum value of F. Then F increased to full submergence and the final F was greater than the initial fully drained slope with no submergence. The position of the minimum F depended on the shape of the failure surface and the strength parameters, and was around a third of the slope height. This reduction in stability was quite marked in some cases varying from 0 - 20% of the original F. These studies reinforce the need for careful consideration of the lake filling process so that slopes can be adequately designed to cope with the critical conditions, which may occur years after completion of mining. Certain final pit slopes might require partial backfilling to buttress the toe of a potential failure before filling.

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MINE AND QUARRY DUMP STABILITY

John P Ashby
Ashby Consulting Services
Auckland

SYNOPSIS

Surface mines and quarries often generate large volumes of rock and soil waste. Those wastes must be disposed of at high rates and low cost. Most dumps are constructed by end tipping from trucks or stackers. Because of cost considerations, the dumps must be located as close as possible to the working faces and the dumps must be compact so that haul distances can be kept as short as possible. Compactness is also a requirement where land is at a premium. Compactness is usually achieved by constructing high dumps. High rate of construction, low strength wastes and high faces lead to instability.

The paper refers to non-water impounding structures (i.e. waste dumps as distinct from tailings dams and lagoons). Mechanisms and types of failure are discussed. A design procedure is presented and methods of dump construction to improve stability, are described.

Although this paper is directed at disposal of solid waste from mines and quarries, the concepts and techniques are equally applicable to large volume clean fill operations.

INTRODUCTION

The application of modern soil mechanics to waste dump stability grew in response to perhaps the most notorious tip failure at Aberfan in south Wales in 1966. Despite subsequent efforts of investigators, much is still not understood about the precise mechanics of dump failures. Fortunately, sufficient is understood to guide mine and quarry operators and to avoid major failures.

TYPES OF DUMP FAILURES

There are a large number of modes of dump failure, six of which are illustrated below.

Failure is sometimes limited to the crest of a dump (Figure 1). Slight oversteepening due to cohesion is observed near the crest of most dumps, even those constructed of granular, "cohesionless" waste. However, when the crest of the dump is over-steepened beyond the normal face profile, failure often follows. This oversteepening is sometimes ascribed to "temporary cohesion" and is most frequently observed when tipping wet cohesive material or rock with fines at too fast a rate.

At Bougainville Copper mine, where many of the concepts described in this paper were developed, it was generally recognised that dumps constructed of slightly weathered and altered andesite at a height in excess of 50 metres invariably exhibited over-steepening and started to fail when the crest advanced at rates greater than 1 metre per day. Critical rates vary from site to site but 1 metre per day appears to be fairly typical. Observation of the slope geometry (with the aid of a clinometer), coupled with control of tipping rate and location are necessary to overcome this mode of failure. Increasing the length and number of active dump faces is obviously important if dumping rates are to be controlled effectively.

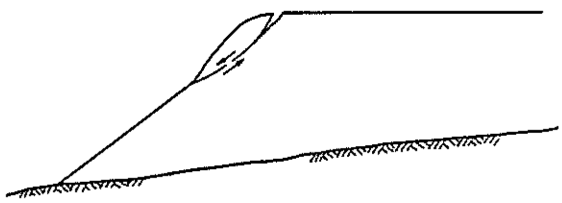


Figure 1: Failure of Dump Crest

Failure by oversteepening or undercutting of a dump face by toe erosion (Figure 2) is an obvious but not uncommon mode of failure, usually associated with side valley filling, where insufficient attention is paid to controlling dump development or to training of streams.

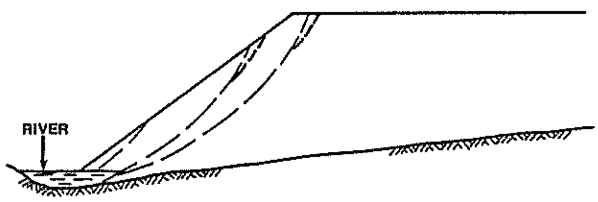


Figure 2: Failure by Undercutting

Failure within a dump may result from pore pressure build-up or loss of strength, which may be caused by heavy rains, earthquake activity, weathering or leaching (Figure 3). Long-term loss of strength is a poorly understood phenomenon which may occur at surprisingly high rates, especially for sulphide rich wastes in tropical climates. Careful design of the final configuration of large scale dumps with adequate allowance for long-term strength loss is the best remedy.

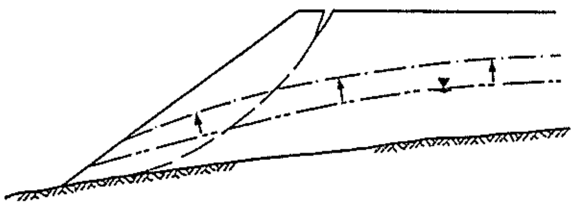


Figure 3: Failure Within Dump

Failure by dislocation along weak layers of weathered or altered rock, overburden or even snow sometimes occur and may be aggravated by pore pressure build-up behind the layers (Figure 4). Careful control of dump materials and construction are necessary to overcome the problem.

Segregation of wastes, involving separation of weaker materials and disposal elsewhere, is often recommended as a means of preventing these failures. Alternative dump sites for weaker waste include layer or retention dumps (Figures 14 & 16).

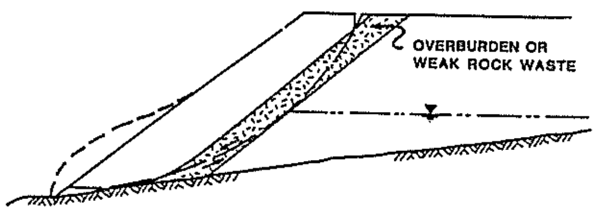


Figure 4: Failure Along Weak Layers

Failure of the dump foundation occurs as the dump loading exceeds the strength of the foundation (Figure 5). Failure sometimes occurs as the rate of the dump advance exceeds the rate at which the foundation can drain. However, failure may also occur as the critical dump height is reached at which the ultimate drained strength of the foundation is exceeded.

Where weak foundations are known to be present, preparation, involving sub-surface drainage, may be sufficient for the foundation to consolidate (i.e. the pore pressures to dissipate) as the dump advances, and for the dump to remain stable.

Where, despite drainage measures, failure is still likely, removal of weak foundations or modification of the dump design involving flattening of the dump face (as discussed below) should be considered.

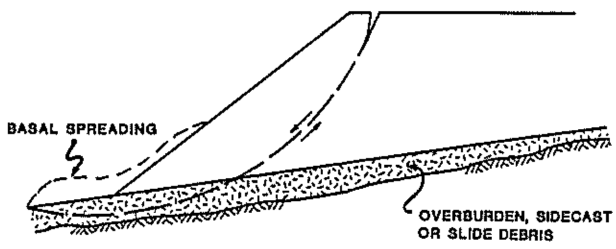


Figure 5: Failure of Dump Foundation

Failure within the dump as it reaches a critical height is most often noted when tipping into valleys (Figure 6). This is a common and most troublesome type of failure which is most pronounced when slightly "cohesive" wastes are end tipped in areas of steep terrain.

Careful design and control of dump construction involving flattening of the dump face (as discussed below, refer also to Figures 13 & 14) is the best approach for overcoming this problem.

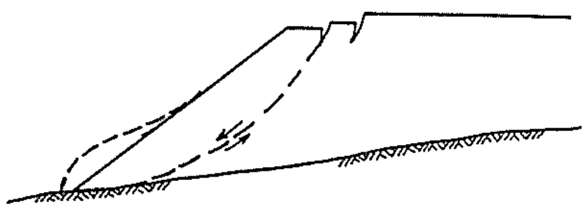


Figure 6: Failure as Dump Reaches Critical Height.

INDICATIONS OF DUMP FAILURE

Early indications of dump failure can be quite subtle but as failures progress, telltale features appear (Figure 7).

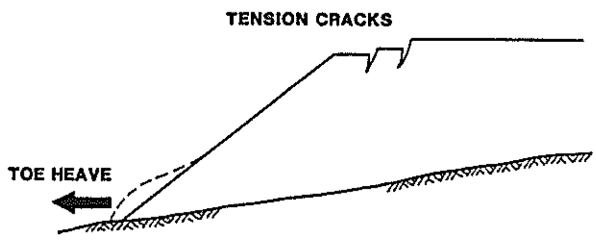
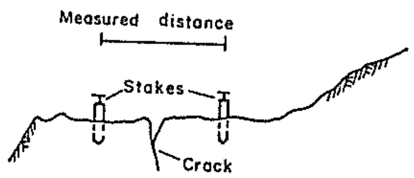


Figure 7: Indications of Dump Failure

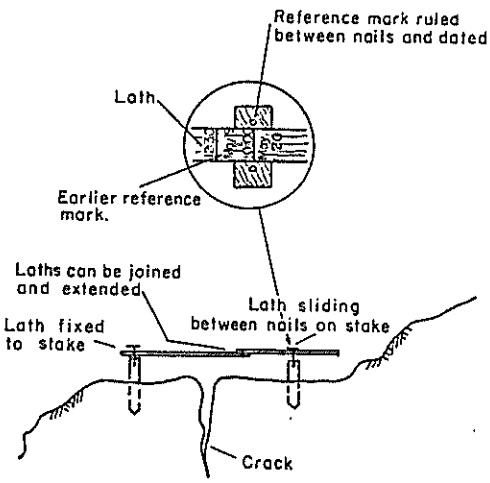
The appearance of tension cracking near the crest of a waste dump alone, is not necessarily symptomatic of failure. More often than not, the presence of tension cracks may be a result of settlement as the waste material compacts or the foundation consolidates. The visual appearance and location of settlement cracks are surprisingly similar to those of cracks caused by failure. The magnitude of settlement varies considerably from site to site but measured settlement often reaches 1-2% of dump height. Displacement prior to failure often reaches 3-6% of dump height before complete failure occurs.

Toe heave, or bulging at the dump toe, or extrusion of the foundation are more reliable early indicators of failure than cracking alone (Figure 7). Distinction can usually be made between settlement and failure (Figure 9) by employing simple methods of displacement monitoring (Figure 8).

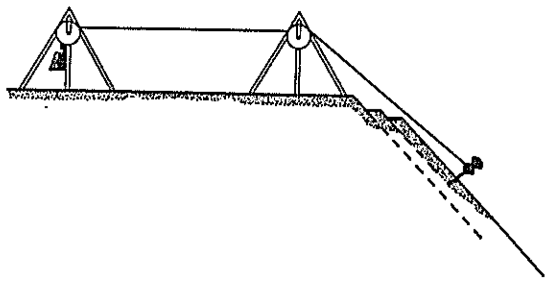
Use of measurement pins (Figure 8a) or laths (Figure 8b) works well with smaller dumps or where the cracks are well defined. For larger dumps or less distinct cracks, wire gauges are recommended (Figure 8c). Sometimes, with very large dumps or situations where a stable anchor cannot be found for a wire, survey monitoring using Electronic Distance Measurement (EDM) may be necessary.



8a: Pin Gauge



8b: Lath Gauge



8c: Wire Gauge

Figure 8: Simple Methods of Slope Monitoring

Increasing displacement with time, associated with failure can easily be distinguished from the monitoring records (Figure 9).

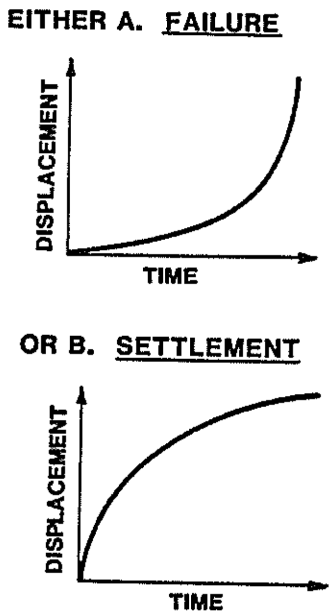


Figure 9: Displacement - Settlement or Failure?

EFFECTS OF FAILURE - RUNOUT

Most waste dumps fail quite gradually with the toe bulging out and the crest stepping down some distance before the dump comes to rest. Some dumps even appear to fail, displace a small distance and then "lockup".

However, if the waste is fine grained and becomes saturated, the failing material may travel considerable distances with disastrous consequences, as was the case at Aberfan. Careful dump design and drainage are crucial to the prevention of mud-slides.

Catastrophic runout of waste dump failures is not restricted to saturated fine grained wastes. Large scale rock dumps sometimes fail dynamically and the debris may run out considerable distances. Empirical relationships between the tangent of the runout angle (the angle measured from the crest to the ultimate toe of the failure lobe) and the height of the dumps or the volume of failed material have been published (Campbell 1993 based on work by Scheidegger 1973). Runout angles ranging from 25° for a dump height of 100 metres to 11° for dump heights of 300 metres have been recorded.

A further effect of failure is that of strain softening. Once failure of the foundation or dump itself is initiated, the shear strength reduces from peak to residual, and the peak strength and stability of the dump are never regained. Careful design to prevent failure of dumps is therefore of benefit in improving dump capacity and economics.

FAILURE SCENARIO

From the Author's experience, the most common failure scenario occurs when the waste is tipped from the top of the dump into a valley. Initially the dump is of modest height but as it progresses down the valley, so the height increases but the angle of repose remains relatively constant. As the shear strength of the waste and/or foundation materials are exceeded, failure occurs (Figures 5 & 6 above).

From this scenario, a design procedure emerges, as follows.

ANGLE OF REPOSE

Angle of repose is the angle, measured from the horizontal, at which a material comes to rest when tipped over a free face. Perhaps the most striking but least understood characteristic of waste dumps is the consistency of angle of repose. Surprisingly, most waste materials, irrespective of their composition share an angle of repose within a degree or two of 37° (or 1.3:1, horizontal : vertical).

It is frequently stated that the angle of internal friction is equal to the angle of repose of the waste. However, not only do frictional granular rock wastes and sands exhibit an angle of repose of about 37° but even many cohesive or clay wastes with low frictional strength and high natural moisture content display a similar angle of repose, at least for a short period after tipping. Thus, rather than a function of static shear strength, it is more likely that the angle of repose is a function of particle shape and the dynamic interaction of particles during tipping until the particles come to rest.

Whereas, many dumps can remain stable at the angle of repose, even when they reach considerable heights, some dumps are stable when tipped, but fail some time thereafter. It is the situations where the shear strength mobilised (once the dump is formed) is insufficient to support the dump formed at the angle of repose that serious failures usually result (i.e. the Failure Scenario described above).

DUMP HEIGHT VERSUS ANGLE PLOT

In the Author's opinion, the most valuable tool for gaining an understanding of slopes and for design of slopes including dumps is a graph of slope height versus slope angle (Figure 10).

Data gathering involves measurement of the *overall* heights and corresponding slope angles of a number of segments of dump faces. A segment must extend from the crest to the toe or from the crest to some point down the slope. The segments are judged to be either stable or failing. Particular attention should be paid to the measurement of failure profiles. The measurements are plotted on the height versus angle graph of Figure 10. Different populations of points for dumps of different materials, or foundation conditions should be plotted with different symbols or on different graphs.

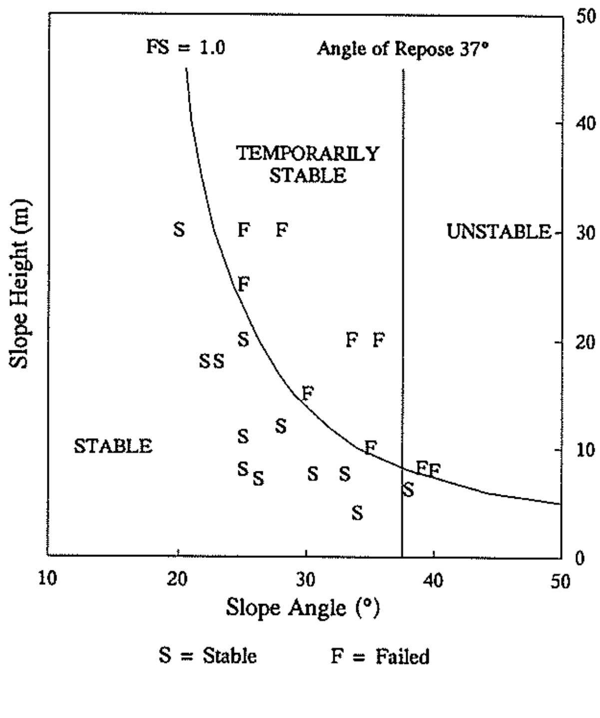


Figure 10: Slope Height Versus Slope Angle Plot

The failing slopes tend to cluster above a curve which can be used for design purposes, as discussed in the following sections of this paper.

DESIGN STRENGTH

Laboratory shear strength determination of mine wastes is not only difficult, but expensive and rarely representative of the real world. The reasons for this are as follows:

- Waste rock often includes very large particles, hence the need to resort to scaling, as for materials testing for large rock fill dams.
- Waste dumps are often very non-homogeneous, with layers of fines, interspersed with large particles.
- Foundations often vary in thickness, shear strength and water pressure conditions.
- Waste dumps may sustain considerable displacement before failure occurs. This necessitates testing with very large displacements to produce reliable, representative residual strength parameters.

From the author's experience shear testing of waste materials, even when carried out with considerable care and cost, tends to provide over-estimates of dump shear strength. Derivation of design strength by back analysis (back calculation from existing failures) is preferred.

BACK ANALYSIS

Provided failures exist in similar materials or situations to the dump, back analysis is the Author's preferred method of obtaining design strength parameters.

Although less precise and a little more time consuming than the widely available computer analyses, the method of back analysis still preferred by the Author is the use of graphical stability charts. The graphical method is interactive and aids the intuitive and slightly subjective process that is a necessary part of the successful derivation of strength parameters for slope design by back analysis.

The circular stability charts from *Rock Slope Engineering* by Hoek and Bray 1977, pp 226-238, can be used readily for back analysis. The method involves plotting the overall slope height versus slope angle graph and sketching the curve that separates failing from stable slopes (Figure 10). The next step is to estimate the water pressure conditions that existed at the time(s) of failure and to select the appropriate stability chart. (Hoek and Bray prepared five charts ranging from fully depressurised to fully saturated). The mean unit weight for the failing material is estimated.

The key part of the back analysis process is finding the *unique* combination of the effective strength parameters (the Friction Angle and the Cohesion) which describes the sketched curve separating the sketched curve of Figure 10. It is assumed that slopes falling on the curve have a Factor of Safety of 1.0. The parameters used and derived in matching the curve in Figure 10 are shown in Table 1.

Table 1: Back Analysis Parameters

Parameter	Symbol	Value
Factor of Safety	FS	1.00
Unit Weight	γ	1.7 t/m^3
Water Condition	u	Chart 3
Friction Angle	ϕ'	31°
Cohesion	c'	5 kPa

DESIGN CURVE

Having estimated the strength parameters in Table 1, it is a straightforward matter to develop the design curve shown in Figure 11. The design water conditions are assumed and the appropriate chart is selected. A design Factor of Safety is chosen to provide sufficient margin of safety. In the following example a Factor of Safety of 1.1 has been adopted. Smaller design Factors of Safety can usually be employed when using strength parameters derived by back analysis than those produced by laboratory testing. This is because in comparison with lab testing, there is less uncertainty associated with back analysis results because the ideal "test bed" of a real dump failure has been used.

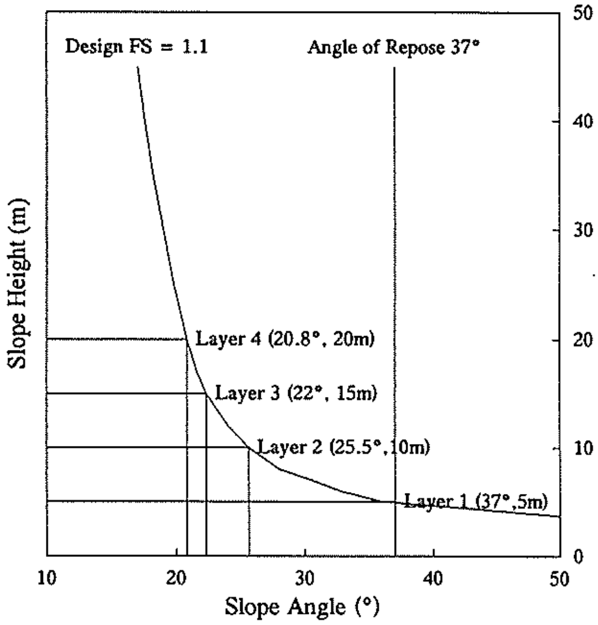


Figure 11: Dump Design Curve (FS = 1.1)

The dump design can then be expressed in tabular form as shown in the following example for a layered dump (Table 2). The layer height (5 metres) is chosen as the maximum height that the material can be end tipped at the angle of repose (37°). (This is where the curve intersects the angle of repose line in Figure 11).

The design overall dump angles for varying dump heights that are usually multiples of the individual layer height are determined from the curve as shown in Figure 11.

Table 2: Design for Layered Dump (FS=1.1)

Layer	Dump Height (Overall crest to toe) (metres)	Design Angle (degrees)
1*	5	37.0
2	10	25.5
3	15	22.0
4	20	20.8
5	-etc.-	

DUMP CONSTRUCTION

This section of the paper serves not only to classify methods of dump construction but more importantly to show how dumps can be constructed to avoid instability.

The most common method of dump construction is by end tipping from trucks or stackers (Figure 12). End tipping is usually the cheapest method of waste disposal but offers little control of the dump configuration in plan and almost none in profile. Operational control is limited to direction and rate of dump advance and sometimes the segregation of waste materials.

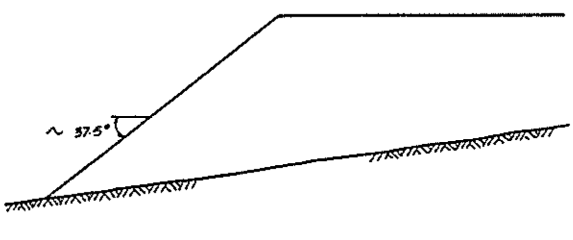


Figure 12: End Dump

Drainage of the foundation, removal of particularly troublesome vegetation and organic soil layers or even complete removal of particularly weak foundation layers may be justified in order to maintain the stability of end dumps, thus avoiding more expensive alternatives discussed below.

Where the stability of end dumps is marginal, despite use of operation controls and foundation preparation as discussed in the previous section, the overall slope angle of the dump may be moderated below the angle of repose by sequenced or buttressed end tipping (Figure 13). This involves haulage of waste around the dump face, either by establishing access on natural terrain or by cutting and sidecasting from the crest of the dump using bulldozer or excavator. This results in a terraced configuration which is usually considerably more stable than the corresponding end dump.

The geometry of the sequenced end dump should be designed to incorporate a stable overall design angle (discussed above) with access constraints determining terrace intervals and widths.

In addition to improving overall dump stability, sequenced end tipping simplifies the task of revegetation of the dump faces. Indeed, rehabilitation considerations may dictate that sequenced end tipping is carried out. By flattening the face angle of the terraces from the angle of repose (say 37°) to an angle, crudely referred to as the "vegetative angle of repose" of say 25°, regrowth can be encouraged. Although the same result can be achieved by reshaping of an end dump by sidecasting using a bulldozer, sequenced end tipping may be more economical where large dumps are involved. This is because the material is hauled directly, rather than rehandled.

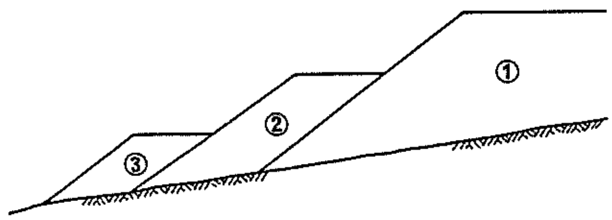


Figure 13: Sequenced End Dump

Layer dumps (Figure 14) are frequently constructed on flat terrain. When hauling from working faces below the dump base, layer tipping is used on flat terrain in preference to a single face end dump for reasons of haulage cost. In these situations an optimisation study is recommended to determine the height of each layer and the distance hauled on the layer, before ramping to a higher layer.

Layer tipping is also used to dispose of lower strength "cohesive" wastes that can only sustain a tip face of modest height. The layers may be horizontal or inclined and may be placed by truck with minor blading by bulldozer or other mobile equipment. Motor scrapers are ideally suited to constructing layer dumps.

Alternatively the layers may be placed by long push blading from semi-mobile tipheads. The latter technique has proven very successful at a north Auckland cleanfill operation where tied back shipping containers are used to create safe clean semi-mobile tipheads

The precise geometry of the layer dump is usually chosen to suit the equipment used. Economics play a very important role and particular care must be taken to optimise haul and blade work. In addition to the ability to control dump face angle precisely, layer dumping progressively loads the foundation and previous layers, resulting in highly consolidated and compacted dumps.

One concern sometimes raised about layer dumps is the potential for build-up of water pressures on impermeable surfaces on the top of each layer caused by the compactive effort of mobile equipment. Scarifying the surface by ripper before placing the next layer is sometimes carried out.

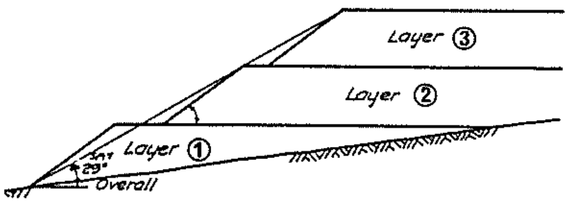


Figure 14: Layer Dump

Although not possible until later in the life of an operation, modest quantities of weaker material can be placed as a layer dump on top of an end dump (Figure 15). This aids the rehabilitation of an end dump without significantly affecting its stability.

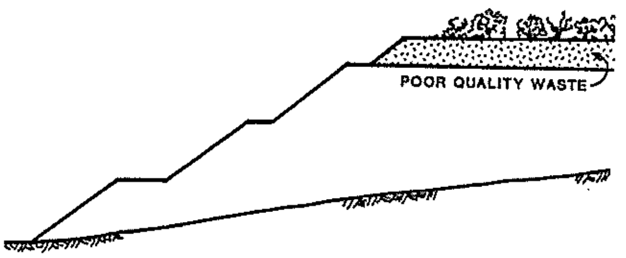


Figure 15: Layer Dumping of Weak Wastes

When waste is too weak to be layer dumped or in hilly terrain, or when there is insufficient space for layer dumping, use of a bund or retention dump is necessary (Figure 16). The weaker retained waste is usually placed in layers as described above.

Bunds are usually constructed by end tipping of well draining, strong, granular waste. Migration of retained waste into the bund can be controlled using filter layers or geotextile.

Ideally bunds are constructed on strong foundations. Weaker materials, especially organic soils are usually stripped off during the preparation of the bund "footprint". Where strong foundations are absent, particularly careful design and preparation of the foundation is required, and the bund must be constructed with flat slopes to control loading. In low permeability foundation situations, thorough under-drainage must be incorporated and piezometric monitoring and progressive loading may be necessary if the pore pressures are to dissipate sufficiently for the bund to resist the thrust of the retained waste.

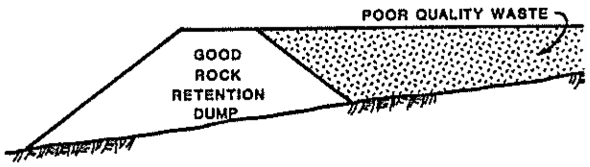


Figure 16: Bunded or Retention Dump

DUMP CAPACITY

The more compact the dump, the greater the capacity. The dump configuration with the smallest free face generally provides the greatest capacity.

Compactness of a dump and hence its capacity are dependent upon the geometry of the dump face and the underlying topography as illustrated by the curves in Figure 17.

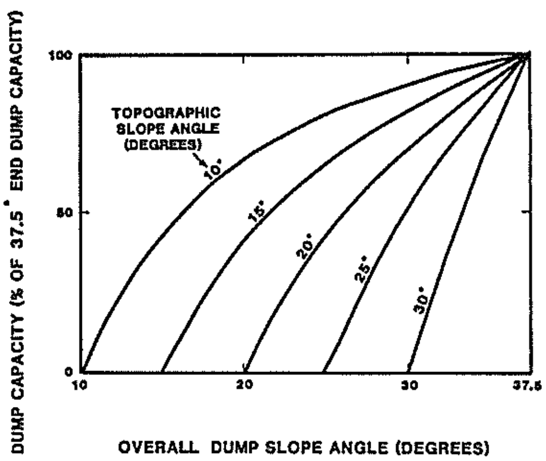


Figure 17: Dump Capacity on Sloping Topography

The main reason for moderating the dump face angle is for stability, as discussed above. Occasionally the dump face must be modified to accommodate access around the dump face.

In areas of flat terrain, with good foundation conditions and strong waste, dump design is straightforward and dump capacity is rarely a problem. In such situations, dumps are usually designed to minimise haulage costs.

In areas of steep terrain, the situation can be entirely different, as discussed below.

SIDE VALLEY FILLS

Side valley tipping (Figure 18) is often carried out where stream flows are too great or where stream diversion has not, or cannot, be considered. Side valley dumps have large free faces, offer small capacity and tend to be susceptible to scour and failure (Figure 2). Lack of compactness invariably leads to sprawling dumps and ever increasing haulage costs.

Because of their small capacity, susceptibility to erosion and instability, side valley dumps should be avoided, wherever possible.

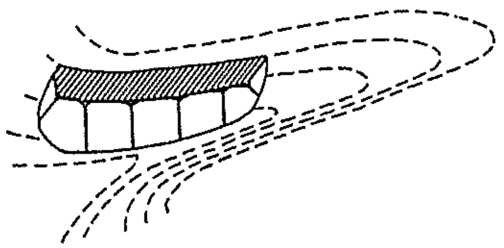


Figure 18: Side Valley Dump

HEAD OF VALLEY FILLS

By contrast with side valley dumps, head of valley fills (Figure 19) are compact, have smaller free faces and are inherently more stable. Head of valley dumps frequently have a capacity of at least double that of two side valley dumps in the corresponding situation.

The problem with head of valley filling is the control of surface runoff. Initially it may be possible to capture and divert streams around the head of the dump and to discharge into adjacent valleys. As the dump advances it may be necessary to channel the stream over the dump surface and discharge into more remote valleys. Construction of non leaky channels and selection of appropriate discharge structures are key considerations in the design of head of valley dumps.

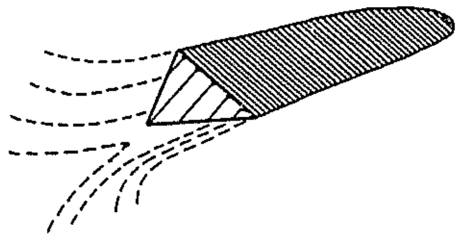


Figure 19: Head of Valley Dump

ACKNOWLEDGEMENTS

The author would like to thank operators and colleagues at a number of mines and quarries for assistance in developing and applying the concepts of waste dump design described in this paper.

Operations include CRA's Bougainville Copper Mine, Papua New Guinea; South African Iron and Steel Industrial Corporation's mines at Thabazimbi and Sishen, Republic of South Africa; the Molycorp Inc. Questa Mine, USA; and Otaika and Hunua Quarries, Winstone Aggregates Ltd. , New Zealand.

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STOCKTON OPENCAST COAL MINE CASE STUDY

D H Bell
University of Canterbury
Christchurch

T P Coote
Institute of Geological & Nuclear Sciences
Lower Hutt

G Harris
Coal Corporation of New Zealand Ltd
Westport

K Poynter
Coal Corporation of New Zealand Ltd
Westport

SYNOPSIS

High quality bituminous coal is produced from three separate opencast mines on the Stockton Plateau, and production in 1992 totalled 748 kt. The coal is extracted from an average 8-10 m thick seam by up-dip highwall advance after contract stripping of overburden, which consists of quartzose sandstones, siltstones and carbonaceous mudstones. Overburden is disposed of by end-tipping in waste dumps or by backfilling worked-out sections of the mines. Localised highwall instability involves toppling and complex wedge failures, collapse above former underground openings, and some translational movements on low-angle bedding plane shears. Water management is the principal environmental constraint to mining given the high rainfall on the Plateau (5-6000 mmy^{-1}), and the potential for acid generation from the coal measures rocks.

INTRODUCTION

The most important bituminous coal resources in New Zealand occur in the Buller and Greymouth Coalfields in the north-west of the South Island. Currently the principal mining activity involves opencast recovery of high fluidity bituminous coal from the Stockton Plateau, near Westport, where some 748,000 t were produced during 1992. Mining in the Stockton area commenced in 1872 and some 8-10 Mt of coal were produced from underground mines at Millerton before closure in 1969, due in part to uncontrollable fires resulting from spontaneous combustion (Todd, 1989). Three separate areas of slightly differing coal properties are presently mined to the south of Millerton, these being the Webb, Stockton No2 and Mt Frederick blocks, and total mineable reserves exceed 20 Mt. Discontinuous underground production of 0.3-0.4 Mt took place at the Webb Mine between 1922 and 1971, and the Stockton No2 Opencast commenced operations in 1967: a total of 3.6 Mt of coal is estimated to have been produced from the Webb Opencast to December 1988, and more than 2.2 Mt from Stockton No2 in the same period (Todd, 1989).

The Stockton Plateau varies in elevation between 400 and 1100 m above sea level, and annual rainfall exceeds 5000 mm with November to February being the wettest months (Todd, 1989). Much of the Plateau is characterised by discontinuous rock outcrops with stunted high altitude shrub and tussock cover, and only localised areas of bush: the acidic pakihi soils, where present, are unsuited to agriculture or forestry, and coal mining and limited tourism are the only current activities (Todd, 1989). There is road access to the mine area from Granity for equipment and personnel, and the coal product is delivered to the railhead at Ngakawau via a 7.7 km long, recently upgraded, aerial ropeway system: blending takes place at the upper loading station adjacent to the Webb Mine in order to meet particular coal specifications. The majority of the coal produced is exported via Lyttelton for use as a specialist material in the chemical industry and as a coking blend (Lavill, 1987): some of the coal is also used to fire kilns at the nearby Cape Foulwind cement works.

MINE GEOLOGY AND COAL EXTRACTION

The Stockton Plateau is a gentle (10° ±) dip slope, formed on Eocene Brunner Coal Measures rocks, that is bounded to the west and east by major NNE-trending faults: the coal-bearing strata are further offset by a series of northerly and westerly trending structures which are related to post-depositional faulting in the underlying basement. The Brunner Coal Measures consist of quartzose sandstones and grits, locally carbonaceous siltstones and mudstones, and bituminous coal seams in a sequence typically 20 - 50 m in thickness. Mining at Stockton is confined to the Mangatini Seam which, in the Webb Opencast area, is up to 16 m thick but more commonly ranges from about 8 to 10 m where currently mined: the floor of the seam is normally high in sulphur, as is the roof locally, but the majority of the coal is low in ash and sulphur. Close drilling, sampling and analysis has established quality variations within the mining areas to the extent that blends can be readily produced to meet particular customer requirements.

Mining operations at Stockton involve contract removal of overburden by drilling, blasting and truck haulage: the overburden is either stockpiled in end-tipped dumps adjacent to the mine site or backfilled into worked-out pit areas. Waste coal, typically high in sulphur or containing debris from past underground mining (such as timber or steel) is similarly stockpiled or dumped: facilities exist at the Webb Mine site for upgrading waste coal by screening, although there is no coal washing plant as such. Coal production involves truck loading by hydraulic excavators, and haulage to the loading station at the head of the aerial ropeway: bin storage capacity there is sufficient to meet current demand, and the recent upgrade of the aerial ropeway will permit annual production in excess of 1,000,000 t. Rail haulage to Lyttelton for export of Stockton coal adds significantly to the costs, and alternative methods of shipment have been investigated: these include the options of barging to Picton and the construction of a deep-water port near Westport, whilst alternative methods of removal from the Plateau (such as a conveyor belt line) have also been evaluated.

GEOTECHNICAL CONSIDERATIONS

The overburden to coal thickness ratio at the currently producing Stockton mines typically ranges between 2:1 and 3:1, and extraction proceeds by up-dip advance with a benched highwall to about 30 m. In the Webb and Stockton No 2 mine areas the overburden consists of coarse-grained quartzofeldspathic grits and sandstones, together with thinly interbedded sandstone/siltstone units and locally carbonaceous mudstones (Todd, 1989). Variable silicification along subvertical joint sets adds to the rock mass strength, and studies by Bell & Pettinga (1983) concluded that fragmentation during overburden blasting at the Webb Mine was strongly influenced by the complex lithological variation and the presence of closely spaced bedding laminations. The rock is drilled in 15 m lifts to provide an intermediate "catch bench", and blasting using slurry explosives generally results in satisfactory fragmentation if carefully monitored (Todd, 1989): some secondary blasting is necessary, especially in the coarse-grained grits and sandstones with widely spaced joints.

Detailed examination of rock material and rock mass characteristics in the overburden rocks at Webb Mine were made by Coote (1991), and a Geotechnical Hazards Plan at 1:2000 was produced to assist with future mine planning. Coote identified three separate lithotypes (coarse massive sandstones; laminated micaceous siltstones; carbonaceous mudstones), and determined air-dry unconfined compressive strengths in the range 24 to 72 MPa: the strongest lithotype was the carbonaceous mudstone and the weakest poorly cemented coarse massive sandstone, whilst specimen saturation was shown to reduce strength values by as much as 25%. Inverse relationships were obtained between porosity and dry density, the dynamic modulus of elasticity values ranged between 9 and 19 GPa, and the dynamic Poissons Ratio varied between 0.17 and 0.26: second cycle slake durability index data gave a range in averages from 62% for the poorly cemented coarse massive sandstones and 98% for the laminated carbonaceous mudstones. In general terms the rock material properties obtained for overburden at the Webb Mine by Coote (1991) are similar to those reported by Bell & Pettinga (1983): Todd (1989) notes uniaxial unconfined compressive strengths of between 10 and 50 MPa for samples of coal from the Mangatini Seam, with a generally well-developed cleat facilitating shovel loading without any requirement for blasting.

Bedding in the Brunner Coal Measures at the operating Stockton mines typically dips to the northeast at about 10°, except in the vicinity of basement faults which monoclinally fold and locally offset the sequence. At the Webb Mine Coote (1991) has recognised three principal joint sets which dip steeply (70°±), and also the presence of bedding plane shears of which some appear to be laterally continuous over hundreds of metres: the presence of groundwater within the fractured overburden rocks was also identified as a significant factor controlling batter stability. Defect studies and observation of the Webb Mine opencast workings has shown that highwall failure types include minor topples, complex wedge development in fault zones, collapse above old underground workings, and translational block slides: Coote (1991) considered that further batter failures up to about 1000 m^3 in volume could be anticipated. The Geotechnical Hazards Plan identified seven separate domains within the Webb Mine area, each being characterised by a distinct set of rock mechanics parameters influencing batter stability and therefore future mine operations. Coote (1991) recommended several measures to reduce instability, including modifications to highwall benching criteria and also improved surface and subsurface water control where possible: down-dip mining through the Augustus and Baynes Fault Zones was preferred, with a stand-off distance of at least 30 m.

ENVIRONMENTAL MANAGEMENT

According to Todd (1989) potential environmental constraints at Stockton can be grouped as 1) land tenure; 2) vegetation; 3) wildlife; 4) water management; and 5) historical sites. Although extensive areas of land controlled by the Department of Conservation (DoC) exist to the north and east of the current mining sites, the great majority of coal-bearing areas fall within mining privileges held by Coal Corporation of New Zealand Ltd (Coalcorp). Extension of ecological reserve status to parts of the Stockton Plateau has apparently been suggested because of the "unique" heathland vegetation present on the coal measures rocks, whilst wildlife habitat areas for the protected fernbird and the native landsnail are also relevant (Todd, 1989). Numerous historic sites exist in the area which are related to the earlier coal mining activities, and the former mining township of Millerton has been allocated to DoC in order to preserve its historic places value: part of the Granity Incline, which was constructed in 1891 as a series of self-acting endless ropeways for the haulage of coal from Millerton down the escarpment, has been restored for public access.

Water management is the most significant environmental constraint for coal mining on the Stockton Plateau. With annual rainfall in the range 5-6000 mm and rapid runoff during storm events, adequate surface water collection systems are essential during mining operations: in addition, northward draining streams at the Webb and Stockton No2 Mines are approximately normal to the highwall, and require diversion during the development of mining blocks. Surface (and subsurface) waters on the Stockton Plateau are naturally acidic due to the acid-generating potential of the Brunner Coal Measures, and Todd (1989) suggests that periods of low flow are of greatest significance in terms of stream acidity because of the absence of flushing. Resource consents for the mining licence areas require routine monitoring of surface water quality and quantity, as well as detention dams to limit sediment discharges into the headwaters of either the Ngakawau River (from the Webb and Stockton No2 Mines) or the Waimangaroa River (Mt Frederick Mine).

A further environmental consideration in mining at Stockton is the disposal of overburden, of which some 1.5-2.5 Mt are produced annually, and of high sulphur or contaminated coal for which there is no immediate market. As previously discussed the overburden material consists of quartzose conglomerates, sandstones and siltstones, together with some carbonaceous mudstones, and is either dumped in areas of no coal production potential or is backfilled into worked-out sections of the respective mines. No systematic compaction of the overburden is carried out, and perimeter drains are likewise not required: revegetation trials on some waste dump surfaces have been carried out, and further work is planned. The potential for acid water generation from stockpiled overburden and poorer quality coal has yet to be assessed, and the impacts of mining on the already naturally depleted aquatic biota has similarly not been quantified (Todd, 1989).

CONCLUSIONS

1. Opencast recovery of high quality bituminous coals is presently taking place from three separate mine areas on the Stockton Plateau: total coal production in 1992 was 748 kt, and an increase to a level exceeding 1 Mty^{-1} is feasible.

2. Mining typically takes place using hydraulic excavators and trucks after contract stripping of overburden involving drilling and blasting of the medium strength (25-75 MPa) quartzose sandstones, siltstones and carbonaceous mudstones: up-dip advance of the highwall is practised, with 15m high benches and catch berms.

3. Localised highwall instability results from toppling and complex wedge failures, and from collapse into former underground workings: the presence of bedding plane shears near the coal/overburden interface may result in translational slide failures.

4. Overburden is disposed of by dumping in areas with no potential for coal production, or by backfilling worked-out pit areas: some poor quality coal is stockpiled for screening or similarly disposed of.

5. Water management is the principal environmental constraint to mining, with regular monitoring of discharge water quality and quantity required as a condition of licensing: the stream waters on the Stockton Plateau are naturally acidic, and mining has the potential to increase acid water generation.

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DESIGN, CONSTRUCTION AND OPERATION OF TAILINGS DAMS IN NEW ZEALAND

T Matuschka
Engineering Geology Ltd
Auckland

SYNOPSIS

Three large goldmines are now operational in New Zealand (Martha; Waihi, Golden Cross; Waihi, Macraes; Eastern Otago). All use large dams for the storage of tailings. Design, construction and operation of such dams are reviewed and comparisons made with conventional water retaining dams.

INTRODUCTION

In recent years three large goldmines have been planned and developed in New Zealand at sites of previous mining activities which ceased operation some time ago. They are the Martha and Golden Cross Mines (Waihi) and the Macraes Mine (Eastern Otago). The Martha and Macraes Mines are open-pit, while Golden Cross involves both underground and open-pit mining. Martha began production in 1987, Macraes in 1990 and Golden Cross in late 1991. All three mines have large dams for the retention of processed residue (tailings) as well as dams for water storage and silt control purposes. The tailings dams associated with these projects have all been designed in accordance with modern design standards. Design features of the tailings dams are summarised in Table 1.

Tailings storage dams differ from conventional water storage dams in a number of ways which are reviewed later in the paper. One of the more important differences is the consideration of environmental aspects due to the potentially hazardous nature of the tailings and waste rock. Special importance is given to incorporating measures to mitigate and control seepage, as well as to monitor seepage quality and quantity. Rigorous water quality standards are embodied in the conditions of resource consents.

Another difference from conventional water storage dams is that dam construction and operation occur concurrently over the life of the mine. This has advantages and disadvantages. At the end of their relatively short operational life, tailings storage dams must be rehabilitated to a long-term safe state. However, provision must be made for monitoring and surveillance to continue beyond the operational life.

TAILINGS DESCRIPTION

Tailings can be defined as the processed residues or waste products from any number of industries including mining, industrial, chemical etc. Wide variations exist in the type of tailings that are produced by different industries.

Mine tailings consist of finely ground-up rock that remains after the mineral values have been removed from the ore. The grain size distribution depends upon the characteristics of the ore and the mill processes used to concentrate and extract the metal values. A wide range of tailings gradation curves exist. Consequently tailings materials may range from fine sands to clay-sized particles.

Tailings are normally transported to the disposal area as a slurry at concentrations varying from 30% to 55% by weight of solids to the weight of solids plus liquids. The coarser fractions of the tailings tend to settle out nearer to the point of discharge and so there are changes in properties from one side of the discharge beach to the other. In addition, there are vertical variations because tailings permeability decreases as the tailings consolidate. The permeability and consolidation characteristics of the tailings depend on the nature of the ore material and the grinding process. Determination of these parameters is important for assessing likely tailings densities which control necessary dam storage volumes. The tailings associated with the Golden Cross, Macraes and Martha Mines have permeabilities in the range of about 10^{-7} to 5×10^{-9} m/s.

DESCRIPTION OF TAILINGS STORAGE DAMS AND COMPARISON WITH WATER STORAGE DAMS

Tailings storage dams can consist of many types. In New Zealand, because of limited availability of large flat areas, goldmine tailings storage dams are either cross-valley (e.g. Golden Cross and Macraes) or in the case of Martha, a dike which abuts against a hillside.

Large tailings storage dams are normally built in stages. In the first stage an initial dam is constructed before the mining operation starts and is subsequently extended in stages or continuously during mining operations. Often in the first stage the dam may store water, either intentionally or because there is no way to release water. In these cases the initial dam should be designed as a conventional water storage dam.

Tailings storage dams are generally constructed from waste rock or tailings materials themselves. Many overseas tailings storage dams have been constructed from tailings materials, with the tailings being processed with cyclones to separate the sand sizes from the slimes. The sands are then used in the dam construction and slimes are deposited in the tailings pond. This method is not favoured in New Zealand or other seismically active regions.

Where dams are constructed from tailings materials three methods can be used; upstream, downstream or centreline. The downstream method is favoured in modern dams. Dams constructed from waste rock use the downstream method. All tailings storage dams associated with the Golden Cross, Macraes and Martha mines are constructed from mine waste rock with local borrowed fill also used in the initial dam construction.

Tailings storage dam design differs from conventional dam design [1] in that:

a) Unlike conventional water storage dams they cannot be breached at the end of their useful service and the valley allowed to return to its original condition. Tailings storage dams often store hazardous fluids and solids, which cannot be discharged at the end of the operating life of the mine. Therefore the rehabilitation aspects of tailings dam operations require careful study. At the Golden Cross Mine new technology has been developed to drastically reduce cyanide concentrations in the tailings by recycling the cyanide reagent.

b) The bulk of material stored behind the dam is saturated, relatively impervious slurried tailings at various stages of consolidation. The consistency of the tailings may range between the solid state and the semi-fluid state. Under seismic loading saturated tailings may liquefy, becoming a fluid of high unit weight and therefore placing additional loading on the dam.

c) Most of the dam construction is carried out by the mining operators with the dam being raised as required to stay ahead of the rising tailings pond. Because tailings storage dams usually reconstructed slowly over a period of many years, the designer is able to select a design and then check its performance, making modifications as required throughout the long construction period. This allows more flexibility than is available for design of conventional water retention dams. Historically, construction by mining operators presented the risk of less attention to quality control than would be the case for conventional dam construction. However, this is not the case nowadays, especially for larger dams.

d) Tailings storage dams normally are subjected to only a nominal amount of drawdown of the overlying free water as the tailings deposits are continually rising as the dam is raised. Consequently tailings storage dams develop a significant amount of upstream support from the tailings in the pond and this feature can be utilised in their design.

(e) In tailings storage dams there is not the same freedom of material selection that there is with water storage dams because for economic reasons, most of the construction material must come from the mine-pit. However, this concern is often offset by the fact that large quantities of waste rock are available so that conservative shoulder slopes can be adopted.

(f) Mining operations normally continue throughout the year in all but the most severe weather conditions, including the disposal of waste rock and the construction of tailings storage dams. In comparison, water storage dams are normally only constructed during months of favourable weather. Consequently tailings storage dam design must make some allowance for this in the strengths adopted and in the standards specified. Provision must be made for placement of unsuitable materials in appropriate zones. Construction may be programmed so that most structural fill is placed during favourable summer months with largely non-structural fill in winter months. Material suitable for structural fill may be stockpiled in winter for later use in summer.

TAILINGS STORAGE DAM DESIGN

General

In designing tailings storage dams there are two basic items to be considered. The first is structural stability of the dam against failure by sliding, slumping, overtopping, piping etc. The second is environmental safety which relates to the containment of any hazardous materials that might be stored. Guidelines for design are contained in ICOLD publications [1,2] and a brief review of current design criteria follows.

Storage Requirements

Tailings storage dams must be designed to safely store tailings plus accumulated water, including large storms, with an adequate freeboard. This requires tailings pond storage curves to be developed along with estimates of tailings production, tailings density and total volume of free water in the pond for an average year. Large tailings storage dams are also normally designed to accommodate a Probable Maximum Flood (PMF) event on top of the anticipated tailings volume plus average free water, with an adequate freeboard to prevent overtopping. In some cases this can be difficult to provide in the early stages of development and so less demanding criteria may be acceptable for the first months of operation.

On the basis of the above information, target crest levels for tailings storage dams can be specified. However, it is noted that storage requirements are likely to change throughout the life of the dam as mining operations may expand or contract depending on market forces or as additional resources are discovered. Therefore monitoring and feedback are essential to confirm design assumptions and to review target crest levels.

Hydrology and Hydraulics

Tailings and water stored behind the dam may be of a hazardous nature, and in such cases, no discharge is allowed unless surplus water is treated. Treatment is expensive so normally the stored water is recycled, as far as is possible, for use in processing of ore. However, water treatment plants are often required to treat excess stored water as well as water from mine de-watering, tailings and leachate seepage as well as surface runoff.

Water balance is important. If too much water enters the pond, water treatment may be required to maintain a safe freeboard level. The surplus water also has an adverse impact on tailings densities. If tailings are exposed to air drying they achieve a denser state than if submerged.

In dry climates, the problem is the scarce availability of water. To allow operations to continue in dry periods, and as a source of potable water, water storage dams are often required.

Water balance studies are also very important in assessing potential environmental effects. Consideration of such effects is necessary for assessing the need for size and characteristics of water treatment plants and affect the way in which surface and seepage waters are managed.

In New Zealand tailings storage dams are normally located so that the catchment contributing to the pond is minimised or diversion drains are constructed to divert runoff around the ponds. Tailings storage dams associated with the Golden Cross, Macraes and Martha Mines all have diversion drains to minimise runoff into the tailings ponds.

The design of major tailings storage dams, where failure could result in loss of life and extensive property damage, should be based on the probable maximum flood (PMF). For dams where no discharge is permitted, sufficient freeboard to allow the safe storage of the PMF on top of the tailings must be provided.

Structural Design

Tailings storage dam design should conform with modern dam design practice taking into account their special characteristics. Geotechnical investigations are necessary to determine foundation conditions and the properties of the material from which the dam is to be constructed. Foundation investigations are normally conducted in conjunction with hydrogeological studies to determine groundwater characteristics.

Tailings storage dams are often designed based on the most probable foundation conditions that can reasonably be expected, rather than the most unfavourable conditions which could be deduced from foundation exploration. However, if this approach is adopted it is necessary for careful observation during construction to allow amendments as necessary.

Designs should bear in mind that mining operations are principally concerned with recovery and processing of ore. As such, mine operators have an added difficulty of fitting together a mining and dam construction programme. Mine operators require that designers appreciate that mining should not generally be constrained by the demands of tailings storage dam construction. Design and proposed construction of tailings storage dams must be simple, cost effective, flexible and reliable. The design should include drawings and specifications which detail the extent of work, materials to be used and quality control standards.

A special feature of the tailings storage dams at Golden Cross and Martha Mines is the necessity to encapsulate unoxidised waste rock with low permeability material. This is necessary because of the potential for unoxidised rock to produce leachate containing unacceptable chemical and heavy metal concentrations. This is not an uncommon problem associated with gold mines because of the presence of sulphides. Mining exposes a larger surface area to the atmosphere than when *insitu* and results in a greater oxidation rate. Compaction and encapsulation reduces the availability of oxygen and hence the rate of leachate production.

Seismic Design Considerations

The effects of earthquakes must be considered in the design of tailings storage dams. Tailings storage dams associated with the Golden Cross, Macraes and Martha Mines have been designed using two levels of earthquake shaking, consistent with modern dam design practice. The lower level of shaking is termed the design basis earthquake (DBE) or operational basis earthquake (OBE) and typically corresponds to ground shaking with a return period of 150-200 years. Estimates of shaking can be obtained from probabilistic seismic hazard studies. The dam should withstand this loading with no significant damage. The dam should also be designed to resist, without collapse or loss of the reservoir, ground shaking generated by large rare earthquakes. Often this level of shaking is taken equal to that associated with the maximum credible earthquake (MCE). The MCE is defined as the largest earthquake that appears capable of occurring along a fault in the existing tectonic environment.

Seepage

Seepage control is a critical aspect in the design, construction and operation of tailings storage dams as it affects stability, internal erosion due to piping and pollution of ground and surface waters downstream of the dam.

Techniques for minimising seepage flows include the use of low permeability compacted structural fill, foundation cutoffs and impervious blanketing as well as appropriate site selection in the first place. In addition, the method of tailings deposition can affect seepage flows. Tailings themselves often have low permeabilities and can be deposited to provide a low permeability blanket over the upstream shoulder of a tailings storage dam as well as the natural ground forming the pond. Furthermore, the development of a beach along the upstream shoulder and abutments of a tailings storage dam prevents direct contact of the tailings liquor and reduces potential seepage quantities.

The tailings storage dams at Golden Cross, Macraes and Martha Mines have all been designed as zoned fill structures with low permeability zones on the upstream shoulder providing the primary defence against seepage. The design permeability of these zones vary from 10^{-7} m/s - 10^{-8} m/s. The Golden Cross tailings storage impoundment area also has a blanket consisting of compacted fill and reworked natural soils with a design permeability of 10^{-7} m/s.

As with conventional water storage dams seepage does occur and so adequate filters and drains should be provided to intercept and allow seepages to safely pass through foundations, abutments and dams. Seepage can be collected by underdrains beneath the tailings pond, subsoil drains along the dam/abutment interface, internal chimney drains within the dam, intermediate and downstream toe drains and pump wells. Provision should be made for collection, monitoring and treatment of such seepage. Seepage from drains upstream of the dam (underdrains etc) can flow by gravity in sealed pipes through the dam, or be pumped via submersible pumps located down inclined wells on the upstream shoulder or self-supporting vertical wells.

Instrumentation and Monitoring

Instrumentation and monitoring are an important part of modern tailings storage dam design and include piezometers for monitoring pore pressures, wells for sampling water quality, flowmeters and weirs for monitoring flows and benchmarks for monitoring settlements and movements. Observations are important for evaluation of structural stability and for monitoring environmental conditions. Due to the long time of construction of tailings storage dams, there exists the possibility for amending the design on the basis of ongoing observations resulting in improvements to performance and possible cost savings.

CONSTRUCTION AND SUPERVISION

Typically, construction of tailings storage dams is in two stages. The first stage is the construction of an initial dam prior to deposition of tailings. The second stage involves ongoing construction throughout the life of the mine. Often the first stage may form part of a separate contract associated with construction of the process plant and other works necessary before ore processing can commence. The second stage is normally undertaken by the mining operator or on contract under his direct supervision. Where contractors are used it is better not to separate the operations of mining the ore and construction of the tailings storage dam. The operations are dependent on one another.

The Design Engineer should be involved in the construction phase in at least an advisory role with visits to the site. This is necessary to ensure design intentions are being fulfilled and to allow revision and adjustment to the original design to best suit operating and construction requirements.

Mine operating staff responsible for ongoing construction should be made fully aware of design details and the reasons for their incorporation into the design. A document summarising such information is useful for the mine operating staff.

Full-time supervision is necessary to ensure that specified construction standards are met for large tailings storage dams. During construction of the initial dam, representatives of the design staff should be involved as this normally is the critical stage.

For the tailings storage dams associated with the Golden Cross, Macraes and Martha Mines initial dam construction has been let by tender with drawings and specifications detailing the work and quality standards. Such standards are not too different from those associated with other types of dam construction. Construction of the second stage (i.e. operational stage) at Golden Cross, Macraes and Martha is by the mining contractor, rather than a separate dam contractor, with a schedule of rates contract similar to the first stage. The mining operators supervise the construction and undertake control testing with 2 - 3 staff dedicated to this depending on demand. They are also responsible for ensuring that accurate as-built records are maintained, including photographs of critical areas, as well as monitoring and inspection roles as discussed later. Typically this work is audited by a Peer Review Panel, appointed as a condition of the Water Rights, as an additional safeguard.

OPERATION

Tailings storage dams differ from conventional water storage dams in that operation (i.e. discharge of tailings) generally occurs concurrently with construction and the operational life is short, typically less than 15 years.

Adequate freeboard should be maintained at all times to store the design storm (PMF for large tailings dams) on top of the normal pond operating level. This requires adequate monitoring of tailings and dam crest levels along with review of anticipated tailings production levels.

Tailings are normally discharged from the dam shoulder so that a beach is maintained between the free water in the pond and the upstream face. Water is reclaimed from the tailings pond for re-use in the mill. This is achieved with either floating pontoons with pumps mounted aboard or by decant towers and pipelines. The former is preferred as they avoid pipelines passing through the dam, which if they fail, can cause tailings and effluent to be discharged downstream.

MONITORING AND INSPECTIONS

Regular monitoring of instrumentation, tailings and dam crest levels and inspections of critical areas form an important part of the safe operation of a modern tailings storage dam. Monitoring should commence prior to filling to establish baseline values and a thorough visual inspection should be undertaken by the designer prior to deposition of tailings.

Guidelines for the frequency of monitoring instrumentation and undertaking inspections during operation should be specified by the designer. At Macraes and Martha, piezometers and seepage flows are measured at least once every two weeks. Ideally a monitoring and surveillance manual should be produced by the designer to be used by site staff. It should include general background information and a description of instrumentation, as well as listing instrumentation to be monitored and items requiring inspection. General background information should cover a description of the dam, assignment of responsibility, requirements for reporting the results of monitoring and inspections and public warning procedures in the unlikely event of a breach. Standard forms for reporting results should be devised and included in such a manual.

To be effective it is necessary for the results from monitoring and inspections to be conveyed to the designer to enable rapid evaluations of their importance. Critical levels can be identified in many cases and included in a monitoring and inspection manual which if exceeded should trigger immediate response.

Much of the monitoring and routine inspections can be conducted by the same staff involved in quality control of the earthworks. This is the case at Macraes and Martha Mines. However, at least once a year, during operation, a detailed inspection of the dam should be undertaken by the Designer.

Monitoring and surveillance should continue past the operational phase into the long-term to ensure that no release of hazardous material occurs and that the structural integrity of the dam and associated drainage works is maintained.

REHABILITATION

Rehabilitation of tailings storage dams is an important phase so that the dam and tailings are left in a long-term safe state. Design features need to be incorporated to protect against extreme events such as earthquakes and floods as well as long-term concerns relating to water erosion, wind erosion, surface and subsurface seepage, weathering of construction materials, clogging of filters or drains, settlements as tailings consolidate, leaching, chemical reactions, animal disturbances and penetration by plants and trees. Many of these issues are addressed at the initial design stage, but some decisions can be left until later. Design solutions must be simple and robust, requiring minimal ongoing maintenance.

Rehabilitation often involves draining the pond and diverting surface water, sealing the surface of the tailings pond, providing vegetative cover and providing permanent drainage and spillway facilities that might be required to handle surface runoff. Some of this work can be completed during the operational stage while the dam is being constructed. For example, downstream shoulders can be sealed, grassed and surface drainage constructed. Other works cannot be finalized until mining ceases. For example, surface drainage on the tailings pond because its final level is not known with any certainty until then.

Nowadays rehabilitation issues must be properly addressed as part of the approval process and appropriate measures included in the design. Prior to closure of an operating tailings storage dam a final rehabilitation plan should be prepared. It should address the issues discussed above as well as outlining requirements for long-term monitoring and surveillance. It should also outline provisions for long-term funding to enable maintenance, monitoring and surveillance to be continued.

REGULATORY REQUIREMENTS

The Golden Cross, Macraes and Martha Mines have been through rigorous evaluations in the permitting process to obtain water rights and mining licences, with a number of special conditions attached. These projects were licensed prior to the Resource Management and Building Acts, although extensions to the Macraes Mine have been subject to consent approval under the Resource Management Act. The conditions require good principles for modern tailings storage dam design. A summary of the more important conditions follow:

a) The water rights set water quality standards for all discharges that are principally modelled on USEPA standards. As far as design of tailings storage dams and associated works (i.e. diversion drains, spillways) is concerned, the water rights have required Management Plans to be produced which describe in detail how construction will be managed to meet the water quality standards.

b) Designs are required to be undertaken by people with appropriate qualifications and experience.

c) Water rights have included the requirements for a Peer Review Panel to be appointed to review design prior to construction and any design modifications during construction, inspect construction at critical stages and to review programmes for monitoring and inspection and their results. This concept is common for other types of dams and has worked well in the case of large tailings storage dams in New Zealand.

d) The water rights also stipulate design floods to be used in assessing dam freeboard levels and in the design of diversion drains, spillways and silt ponds.

e) Requirements for monitoring and inspection are included. In some cases they are general and require the mining operator to prepare a programme, subject to Peer Review Panel approval, to the satisfaction of the Regulatory Authority and in some cases they are quite specific and stipulate that annual inspection reports shall be produced.

f) Rehabilitation is an important part of tailings storage dam construction to ensure that it is left in a long-term safe and stable condition. Water Rights require submission of detailed proposals in this regard with review by a Peer Review Panel and provision of bonds or funds as a contingency for completing rehabilitation, maintenance and ongoing monitoring at the completion of mining.

The Resource Management and Building Acts require for consents to be given by both Regional Councils and District or City Councils. In most cases, such organizations will not have all the necessary expertise to evaluate mining projects and will need to hire consultants. There are advantages if the same consultants can be used by the different territorial authorities.

SUMMARY

Tailings dams differ in a number of ways from conventional water storage dams. Public and owner expectations for the acceptable performance of such dams are met by the criteria adopted for design, construction and operation. The criteria adopted for recent major tailings dams associated with goldmining operations in New Zealand are reviewed in this paper.

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1. ICOLD Bulletin No 74 (1989) ***Tailings Dam Safety Guidelines.***

2. ICOLD Bulletin No 45 (1982) ***Manual on Tailings Dams and Dumps.***

TABLE 1: FEATURES OF NEW ZEALAND TAILINGS DAMS

	Golden Cross	Macraes Concentrate	Macraes Flotation	Martha
Designer	Tonkin & Taylor	Woodward-Clyde/ Engineering Geology	Woodward-Clyde/ Engineering Geology	Tonkin & Taylor
Height (m)				
-centreline	63	59	120	48
-toe to crest	119	78	135	52
Crest Length (m)	750	700	2,000	1600
Shoulders (H:V)				
- upstream	2.5 : 1	2.0 : 1	1.4-2.0 : 1	2.5-2.8 : 1
- downstream	4.0 : 1	2.5 : 1	2.0-3.6 : 1	3.6-4.5 : 1
Tailings Storage (m^3)	7,500,000	1,200,000	25,700,000	8,500,000
Embankment Volume (m^3)	3,500,000	1,060,000	5,900,000	16,500,000
Embankment Type	zoned earthfill	zoned earth/ rockfill	zoned earth/ rockfill	zoned earthfill
Construction Materials	volcanic ash and weathered andesite	schist	schist	volcanic ash and weathered andesite

ASPECTS OF TAILINGS DAM DESIGN FOR THE GOLDEN CROSS MINE

T J E Sinclair
Tonkin & Taylor Ltd
Auckland

and

C J Freer
Tonkin & Taylor Ltd
Auckland

SYNOPSIS

The Golden Cross mine in the Coromandel, includes a tailings dam which incorporates the main waste rock stack as the downstream shoulder. This paper describes some of the special issues for which the dam had to be designed including zoning the dam to take account of both structural and geochemical considerations, internal and under-drainage measures and provisions for accommodating ground rupture on potentially active faults.

INTRODUCTION

Tailings dams take many forms. This paper is concerned with the usual concept of a dam whereby a separate structure is constructed to "impound" or "retain" tailings without using the tailings to form part of the dam. In this respect the physical concept is similar to that of a water retaining dam. However there are several aspects of design and construction for tailings dams which differ from those for water retaining dams. Some of these would be general to many tailings dams and some more specific to the Golden Cross Dam. For example:-

- Tailings dams impound solids which tend to influence the stability and seepage characteristics, particularly on the upstream shoulder.

- There is almost no risk of rapid drawdown for a tailings dam, thus nullifying one of the most critical design issues for water retaining dams.

- The construction materials for a tailings dam tend to be governed by the mining operations and this may place severe constraints on the engineering nature and quality of available materials. For water retaining dams, there is usually an opportunity to select borrow areas on the basis of suitability and convenience.

- Mine waste materials are usually sufficiently plentiful to allow flatter than normal downstream slopes. The ability to use the dam as a waste stack, therefore, tends to remove the risk of a complete dam break.

- The differences in the environmental impact risks are probably the most significant. For tailings dams the primary issue is that of ensuring security of the impoundment system against escape of toxic substances. For water retaining dams, the primary issue is the catastrophic dam break.

- Because mines tend to have a limited lifetime, tailings dams and water retaining dams have very different long term and short term design requirements. The latter would be built within a definite pre-operational construction period and would have an operational life of "hundreds of years" with generally similar impoundment conditions. Tailings dams, on the other hand, would tend to have a staged construction programme through the mine life of "tens of years", with a long term design condition very different to the operational conditions.

The purpose of this paper is to illustrate how these differences have influenced the design of the tailings retention dam for the Golden Cross gold mine at Waitekauri, Coromandel, particularly with regard to geotechnical issues. Space constraints preclude discussion of the overall scheme, design and construction details and many other issues which may also be of interest.

BACKGROUND

The Golden Cross mine is designed for both open pit and underground mining of gold bearing ore. The processing results in a tailings slurry which is pumped to a tailings impoundment area in a nearby valley. The tailings are retained by a "main" dam across the valley and a "saddle" dam to one side. The surface water run-off from the catchment is diverted around the tailings storage area and under-drainage is provided for the streams within the area, as is normal practice for such facilities. The main tailings dam and saddle dam are constructed of overburden and mine waste materials, particularly from the open pit. Other features of the mine include process and water treatment plants, ore stockpiles, silt retention dams and various road and drainage works.

The main features of the dam are shown on Figure 1.

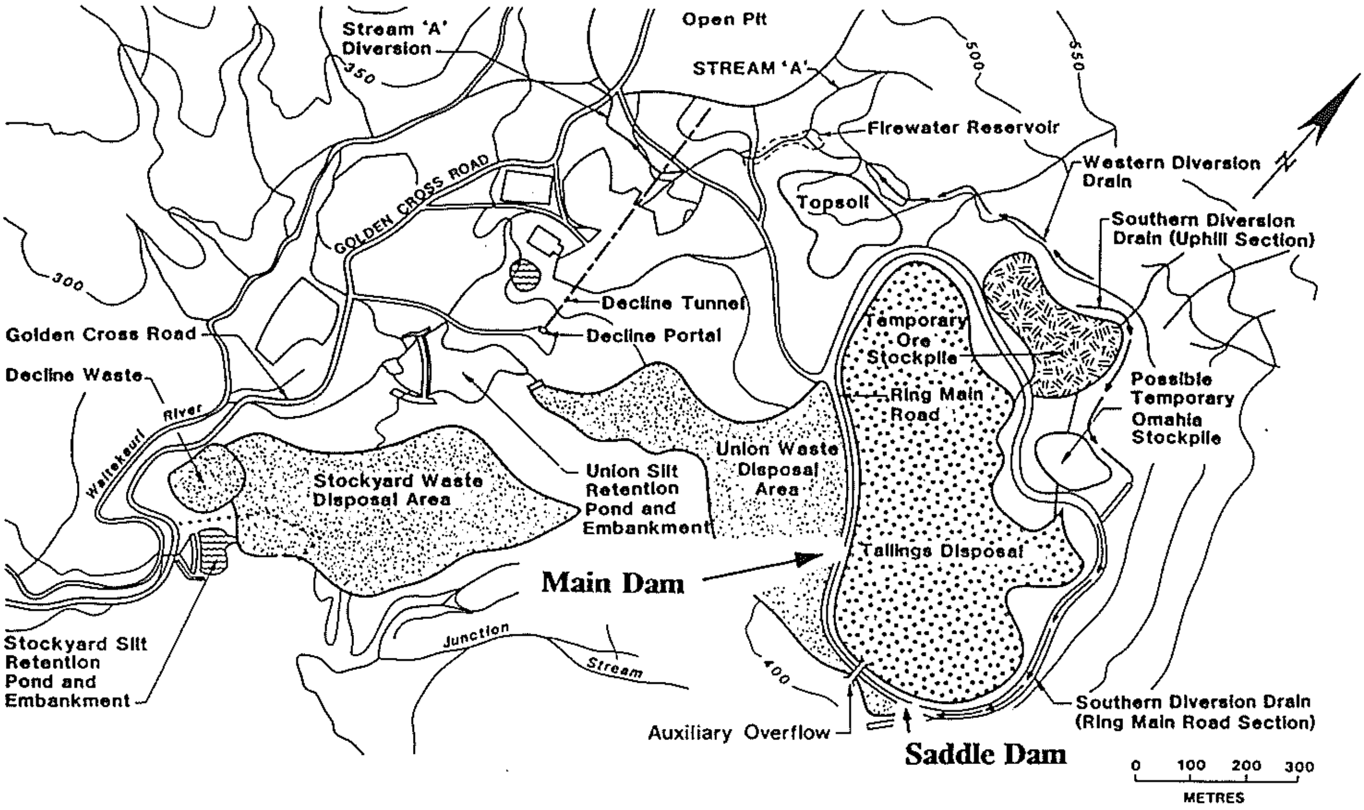

Figure 1: Layout of the waste disposal system for the Golden Cross Mine

Site investigations were carried out for all areas of the project, using both drilling and test-pitting. A detailed groundwater study was carried out together with other design and environmental studies. A detailed geological/geotechnical mapping exercise revealed the presence of several faults through the general area, some of which have been termed "active" faults for design purposes. A site specific seismic hazard study was carried out for the project, providing design acceleration and response spectra, and highlighting the need for special provisions to allow for possible movement on the faults.

STRUCTURAL CONSIDERATIONS

Design Influences

The two main issues which have influenced the structural design of the Golden Cross tailings dam are:-

- Staged construction
- Available construction materials

For this particular project, there were obvious economic benefits in using the open pit overburden and mine waste as construction material for the dam and, at the same time combining the mine waste stack and dam as one structure.

The originating source of mine waste and the rate at which it is produced is determined by the mine planners and detailed in the "pit schedule" (or "mine schedule" if underground as well as open pit is considered).

Both of the above issues depend on the pit schedule which, ideally, should be finalised prior to detailed design of the waste management system. In practice, however, the mining schedule tends to change and the design must therefore have some flexibility.

There are, of course, many other technical issues which have influenced the structural design and some of these are discussed later in this section.

Staged Construction

It is self evident that economic considerations would demand a minimum practical outlay for mine start-up, followed by on-going construction through the mine life, to an ultimate configuration which might not be realised until many years after start of construction. Design, however, must consider these stages in a different order:-

i) Design commences with the final operational conditions; final tailings storage requirements give the ultimate crest level, and the final waste rock quantities give the total volume of the dam.

ii) Design for start-up is governed by a selected minimum tailings storage volume (and hence crest level) and the requirement for a minimum - volume starter dam with maximum side slope gradients.

iii) Design for intermediate conditions is governed by the mine schedule (or "pit schedule" if only open pit considered). The dam cross-section must be designed:-

- to take all waste fill in accordance with the schedule of waste materials being generated
- to allow the crest to rise according to the tailings storage requirements
- to have a minimum safe section at all times, assuming that suitable structural fill will always be in short supply
- to accommodate weak fills at times when they need disposal (most likely at very early stage of construction)

Structural Zoning

Initial appraisal of the site investigation data and pit schedule soon indicated that available fill materials would be highly variable and, indeed, structural fill would be very limited. It was necessary to zone the dam, therefore, to address the following objectives:-

- Good quality structural fill would be required on the upstream shoulder to ensure a maximum feasible upstream face angle
- Low permeability fill is desirable on the upstream face
- There must be areas to place weak (and sometimes very weak) fill at the early stages of construction
- A "cap" of good quality structural fill is necessary to resist magnified crest accelerations (see later discussions)
- Zoning must be compatible with the staged construction previously described and the intermediate dam crest embankments

Miscellaneous Stability Considerations

a) Saddle Dam

The saddle dam was not part of the main dam incorporating the waste stack and was therefore "optimised" to give a minimum cross-section using only structural fill. The design was governed primarily by foundation conditions which were relatively unfavourable compared with foundations for the main dam. Both upstream and downstream shear keys were required to ensure adequate factors of safety under the initial static and dynamic loading conditions.

Some of the features of the Saddle Dam and the critical stability analyses are shown on Figure 2.

b) Main Dam : Upstream Shoulder

This was relatively straight forward due to the presence of tailings which were assumed to have no shear strength but offered stability counterweight. The

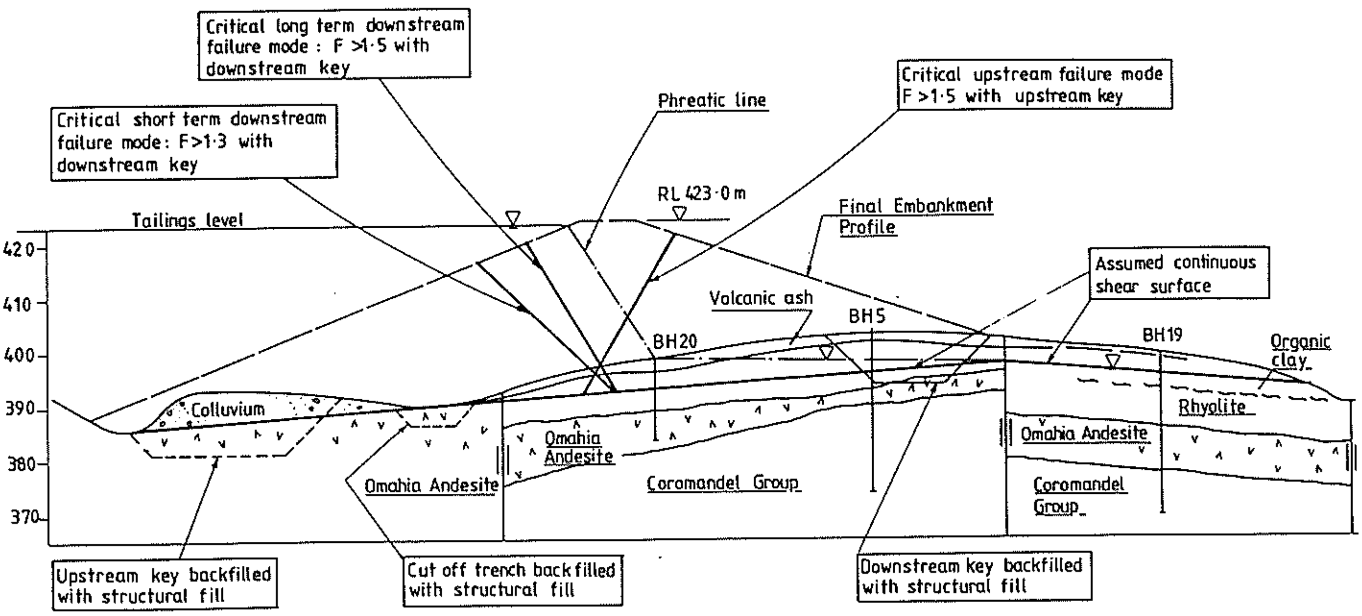

Figure 2 : Features of the Saddle Dam influencing stability

selected face angle was governed by the assumed strength of the structural fill in this area. In order to accommodate more storage of very weak fill material at an early stage of construction, part of the upstream face was steepened from a slope of 22°(2.5:1) to 26.5°(2:1), and a weak-fill buttress placed against it.

c) Main Dam : Downstream Shoulder

Initial, simplified, two-dimensional stability analyses indicated that this was the critical stability issue, primarily because of the need to place very weak fill materials at the base of the dam.

With more detailed analysis, it was necessary to take account of several factors which clearly would enhance the stability, namely:

i) The confining valley for the downstream shoulder (and waste stack) is narrow, steep-sided and twisting, so that the three-dimensional effects would be considerable.

ii) A constriction in the valley offered the opportunity to place a high-strength rock fill buttress or shear key within the fill. This served as an intermediate dam to retain the very weak fill material ("slop") on the upstream side and also provided significant shear resistance for the assumed deeper shear surfaces.

iii) Laboratory tests on samples of the weak fill indicated that there would be significant increases in shear strength as the dam was raised through the planned nine year period of construction.

Factor (i) is difficult to quantify. For this project a worst-case scenario was adopted (ie assuming a broken section following the valley invert line and line of deepest fill) and accepting the lower-than-normal value of 1.3 for factor of safety. The topography and geometry of the valley would clearly provide for a better factor of safety than this.

The intermediate rock fill buttress/shear-key of Factor (ii) proved very effective in increasing the factor of safety and was relatively economic due to the narrow constriction in the valley.

Figure 3 shows a simplified section of the Main Dam following the valley invert, together with some critical failure surfaces.

The effects of Factor (iii) are easy to include in the stability analyses, but clear thinking is necessary. If an effective stress method is employed, the pore pressures must allow for increases due to loading of overlying layers (the $\bar{B}$ effect) as well as decreases due to consolidation with time. This, in effect, gives a reduced $\bar{B}$ or 'r_u' for input in the computer model.

At this point, it is worth clarifying a common confusion:-

$\bar{B}$ - is a 'response' parameter which defines the change in pore pressure due to a change in total stress (or loading) i.e. $\bar{B} = \Delta u/\Delta\sigma_v$.

r_u - is a "state" parameter which is a convenient way of defining the pore pressure at a point in terms of the vertical total stress at that point, i.e $r_u = u/\sigma_v$.

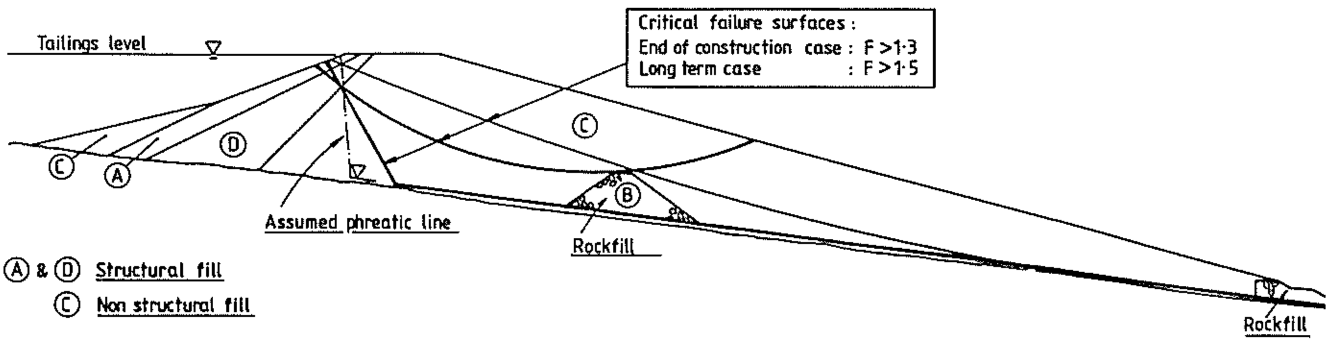

Figure 3: Simplified section through Main Dam

These are not the same but can be equivalent for some analyses if there is no hydrostatic component.

Figure 4 illustrates this for a case where the process of pore pressure generation and dissipation for an element of soft soil (say) near the base of the dam can be simplified with separate undrained and drained phases.

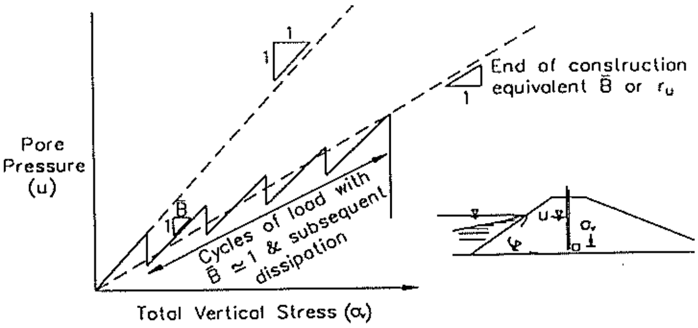

Figure 4: Generation and dissipation of pore pressures during dam construction

If a total stress method is employed, the initial undrained shear strength must be known, together with an estimate of the rate at which shear strength will increase. The initial shear strength can be governed to some extent by the specification. The final available change in shear strength can be determined from consolidated, undrained triaxial tests with pore pressure measurements (CUP tests), giving a characteristic s_u/p' ratio. The proportion of the available increase in strength realised over the construction period must be estimated.

Whichever method is employed (total stress or effective stress) the design assumptions are exactly equivalent (ie. of dissipation or available increase in shear strength). For the Golden Cross project it was assumed that only 33% of available increase in shear strength (or r_u of 0.67) would occur by end of construction. Piezometers installed in the fill and monitored throughout construction to date, indicate that conditions are far more favourable then assumed for design.

d) Intermediate Stability

As discussed previously, the intermediate condition between the starter dam and the final profile had to be designed to give maximum flexibility in the programme for placement of mine waste. It was necessary to work out how far the dam crest could be constructed ahead of the bulk of the downstream shoulder, assuming that this downstream shoulder would always be of uncertain strength. Figure 5 shows some of the dimensions to be determined at this stage.

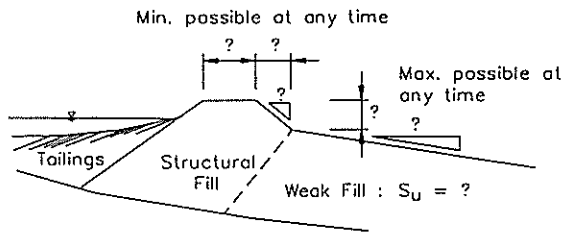

Figure 5: Factors to be evaluated for intermediate stability condition

Seismic Stability

Although the construction period would be relatively long (9 years), the seismic stability checks were considered only appropriate for the long term condition.

In this respect the seismic stability analysis would be similar to any other type of dam. Because of this there is little point in dwelling on the subject though it is, of course, of considerable importance. In summary, the general stages of the seismic design procedures can be listed as follows:-

i) Site specific seismic hazard study to give levels of peak ground acceleration and response spectra for Operating Basis Earthquake (OBE) and Maximum Credible Earthquake (MCE).

ii) Response analysis of the dam to give crest acceleration and profile of inertia forces through the dam, taking account of probable stiffness and damping variations in the construction materials.

iii) Stability analyses to give yield accelerations for various modes of failure.

iv) Displacement estimates for conditions where accelerations are likely to exceed the yield accelerations.

As is usual for most dams, the design brief allowed for the following design criteria:-

- OBE: No damage to dam (ie acceleration to be less than yield acceleration).

- MCE: No catastrophic failure but limited damage (ie displacement) acceptable.

GEOCHEMICAL CONSIDERATIONS

The materials used to form the embankment are sourced mainly from the open pit but also from haul road excavations, underground workings, borrow areas and general earthworks. These materials may be classified in terms of one of the following principal generic categories:

- Surficial ash, rhyolites and colluvium (Ash)
- Omahia Group Andesite (Omahia)
- Argillic Coromandel Group Andesite (Argillic)
  - oxidised
  - unoxidised
- Non-argillic Coromandel Group Andesite (Non-argillic)

The first two of these (Ash and Omahia) are non-mineralised and do not present a risk of acid leaching. The mineralised Coromandel Group materials are high in sulphides and have high acid producing potential if unoxidised. It is the process of oxidation which results in the acid leachate. For naturally oxidised material, the damage is done - over geological time scales - and these no longer present a risk. Very little of the non-argillic is naturally oxidised and this material tends to be more free-draining as a fill than the argillic, and consequently provides the greatest risk of acid leachate.

The control of acid leachate potential, therefore can be achieved in two ways:

a) Prevention of oxidation: Ensuring low air voids by compaction, mixing or encapsulating with fine-grained materials.

b) Control of seepage: Placing low acid producing material in controlled seepage zones, inhibiting seepage through the main stack and providing a drainage system to collect all leachate.

The embankment can be zoned, therefore, for both these objectives; prevention of oxidation and control of seepage being mutually compatible within the main downstream shoulder. These geochemical requirements, however, must also fit into the zoning for structural objectives as discussed previously. Clearly, the specification for placing and compacting materials is closely related to the spatial zoning.

ZONING

To address both the structural requirements and the geochemical control requirements discussed above, the dam has been "zoned" to receive the various material types, which are then placed in accordance with a specification related to the structural and geochemical objectives. In order to maintain maximum flexibility in earthworks operations and provide against a shortfall in suitable structural fill, the specification is necessarily complex; defining five structural types (Types 1 to 5) and three geochemical types (X, Y & Z) to be placed in the seven zones in various combinations, each with specific placement and compaction criteria. The zones are illustrated on Figure 6 and Table 1 summarises the various requirements and objectives.

INTERNAL AND UNDER-DRAINAGE

The primary purpose of the embankment internal and under-drainage system is to control leachate. The control of seepage for structural stability reasons is also important, but the leachate control requirements proved the more demanding. The main points of interest in this regard are the special provisions necessary to ensure that the system continues to work following an earthquake, with possible movement on any one of the so-called "active" faults underlying the dam.

The primary components of the leachate and seepage control system may be summarised as follows:

i. Main Embankment Under-drainage

   The embankment under-drainage is physically separated from the tailings under-drainage (not discussed in this paper) and is designed to achieve the following objectives:

   - control of natural groundwater seepage from the sides of the valley

   - collection of percolation of surface water through the embankment (leachate)

   - emergency conduction of tailings under-drainage in the event of earthquake induced rupture.

   The drain is composed of three parts. The first is a "sacrificial", filtered rock drain constructed in the streambed during a low flow period. This base-drain conducted the summer baseflow while allowing access for construction of the main underdrain.

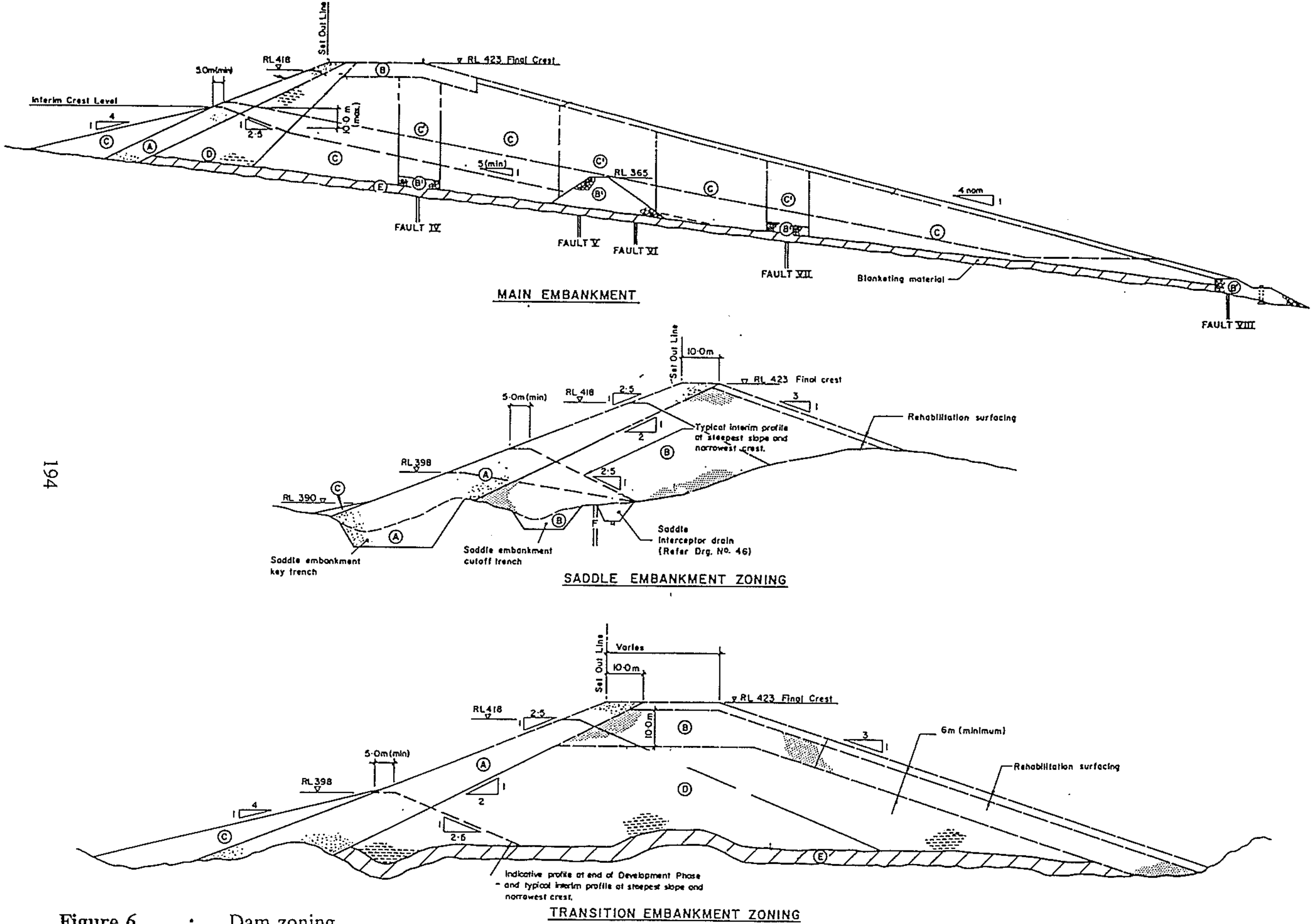

Figure 6 : Dam zoning

Table 1 - Summary of Types and Zones

ZONE	GEOLOGICAL TYPE	STRUCTURAL TYPE	GEOCHEMICAL TYPES	COMMENT
A	Omahia Ash (some Argillic)	Fine grained, structural quality fill compacted to high strength and low permeability.	Low acid producing.	Some low acid producing argillic included to make up sufficient quantities.
B	Omahia Ash (some Argillic)	Structural fill of any grain size.	Low acid producing.	
B′	Omahi	Boulders or rock fill: free draining and high strength fill.	Non acid producing.	For rock buttresses and fault zones.
C	All materials	No structural requirements.	All types including high acid producing material.	Maximum geochemical control requirements by mixing, compaction on encapsulation.
C′	Omahia Ash Argillic	Fine grained for low permeability but otherwise no structural requirements.	Low acid producing.	Placed over faults to control leaching through ruptured/sheared genes. Geochemical control by compaction.
D	Argillic Non-argillic	Structural quality fill.	All types including high acid producing material.	Used only in transition with saddle down where any seepage can be collected by under-drainage.
E	Omahia Ash (some Argillic)	Low permeability but otherwise non-structural.	Low acid producing.	Acts as a sealing blanket on valley base and sides. Low permeability. Some argillic mixed in to make up sufficient quantity.

The main underdrain itself comprises two sections. The lower blanket section is a geofabric-wrapped, filtered rock drain, designed to provide effective base drainage to the waste embankment. The design seepage flow is less than 20 ℓ/s but the actual capacity would exceed 200 ℓ/s.

The upper part of the drain comprises a rock-filled, filter-cloth-protected trench containing the tailings under-drainage conductor pipe. This ABS, unslotted pipe is surrounded by -19mm drainage material to provide protection for the pipe from rock fill stress concentrations and embankment loadings. This part of the drain is designed to pass the tailings under-drainage flow in the event of pipe rupture associated with fault movements.

The capacity would exceed 100 ℓ/s, which is more than sufficient for this purpose, but additional capacity is also available in the lower parts of the under-drain.

In areas where potentially active faults have been delineated, the under-drainage has been "thickened" with increased section drainage zones extending to 20m either side of the fault trace. This thickening is designed to ensure that the under-drain has an adequate connected section after a vertical fault movement of two metres.

ii) Saddle Drain Under-drainage

The saddle dam is constructed entirely of non-acid producing structural fill. The under-drainage, therefore, is designed only as a control of the piezometric surface. The rhyolitic foundation soils are relatively permeable and would tend to act as a drainage blanket, attracting any possible flow from the tailings. Collector trenches and pipes lead this flow back into the tailings area and connect into the main embankment under-drainage. A fault and a cut-off trench act as cut-offs to control possible flow under the dam from the tailings.

A chimney drain throughout the saddle dam length was not considered necessary as the above seepage control methods were found to be sufficient. However, short lengths of vertical chimney drain have been centred on each of two faults which cut through the saddle dam alignment to intercept seepage which might occur in a fault-caused shear zone.

iii) Main Embankment Chimney Drain

A chimney drain is formed within the structural fill zone near the face of the dam. For convenience, this "chimney" drain is formed of slots, 150mm wide and one metre deep, cut into the fill as the earthworks progress.

Collector drains are designed to intercept the water from the slot drains and discharge into the under-drainage system. They are arranged in such a way as to accommodate significant fault movement (i.e. they are enlarged and/or are strategically placed in relation to faults to remain operational with a 2m fault displacement).

iv) Abutment Contact Drains

These are designed to intercept natural seepage from either side of the Main embankment. This seepage is anticipated to occur in two forms:

a) seepage in surface ash which is more permeable than the basement

b) general seepage from groundwater storage in the high ground of adjacent country.

The drains are set along contours at 10m height intervals and are generally 0.6m wide and 1.0m deep. The maximum length of drain feeding to an outlet collector is 150m even allowing for interruption by fault displacement. Coarse drainage material is required in the drains to enable seepage to reach the collector with minimal head on the drain (0.5m).

Collector drains collect the water from the abutment drains and discharge into the under-drainage system. They have been located between faults so as to remain operational. In the event of fault movement which disrupts a longitudinal drain, the seepage can travel to another collector.

CONCLUSION

The main purpose of this paper is to illustrate the differences between tailings retention dams and water retention dams, and to describe some of the special provisions necessary for the former. The Golden Cross Project provides a useful case history in this respect.

The two main factors which influence the structural design of the tailings dam are:

- Availability of construction materials:

- The mining schedule dictates the types and timing of the construction materials.

- Staged construction:

  - The dam is raised over an extended period, giving opportunity for soil strength increases as construction proceeds.

The additional factors which must be included to address the environmental considerations are:

- Geochemistry:

  The dam must be zoned and specifications worded so as to minimise potential for oxidation and seepage within the stack of problematical waste materials.

- Internal drainage:

  Internal drainage, including interceptor drains and under-drains must be designed with excess capacity to control seepage and collect leachates, and in this case, in such a way as to continue operating in the event of fault rupture.

A recent example of a gold mine tailings dam failure resulting in a flood of toxic mud through a living community emphasises the need for specialised design and appropriate risk assessment.

ACKNOWLEDGEMENTS

The design of the Golden Cross Mine was carried out for Cyprus Gold New Zealand Ltd through Davy McKee Pacific Pty Ltd. Permission to publish this paper by the current owners of the Golden Cross Mine, Coeur Gold NZ Ltd, is gratefully acknowledged.

THE EVOLUTION OF TAILINGS DISPOSAL : MACRAES MINE CASE STUDY.

B H Ashton
Macraes Mining Company Limited
Otago

SYNOPSIS

Macraes mine commenced treatment of gold bearing ore in October 1990. Current process plant treatment rates produce 2.1 million tonnes (1.8 million cubic meters) of finely ground tailings to be disposed of annually. Processing of the ore involves a conventional flotation / Carbon-in-Leach (CIL) circuit, producing two types of tailings, flotation tailings and CIL tailings. These tailings differ in their chemical and physical properties. Since the commissioning of the plant until December 1993 these two types of tailings have been stored in separate engineering designed earth fill dams.

Two factors, storage and chemistry, have resulted in changes to the original, tailings disposal plans of October 1990. In terms of storage, the amount of tailings to be impounded has increased due to increased ore reserves, resulting in substantial design modifications to one of the dams. In terms of chemistry, a decision was made to store both the flotation and CIL tailings together in one combined impoundment, moving away from the separate impoundment strategy utilised previously. This paper looks at reasons for the move to a combined tailings dam. It also compares the current flotation tailings embankment design and construction with that of the planned combined tailings embankment.

INTRODUCTION

The Macraes gold mine is situated 60 km north of Dunedin, New Zealand. Currently the Round Hill pit produces all of the plant feed, which consists of gold bearing schist rock. The gold is in the form of a solid solution with pyrite and arsenopyrite minerals. Another pit has been developed and will commence ore production in February 1994, with another seven pits being developed.

Ore is categorised as either oxide ore or sulphide ore, depending on the degree of oxidation of the mineral content. The degree of oxidation is closely related to the depth of weathering, which varies between 5 and 30m within the ore zones. Approximately 86% of the current reserves are classified as sulphide ore, the balance being oxide ore.

The process plant generally treats sulphide ore, except for separate campaigns of oxide ore treatment which occurs after sufficient oxide ore has been stockpiled. These oxide ore campaigns are generally 1-2 months in duration and occur every second year.

Current plant throughput rate is 2.1Mtpa when treating sulphide ore. Flotation tailings are separated at an early stage and are relatively inert. The remaining material is subjected to gold extraction utilising cyanide to dissolve the gold, the remaining residue forming the CIL tailings. The flotation tailings are pumped as a slurry to the flotation tailings dam, and the CIL tailings are pumped to a separate dam upstream of the flotation tailings dam. These embankments are progressively built up to have sufficient freeboard to contain the Probable Maximum Precipitation (PMP) runoff above the rising tailings level. The CIL tailings contain cyanide and are kept separate to minimise the potential effects on the environment, and to reduce the volume of decant water containing cyanide which interferes with the flotation process.

The move to impounding the flotation and CIL tailings together will, in the light of operating practice in North America, reduce the potential effects on the environment. The combined tailings will be disposed of in the current flotation tailings embankment by increasing the impoundment capacity. This will be achieved by raising the design crest height from 118m to 143m.

Figure 1 outlines the site layout showing the location of the current flotation and CIL tailings dams.

FORMATION OF TAILINGS

Flotation tailings

Treatment of sulphide ore involves initially crushing and grinding the ore to approximately 73% passing

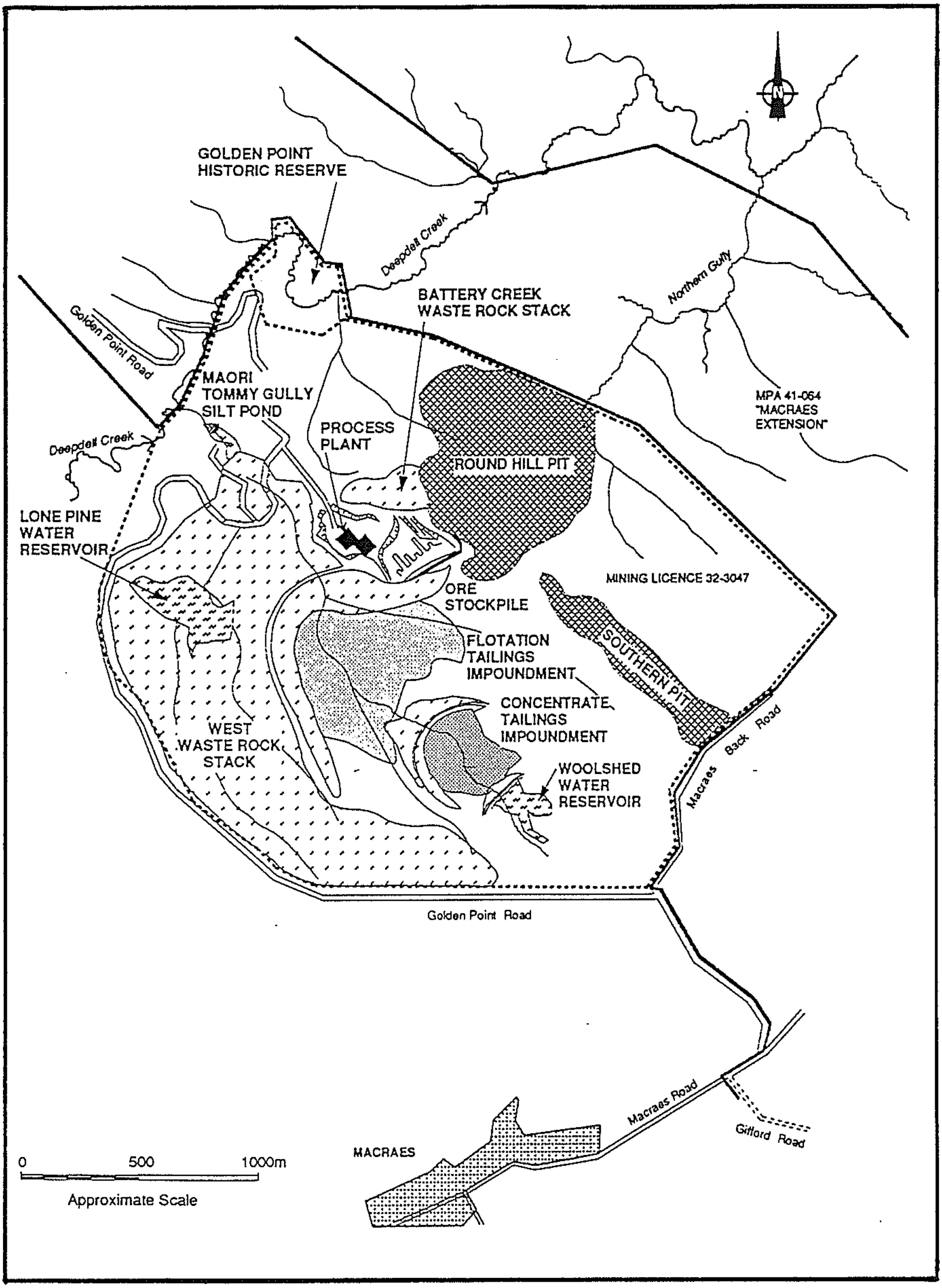

FIGURE 1 : EXISTING SITE LAYOUT

63µm. This material then reports to the flotation circuit. The flotation circuit utilises xanthates to render the gold bearing sulphide minerals hydrophobic causing them to adhere to rising bubbles of air within the flotation columns. Those particles with insufficient sulphide content to make them rise to the top of the column fall to the bottom of the column. This material forms the flotation tailings and is pumped directly to the flotation tailings impoundment. Approximately 90% of the feed is removed from the process at this stage. The SG of the flotation tailings particles is approximately 2.7.

CIL tailings

The portion of the feed that rises to the top of the flotation columns due to its mineral content (ie the remaining 10% of the feed) is termed the concentrate. The concentrate is subjected to further grinding (to 90% passing 19µm) and then gold extraction. This involves dissolving the gold using a cyanide solution in a series of seven large steel tanks. The gold and cyanide form a gold - cyanide solution which precipitates onto carbon pellets. These carbon pellets are removed from the tanks for gold removal, leaving what is termed the CIL tailings. The CIL tailings are pumped as a slurry to the CIL tailings impoundment. The SG of the CIL tailings particles is approximately 3.0.

The CIL tailings are characterised by a high sulphur content (+7%) and the presence of heavy metals; copper, zinc, nickel, lead, cobalt, arsenic, mercury and tungsten. It was recognised in the 1988 *Environmental Impact Assessment*, by BHP Goldmines (New Zealand) Limited that the concentrate tailings had a strong potential to form acid drainage if left in an oxygenated environment.

In summary, the characteristics of the tailings produced from sulphide ore production are:

	Flotation tailings	CIL tailings
Tailings % of feed	90%	10%
Tonnes per annum	1,890,000	210,000
Size	73% passing 63µm	90% passing 19µm
Particle SG	2.7	3.0
Settled density (dry t/m^3)	1.23	1.02
Other components	xanthates	cyanide, heavy metals

Gold production of approximately 3.2 tonnes per annum has essentially no effect on reducing the mass of tailings to dispose of, ie treat 2.1Mtpa of ore resulting in 2.1Mtpa of tailings to dispose of.

CURRENT TAILINGS DISPOSAL STRATEGY.

The current tailings disposal strategy is that which was originally designed and planned for in the 1988 *Environmental Impact Assessment* by BHP Goldmines (New Zealand) Limited. In order to minimise the potential environmental effects of tailings disposal, it was determined that the CIL tailings should be stored in a separate impoundment upstream of the flotation tailings impoundment. These two impoundments are located within Maori Tommy Gully, a tributary to the Deepdell Creek. The flotation tailings would then act as a buffer downstream for any uncollected seepage from the concentrate tailings.

Flotation Tailings Disposal

The flotation tailings are pumped as a slurry to the flotation tailings impoundment. The embankment which forms the impoundment is a zoned, earth fill embankment constructed within Maori Tommy Gully. The embankment is constructed progressively to keep above the level of settled tailings and free water within the impoundment and to provide sufficient freeboard to contain the design Probable Maximum Precipitation storm event. The embankment is constructed of selected mine waste and incorporates an upstream zone of low permeability material and structural fill zones. Tailings under drains, an upstream base collector, and chimney drains are incorporated to collect any seepage. A more detailed description of the design and specifications is given later.

Tailings are discharged from spigots in the pipeline which is layed along the upstream crest of the embankment. The pipeline is moved up as the embankment is built up. A beach of tailings forms as the tailings settle and the decant water and storm water is pumped back to the process plant for reuse. By minimising the free water retained in the impoundment, in conjunction with discharging tailings over different parts of the beach for short periods (eg two weeks) maximum air drying is obtained. This, in addition to drawing off some water from below the tailings via tailings under drains is an attempt to maximise the density of tailings and thus reduce the impoundment capacity needed. To the end of 1993 5,081,000t of flotation tailings have been placed in the impoundment with an average dry density of 1.23 t/m^3.

CIL Tailings disposal

The CIL tailings, being 10% of the total tailings to dispose of, require a much smaller impoundment. The tailings are pumped as a slurry to the CIL tailings impoundment, formed behind the concentrate tailings embankment immediately upstream of the flotation tailings impoundment. The features of the embankment are essentially the same as for the flotation tailings embankment, as is the method of discharge. To the end of 1993 1,105,000t of CIL tailings have been pumped to this impoundment. Due to the finer grind of the particles, the settled dry density to date has averaged 1.02 t/cm. Water reclaimed from the decant pond is treated to destroy the cyanide before being reused in the process plant.

THE CHANGE IN TAILINGS DISPOSAL STRATEGY.

The initial design for the tailings impoundments was based on an initial ore reserve estimate of 6.1Mt. The mass of flotation tailings to be disposed of was estimated at 5.2Mt, with 0.9Mt of CIL tailings. The basis of the 1992 *Macraes Extension Environmental Assessment,* used for the current permitting process for an expanded project was an estimate of 34.9Mt of ore to process. Clearly more tailings storage capacity was needed. Several options were considered for the increased tailings storage capacity required. These included raising the height of the two existing impoundments and constructing similar structures within other gullies. It was clear that the CIL tailings dam would not be able to be increased in height enough to sufficiently increase the impoundment. Thus it was apparent that another similar CIL tailings dam would need to be constructed, with Northern Gully the proposed site.

Following comprehensive re-evaluation of tailings management undertaken in North American mining operations, the option of mixing the flotation and CIL tailings together was investigated. This option is now the preferred way of tailings disposal as it reduces the potential of acid formation. An acid neutralising capacity to maximum potential acidity ratio (ANC/MPA) for the mixed tailings of greater the 4 is consistent with the California Administrative Code, Article 7, which specifies that an ANC/MPA ratio above 3 represents safe non-acid producing material. By moving to a mixed tailings disposal strategy the existing flotation embankment design height could be raised 25m from 118m to 143m high, negating the need to construct another dam in Northern Gully, given the current estimate of ore to process. Permission to modify the design of the flotation tailings embankment was granted under the existing water right conditions. Permits to allow the storage of mixed tailings were sought and are currently being processed.

The system of mixed tailings disposal will be implemented, enhancing security as far as environmental effects are concerned. The environmental advantages of this system as outlined in the 1992 *Macraes Extension Environmental Assessment* are:

- there would be only two tailings impoundments, ie the two existing impoundments in Maori Tommy Gully, as compared with the four previously proposed,
- tailings disposal would be restricted to one catchment,
- a consequential reduction would occur in tailings impoundments surface area,
- reductions to volumes of seepage and mass loads of potential contaminants could also be realised,
- mixed tailings would eliminate the risk of acid seepage from the CIL impoundment,
- the previous contingency measure of relocating CIL tailings into the Round Hill Pit, and its associated bond would no longer be required.

As part of the mixed tailing strategy, the existing CIL tailings will be dredged from the CIL impoundment and blended with the slurried mixed failings as they are slurried to the present flotation tailings impoundment. The resultant empty CIL impoundment will then be used for the storage of mixed tailings along with the present flotation impoundment. The cyanide in the CIL tailings component of the mixed tailings will be destroyed at the plant site before being pumped to the mixed tailings dam.

The move to the mixed tailings disposal plan involves increasing the design crest height of only the present flotation tailings embankment from 490mRL to 515mRL. This involves significant engineering and construction modifications.

There are also financial benefits with the new strategy, brought about mainly by not having to construct new tailings embankments. These savings are most apparent when comparing the incremental cost of increasing the height of the existing flotation tailings embankment with the high capital cost of establishing the foundations and seepage control features of a new embankment. This can also be seen in the impoundment:embankment ratio shown in the table below.

INCREASE IN HEIGHT OF THE MIXED TAILINGS EMBANKMENT.

In order to accommodate the additional future mixed tailings only the existing flotation tailings embankment will be modified. The most obvious modification is the planned increase in height, to be built up gradually ahead of the tailings level. Changes to the design were implemented during July 1993. The embankment height was to be increased regardless of whether separate or mixed tailings disposal was to be used in the future.

A comparison of dimensions and fill quantities of the original embankment design (crest 490mRL) with the increased height embankment design (crest 515mRL) is summarised below. A comparison with Electricity Corporation of New Zealand's Benmore dam on the Waitaki River is included.

supported by two zones of structural fill (Zones B & C). A typical section of the embankment is shown in Figure 2 with the functions of the various zones and specifications of fill material described in appendices A and B.

The design slope on the upstream side is 2H:1V, with the Zone A width being wider than 20m initially, but tapering down to 20m wide for the top half of the embankment. Behind the zone A is a 60m width of Zone B, with Zone C placed behind this. The downstream slope has been constructed to 2H:1V with 5m drainage berms every 20m. This is steeper than shown in the construction plan in order to incorporate a pump station and road downstream of the embankment.

Tailings underdrains were constructed within the gully floor to enable water to be drawn off from under the tailings. Water from these drains is pumped to the

		31-12-93 as built	Original Design	New Design	Benmore Dam
Crest level (above msl)	mRL	470	490	515	363
Max height	m	98	118	143	110
Crest length	m	990	1,160	2,050	823
Impoundment capacity	m^3	5,110,000	12,024,000	25,000,000	2,038,813,000
Zone A fill volume	m^3	795,000	960,000	1,309,000	
Zone B fill volume	m^3	2,846,000	3,509,000	4,892,000	
Zone C fill volume	m^3	6,359,000	7,083,000	7,175,000	
TOTAL FILL	m^3	10,000,000	11,552,000	13,376,000	12,000,000
Impoundment:Embankment ratio	m^3/m^3	0.5	1.0	1.9	169.9

Note that the maximum design height is 33m higher than that of ECNZ's Benmore Dam, with the total fill volume 1.4Mm^3 greater than the 12Mm^3 used at Benmore. It can be seen that the new design embankment is much more efficient in terms of Impoundment:Embankment ratio than the earlier design, however not nearly as efficient as that of the Benmore Dam. This is because the topography is much steeper than that of the Waitaki River bed.

DESCRIPTION OF EXISTING FLOTATION TAILINGS EMBANKMENT DESIGN AND CONSTRUCTION.

Design

The existing embankment has been built using the original contract construction design until July 1993 when the permit for the increased height was issued. The design height of the embankment was 118m.

The embankment is a zoned embankment with an upstream zone of low permeability fill (Zone A)

surface via a submersible pump within a collector well running up the 2H:1V upstream face. An upstream cutoff drain along the foundations beneath the Zone A intercepts seepage through the foundations. This drain consists of coarse gravel wrapped in filter cloth, with slotted pipes included where the fall is less than 1V:100H toward the collector well. Water intercepted in the upstream cutoff drain joins the water from the tailings underdrains to be pumped to the surface.

Within the Zone A, between the upstream cutoff drain and the Zone A / Zone B interface is a chimney drain (Zone D). The purpose of this drain is to intercept seepage through the Zone A fill. The chimney drain consists of a 900mm wide column of sandy gravel. Any seepage through the Zone A fill intercepts this high permeability zone and is collected at the bottom in the chimney drain base collector, of similar construction to the upstream cutoff. The chimney drain slopes at 2H:1V, parallel to the Zone A / Zone B interface. Seepage collected in this manner is directed via pipes under the Zone B and Zone C fill to a collection

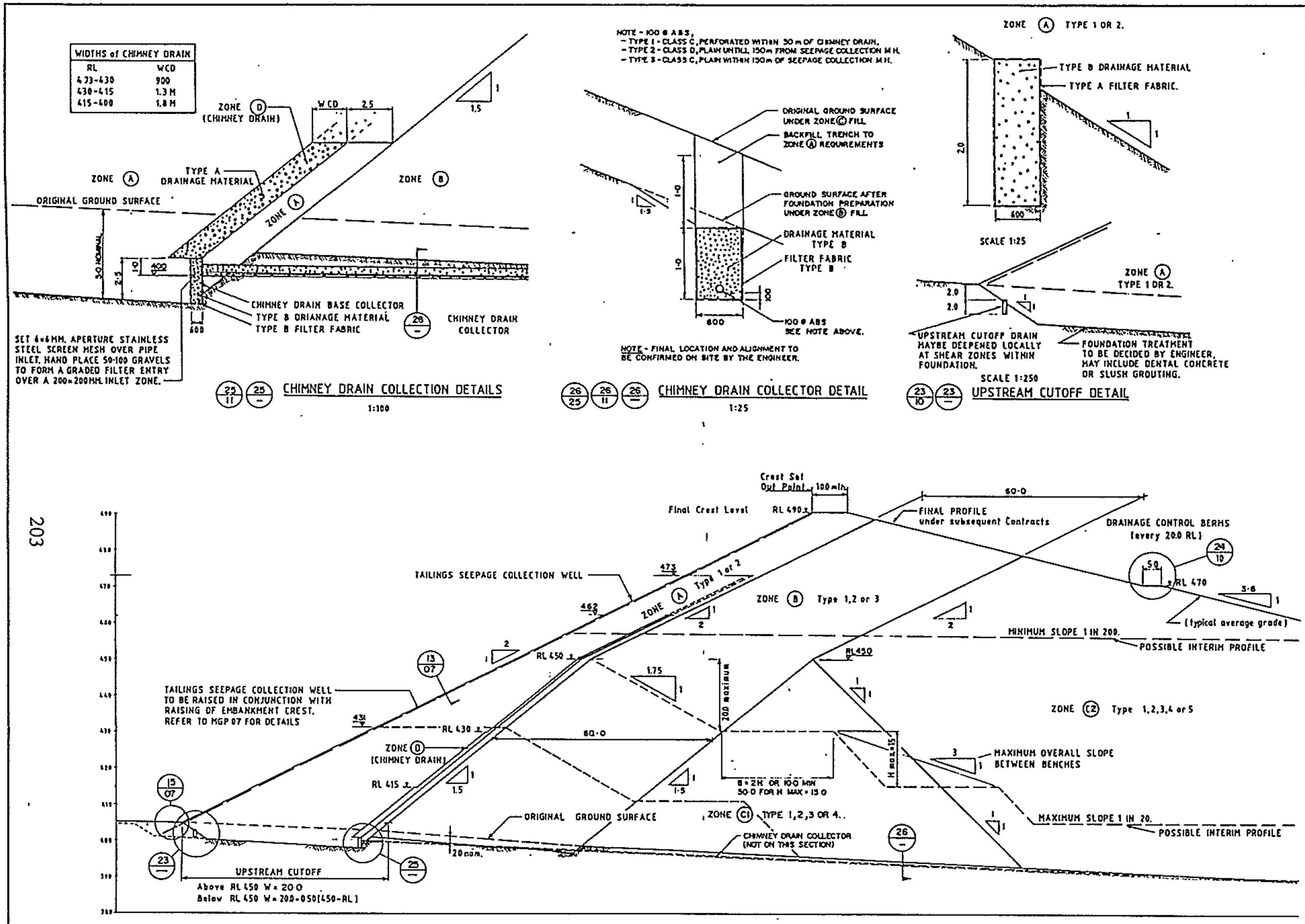

FIGURE 2 : FLOTATION TAILINGS EMBANKMENT - DESIGN TO 490mRL

NOTES

1.) MINIMUM EMBANKMENT CREST RL.

MONTH - DATE	MINIMUM CREST LEVELS
3	RL 462 M.
7	RL 430 M.
10	RL 444 M.
11	RL 447 M.
18	RL 449 M.
31	RL 451 M.

To Be Revised

2.) CHIMNEY DRAIN TO BE CONSTRUCTED TO RL 462. DEPENDING ON PERFORMANCE ENGINEER MAY REQUIRE CONSTRUCTION TO RL 473

3.) DRAINAGE CONTROL BERMS AND LEVELS NOMINAL. TO BE CONFIRMED BY ENGINEER IF REQUIRED IN THIS CONTRACT.

4.) REFER SHEET MGP07 FOR UPSTREAM BERM DETAILS FOR TAILINGS DISCHARGE.

5.) A COFFERDAM TO RL 420 IS REQUIRED IF THE CONTRACTOR WISHES TO PROTECT THE WORKS FROM OVERTOPPING BY A 10 YEAR FLOOD IN THE INITIAL STAGES.

B	ADD RL 431 PROFILE	20.3.90
A	REMOVE TYPE 5 FROM ZONE C1.	FEB'90
No	DETAILS	DATE

REVISIONS

APPROVED FOR ISSUE 9.4.90

GCNZ Consultants

CLIENT: MACRAES MINING COMPANY LIMITED

PROJECT: MACRAES FLAT GOLD PROJECT

TITLE: WASTE ROCK & TAILINGS DISPOSAL
FLOTATION TAILINGS EMBANKMENT CROSS-SECTION & DETAILS

ORIGINAL SCALE: AS SHOWN

DATE	PROJECT No.	SHEET No.
JAN 1990	221-26	MGP 12

REVISION A B

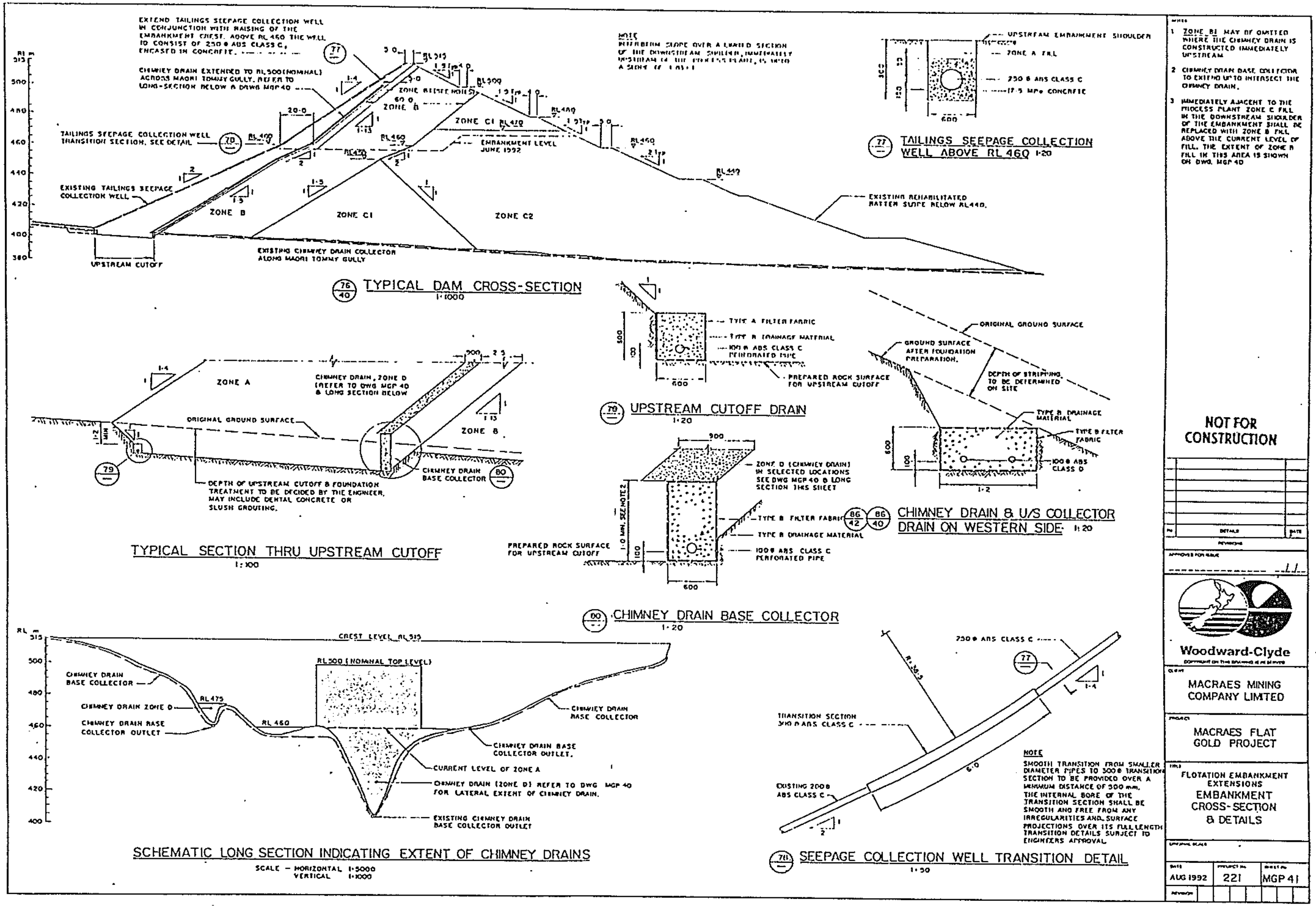

FIGURE 3 : MIXED TAILINGS EMBANKMENT – DESIGN TO 515mRL

manhole (Sump B) where it is returned to the process plant.

Construction

All fill materials, with the exception of Zone D chimney drain, are sourced from mine waste, with the embankment constituting a major mine waste rock disposal site. The foundations below the Zone A footprint are cleared of all loose, weak and broken material by excavation. Brushing and air jetting follow to create a foundation that is tight, sound and free from overhangs and slopes steeper than 0.5H:1V. In the past grout and dental concrete have been used on a limited basis for localised foundation preparation. The foundations are geologically mapped and inspected before placing of fill may commence. Foundation preparation for the Zone B footprint involves stripping all loose material to expose adequately sound rock. Minimal foundation preparation is required within the limits of Zone C fill, other than excavation of Type 5 waste Material (see appendix B).

Material suitable for Zone A fill is placed using scraper or dump trucks and pushed out using a bulldozer or compactor blade. The fill is spread in loose layers not exceeding 200mm. To achieve the tight grading and moisture requirements, when necessary the fill is further broken up with a sheepsfoot compactor after the addition of water. The layer is then compacted to achieve the desired density of 2.15 t/m³ dry density. A loaded Caterpillar 777 or 785 dump truck is usually used for this purpose.

The existing upstream cutoff and chimney drain base collector are extended each time more foundations are exposed and prepared. The chimney drain is constructed within the Zone A fill either by excavating and backfilling or it is built up in strips as the Zone A is placed.

Zone B is dumped and pushed out with bulldozers to form loose layers not exceeding 600mm thickness. Truck rolling is sufficient to bring this zone up to specification. Zone C fill generally needs no further work after dumping and pushing out with a bulldozer apart from shaping of the downstream shoulder.

The downstream shoulder of the embankment is covered with a layer of brown rock followed by topsoil, usually profiled with a bulldozer. Seeding and fertilising on the 2H:1V slope is performed by a contractor with a 4WD tractor.

Monitoring

Inspections and monitoring have been carried out prior to and during the filling of the embankment. Inspections begin with the geological examination and mapping of the Zone A footprint foundations. The company has a full time employee to inspect and test the fill material and drainage components. The fill is tested to ensure that it meets the required grading, placed density, water content and permeability specifications. A nuclear density gauge, New Zealand Standard water replacement and sieving tests and constant head permeability tests are used for this purpose. All pipe work is pressure tested before backfilling. The following table summarises specifications for Zone A and Zone B material.

SPECIFICATION	ZONE A	ZONE B
Dry density	2.15t/m³	>2.10 t/m³
Water content	6.5%	2%
Grading	D_{100}<0.15m	D_{100}<0.50m
Permeability	<10^{-7}m/s	N/A
Layer thickness	0.2m	0.6m

Visual inspections using SEED guidelines are conducted of the embankment crest, shoulders and abutments. Examination and measurement of the chimney drain and collector well seepage is carried out to assess the seepage characteristics. Regular inspection of the seepage return pumps and the decant return pumps are conducted to ensure these are functioning properly.

Located within the embankment are fourteen pneumatic piezometers to allow the measurement of pore pressures within different zones of the embankment, allowing the effectiveness of the various seepage control measures to be assessed.

Ongoing precise three dimensional deformation surveys of the down stream shoulder are conducted using an EDM total station and permanent settlement markers. Maximum total (downstream and downwards) of 198mm has been detected above the deepest part of the gully to date. Regular surveys of the tailings deposition profile and free water allow the settled density of the tailings to be monitored, and provide a check on the freeboard capacity of the embankment.

Research

Macraes Mining Company Limited have funded research by the University of Auckland. Research to date has included tailings density and consolidation modelling and the effectiveness of tailings underdrains. The density and consolidation test results are in close agreement with parameters measured on site using tailings profile surveys. The tests to date indicate the effectiveness of tailings underdrains to be limited in improving consolidation. Further work will be undertaken in the future for comparison to actual values.

DESIGN AND CONSTRUCTION CHANGES REQUIRED TO INCREASE EMBANKMENT HEIGHT.

The construction of the flotation tailings embankment was completed to approximately 460mRL at the time the design modifications were implemented. Thus, the design to 515mRL uses the current embankment to 460m RL as a base. The new design was constrained by the location of the mine office building and process plant. A typical cross section through the new design to 515m RL is shown in figure 3

Features of the new design include:

- upstream shoulder steepened to 1.4H:1V above 460mRL,
- downstream shoulder steepened to an overall slope of approximately 2H:1V above 440mRL, with interberm slopes of 1.8H:1V,
- decreasing the width of zone A,
- crest width reduced to 5m,
- termination of the chimney drains, except at points of greatest curvature and areas of greatest potential differential settlement. ie directly above the gullies,
- berms on the downstream shoulder to collect and control run off,
- no spillway (as per the original design),
- more tailings underdrains to ease the duty on the existing collector well and additional chimney drain and upstream cutoff drain outlets.

Steepened shoulders

Using the as built embankment as the base of the new design, steepening the shoulders was the only way of gaining the additional height needed. The upstream face has been steepened to 1.4H:1V. This change necessitated the installation of a long radius transition bend in the upstream collector well to enable the collector pump to be withdrawn with ease. The downstream interberm slope is 1.8H:1V to ensure clearance from the office buildings and process plant. The steepened downstream shoulder necessitates a different method of profiling the brown rock and topsoil. On the flatter original design slopes a bulldozer was used to profile the brown rock and topsoil. This was done in approximately 10m vertical lifts. The steeper slopes necessitate an excavator sitting on the downstream crest profiling the brown rock and bucketing down and trimming to shape the topsoil. In this manner the lifts are more usually 3-4m, limited by the reach of the excavator. Seeding and fertilising is now carried out by spraying with hydro mulch.

Decreasing the width of Zone A

The original design was for a 20m width of Zone A. This has been changed so that the width tapers off from 20m at 460mRL to 5m at the 515mRL crest level. The formation of a beach against the upstream wall of the Zone A means that the free decant water and run off are not in contact with the Zone A fill itself. In fact, due the size segregation of particles against the upstream shoulder, the tailings in this vicinity are quite permeable. This, combined with the decreasing overburden height of tailings at higher levels works to reduce the hydraulic gradient within the Zone A fill. Hence the Zone A width could be decreased. The Zone A is still required as under PMP storm conditions run off water would pond against the upstream face, covering the beach. Also, under design earthquakes there is a small possibility that the tailings may liquefy, hence the need for the impermeable Zone A.

Reduced crest width

The reduced crest width, bought about by the increased height does not compromise the stability of the structure. The analysis for static loading, using Spencers solution method, indicate acceptable factors of safety.

The design basis earthquake (DBE) and the maximum credible earthquake (MCE) are estimated to have maximum ground accelerations of 0.15g and 0.34g respectively. Under these conditions some minor yielding is predicted to occur. The estimated permanent displacements are small and would not compromise the ability of the dam to retain the tailings.

Termination of chimney drains

The chimney drain is terminated at 460mRL except over the deepest parts of the embankment. As for the reduced width of Zone A material, ie reduced hydraulic gradients, the principal need for chimney drains are under PMP flood and large earthquake conditions. With a PMP flood, run off water will be banked up against the upstream crest. With a large earthquake, any cracks that do form within the embankment are expected to be above the shoulders of the gullies. Hence above the gullies the chimney drain will be continued. Figure 3 shows the position of chimney drains.

In places where the chimney drain is terminated, a new zone is to be constructed between the Zone A and Zone B. This zone, Zone B1 is a transition zone to prevent differential movement between the Zone A and Zone B. Where the chimney drain is not required the chimney drain base collector will still be constructed in order to intercept foundation seepage. There are two new outlets for the chimney drain seepage in addition to the one already in place. These are gravity drains and reduce the load on the existing drain and return system.

Tailings underdrains
Further tailings underdrains have been constructed to assist in dewatering the mixed tailings. These drains have been constructed around contours of the impoundment and drain to a collection manhole.

Monitoring

The current methods of monitoring the performance of the embankment will be continued as the embankment is built up to ensure the integrity of the structure and to form the basis of any design modifications.

The monitoring of the fill material and construction of the mixed tailings embankment will be ongoing to ensure compliance with the design specifications.

A number of additional pneumatic piezometers will be installed within the embankment as the target locations are constructed. Ongoing monitoring and quantification of seepage and pore pressures from the existing and additional collection points will allow the continuing performance of the embankment to be assessed. The exact height that the Zone D chimney drain can be terminated, and the need for further tailings underdrains can be assessed on the basis of the collected information.

The surveying of the tailings profile will continue to allow the consolidation characteristics to be further determined. Ongoing research will be conducted to allow the effectiveness of consolidation to be assessed. Precise deformation surveys of the down stream shoulder will be also be continued.

CONCLUSIONS

The move to mixing the flotation and CIL tailings for disposal will reduce the risk of acid drainage.

In order to have sufficient storage capacity for the future planned mixed tailings, only the existing flotation tailings embankment design height needs to be increased from 118m to 143m. The design fill volume for the embankment is 13.4Mm^3, with an impoundment capacity of 25Mm^3.

Continuation of the fill testing and monitoring of the seepage characteristics and embankment deformation allows the overall integrity of the embankment to be assessed and any design modifications to be implemented.

ACKNOWLEDGMENTS

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APPENDIX A

Zone	Description of Different Embankment Zones
A	The primary function of this zone is to limit seepage. It forms the upstream structural zone of the CIL, flotation and planned mixed tailings embankments. The zone provides sufficient strength to prevent the likelihood of instability, particularly when subject to the design seismic loads. Zone A is constructed primarily from Type 1 material (see Appendix B) which will in almost all cases require conditioning. Blending with up to 25% by volume of Type 2 material shall be permitted. Heavy compaction is vital to achieve the standards as set out in the specifications in terms of permeability, dry density, water content and particle size grading.
B	Zone B forms the bulk of the structural fill portions of the tailings embankments. Its main function is to contribute strength so as to prevent the likelihood of instability. The extent of this zone for the tailings embankments was based on achieving minimum-fill structures during the early stages of mining, consideration of likely fill profiles compatible with the mining programme and consideration of long term stability, particularly when the embankments are subject to design seismic loads. The zone can consist of Type 1, 2 and 3 waste material, and requires compaction to meet the standards for dry density and maximum particle size.
C	Zone C forms the bulk of the existing flotation tailings embankment / future mixed tailings embankment. It provides buttressing to zone B and minimum profiles must be maintained to achieve this. Zone C provides for bulk disposal of mine waste material. The zone can include Type 1, 2, 3 or 4 waste material. Type 5 waste material may be included in limited quantities but only at certain locations. For this zone a limitation on maximum lift heights is imposed.
D	Zone D is a chimney drain and its primary function is to intercept seepage and to limit the development of pore pressures in the down stream shoulders of the embankments. The Zone is constructed primarily from Type A material (sandy gravel) except for the base collector which is Type B drainage material (gravel).

APPENDIX B

Waste material	Definition of Waste Material Types
Type 1	Consists of completely, highly or moderately weathered schist rock which after placement, conditioning and compaction as specified is capable of forming a strong, dense, low permeability fill. Type 1 is obtained from the mine as well as foundation preparation and haul road construction. Schist rock fulfilling the criteria for Type 1 waste material is found at depths of up to 10m below the existing ground surface at the mine site and up to about 3m below existing ground surface elsewhere.
Type 2	Consists of loess, colluvium, solifluction materials or combinations thereof which after blending with Type 1 waste material, placement, conditioning and compaction as specified is capable of forming a strong, dense, low permeability fill. Type 2 is obtained in limited quantities from the mine as well as from embankment foundation preparation and from other borrow areas (eg haul roads). Deposits of loess; colluvium and solifluction material are often intermixed and are found in deposits up to 3m deep overlying weathered schist.
Type 3	Consists of slightly weathered or fresh schist rock which after placement and compaction as specified constitutes a strong, dense fill. Type 3 is primarily obtained from the mine, but some small quantities may be derived from embankment foundation preparation.
Type 4	Comprises all mine waste and materials from embankment foundation preparation and other works not included in the definitions of Type 1, 2, 3 or 5 material.
Type 5	Consists of reject material which is generally derived from excavations which are required to expose the foundations and comprises weak or organic material, largely excavated from the base of natural drainage channels and the like.

THE USE OF A GROUT CURTAIN AS A DISPERSION BARRIER TO ASSIST GROUNDWATER MONITORING

W J Russell
Woodward-Clyde (NZ) Ltd
Auckland

SYNOPSIS

In this presentation I describe the concepts behind and the construction of a grout curtain placed downstream of the tailings impoundment at the Macraes Flat goldmine. The purpose of the grout curtain is to provide a dispersion barrier for groundwater seepage. With dispersion of the groundwater flow, monitoring for seepage can be undertaken with confidence.

THE SETTING

Tailings at the Macraes Flat gold mine are sub-aerially deposited in a tailings impoundment. The tailings impoundment has been formed by damming Maori Tommy Gully, a tributary of Deepdell creek.

Figure 1 shows the design features of the main tailings dam. This dam is an engineered structure designed and constructed to high standards in accordance to the recommendations of the International Commission on Large Dams (Ref. 1)

The dam has been constructed across Maori-Tommy Gully using waste rock obtained from the mine pit. The waste rock consists of variably weathered schist.

The dam is a zoned structure. This is shown on Figure 1.

- Zone A forms the upstream structural zone. Its primary function is to limit seepage.
- Zone B forms the bulk of the structural fill
- Zone C provides buttressing to Zone B
- Zone D is a chimney drain the primary function of which is to intercept seepage through the dam.

The dam incorporates a comprehensive seepage interception and collection system. Included are:

- an upstream cutoff
- underdrains laid in the stream channels
- upstream cutoff drains
- seepage recovery well and submersible sump to remove seepage from the cutoff and underdrains
- a gravity pipeline to collect and remove seepage from the chimney drain
- a toe sump to collect chimney drain discharge for return to the tailings impoundment
- an across-gully concrete wall to act as a downstream cutoff. This is used as the downstream side of the toe sump.

It is the effectiveness of this seepage interception and collection system which is being evaluated by groundwater monitoring.

POTENTIAL FOR SEEPAGE

Tailings pore fluids and decant are in contact with natural ground beneath the impoundment. The natural ground comprises schists. These schists have no primary porosity and permeability except when weathered. However, defects in the rocks, mainly joints and fractures, provide secondary porosity and permeability. These defects are not continuous and hence groundwater pathways are limited. This is inferred from the existence of steep groundwater gradients at the site. These steep groundwater gradients also indicate low rockmass permeability.

The permeability data obtained from the site investigations shows that permeabilities are greater in the relaxed, weathered zone but reduce with depth. The thickness of the relaxed weathered zone varies but is typically up to about 20 m. Greater groundwater movement is possible, therefore, through the relaxed weathered zone.

In constructing the dam cutoff, the ground was cleaned to competent rock. But the cutoff was not able to be taken through the entire relaxed zone due to the depth of weathering. Therefore, there is a potential, albeit limited, for a small amount of tailings fluid to seep beneath the dam and bypass the interception and collection systems.

The potential seepage pathways for seepage from the Tailings impoundment are shown on Figure 2. The principal point of exit for groundwater and tailings fluid seepage will be at the toe of the main impoundment. It is in this location that the collection sump has been constructed.

A strong potential for groundwater movement from the ridges towards the gully is indicated by the groundwater levels as measured in monitoring wells. The groundwater gradients at the toe of the main impoundment, as inferred from these water levels, are shown on Figure 3. From Figure 3 it can be seen that the movement of groundwater and the seepage of tailings fluids beneath the impoundment will be constrained by lateral groundwater movement.

The inferred groundwater gradients on Figure 3 indicate the general direction of groundwater movement. However, seepage in discrete fractures may follow preferential pathways which cross inferred groundwater gradients.

The successful placement of monitoring wells to intercept such preferred pathways would require extensive and comprehensive defects mapping as well as drilling, water level measurement, permeability testing and interference testing. Even with such investigations, there would still be a risk that not all discrete pathways could be intercepted.

SOLUTION

To provide the maximum opportunity for detecting seepage bypassing the collection systems, a grout curtain has been installed downstream from the toe sump.

Figure 4 shows the location of the grout curtain in relation to the toe of the embankment, the mine access road, the silt pond, the compliance point and Deepdell creek.

Figure 5 shows the grout curtain design in relation to topography, water table depth, gully level and the level of Deepdell creek.

The purpose of the grout curtain is not to DAM groundwater movement but to restrict movement along preferential pathways in order to force groundwater flow through finer and more extensive defects. In this manner potential seepage will be dispersed and hence the opportunity to detect seepage will be substantially increased.

IMPLEMENTATION

The grout curtain has been constructed across the floor of Maori Tommy gully, up the eastern slope to the ridge and up the western slope to a comparable elevation to the eastern ridge. Prior to its construction a trial was carried out along the eastern portion of the grout curtain alignment. The locations of the drill holes are shown on Figure 6.

SITE PREPARATION

The ground along the grout curtain alignment was stripped of topsoil and subsoil to expose the bedrock. On the gully floor, fill and alluvium was removed in part. A portion of fill was retained to provide a drilling platform to parts of the gully.

HOLE SPACING

A primary hole spacing of 6m was adopted above a level of 360 m RL. This included sections of the alignment along the eastern ridge,partly down the eastern face and also down the western face. Below this level the primary spacing was 5m.

Secondary holes bisected the primary holes leaving a spacing of 3m or 2.5m. Where required, tertiary and in one section quaternary holes were drilled. This reduced the hole spacing down to 1.25m and ultimately down to 0.6m in places: the distance measured along the ground.

HOLE DEPTH

Along the eastern ridge and down to the gully floor the holes were drilled to a depth of 30m perpendicular to the average ground profile. Across the gully floor, the depth of the holes was 25m. That depth corresponded to the level of Deepdell Creek, the ultimate discharge level for groundwater movement down Maori Tommy gully. The holes down the western face were drilled to 40 m vertical with the holes at the highest elevation being drilled to 50m vertical.

DRILLING

Two rotary percussion rigs were used to drill the grout holes. One was a Gardner-Denver Hydratrac rig. This rig was used for the holes on the holes on the top of the east ridge, within the gully and also for the majority of the holes down the western face. The second rig was an ROC 601 Airtrack supplied by a 1100 cfm compressor. The airtrack worked the eastern face supported by a winch.

Drilling was initiated at 200mm diameter to enable standpipes to be installed. The purpose of the standpipes was to prevent loose material from falling into the holes and in the base of the gully, to allow the packers to set in competent material.

The grout holes were drilled at 75mm diameter. Air flushing rather than water flushing was used to avoid a build up of slimes in the holes. Where holes were drilled into more permeable ground below the water table water was pumped from the holes and flushed the wells.

Drilling records were kept for each drill hole. The records included the colour of the drill cuttings, rock hardness, depths at which water was encountered, indication of the groundwater recharge to the holes and any other comments the drillers noted.

FLUSHING

After each hole was drilled and prior to grouting, the holes were flushed out by placing an alkathene pipe to the base of each hole and flushing out until the stream became clear.

TAKE TESTING

Packer testing to evaluate the permeability of the rock and to assess grout take was routinely carried out during the grout trial. During construction of the full grout curtain, water take testing was only undertaken to assess the permeability of a small section of the fill in the valley floor.

GROUTING

The grout consisted of water, cement and bentonite plus an expander for holes which were located within the gully and where grouting was required to extend to the surface. The mixing proportions were as follows:

- water/cement — varied from 2:1 to 0.8:1 by volume. A rapid hardening cement was used.
- bentonite — 1% by weight of cement
- Darexpand — approximately 80g per bag of cement.

The grout was mixed on site.

Usually the first mix was a four bag mix at a water/cement ratio of 2:1. A second mix would be prepared and injected if required. If a second mix was used, then additional grout would be mixed at a ratio of 1:1 until the closure acceptance criteria were met. The closure acceptance criteria were:

- 0.03 m^3 in 20 minutes where grout pressure is less than 300 kpa or
- 0.03 m^3 in 15 minutes where grout pressure is greater than 300 kpa

The batch of grout was tremmied into the hole using an alkathene pipe inserted to the bottom of each stage or through the packer hose. This ensured that at least the volume of grout required to fill the hole was injected into each hole. The grouting pressure was kept at 100 kpa for the first few minutes after which the pressure was gradually increased to 50 times the packer depth in meters.

As grouting proceeded a hose was fed into the hole until it reached the top of the packer and water circulated to maintain the space above the packer clear of grout either from the initial filling or from grout bypassing the packer via a defect. Similarly holes on either side of the hole being grouted were flushed to check for connections between the holes.

Detailed grouting records were maintained. These records included a summary of time, water/cement ratios, grouting volumes and pressures, bags of cement and average cement takes (kg/m) for each individual stage of grouting.

TOPPING UP OF HOLES

At the end of each day the grouted holes were topped up with a thick grout mix. This was repeated until the grout remained at the surface. The need for repeat top ups was attributed to consolidation of the grout. The use of Darexpand significantly reduced the amount of consolidation although even with this additive a limited amount of slump occurred.

MONITORING

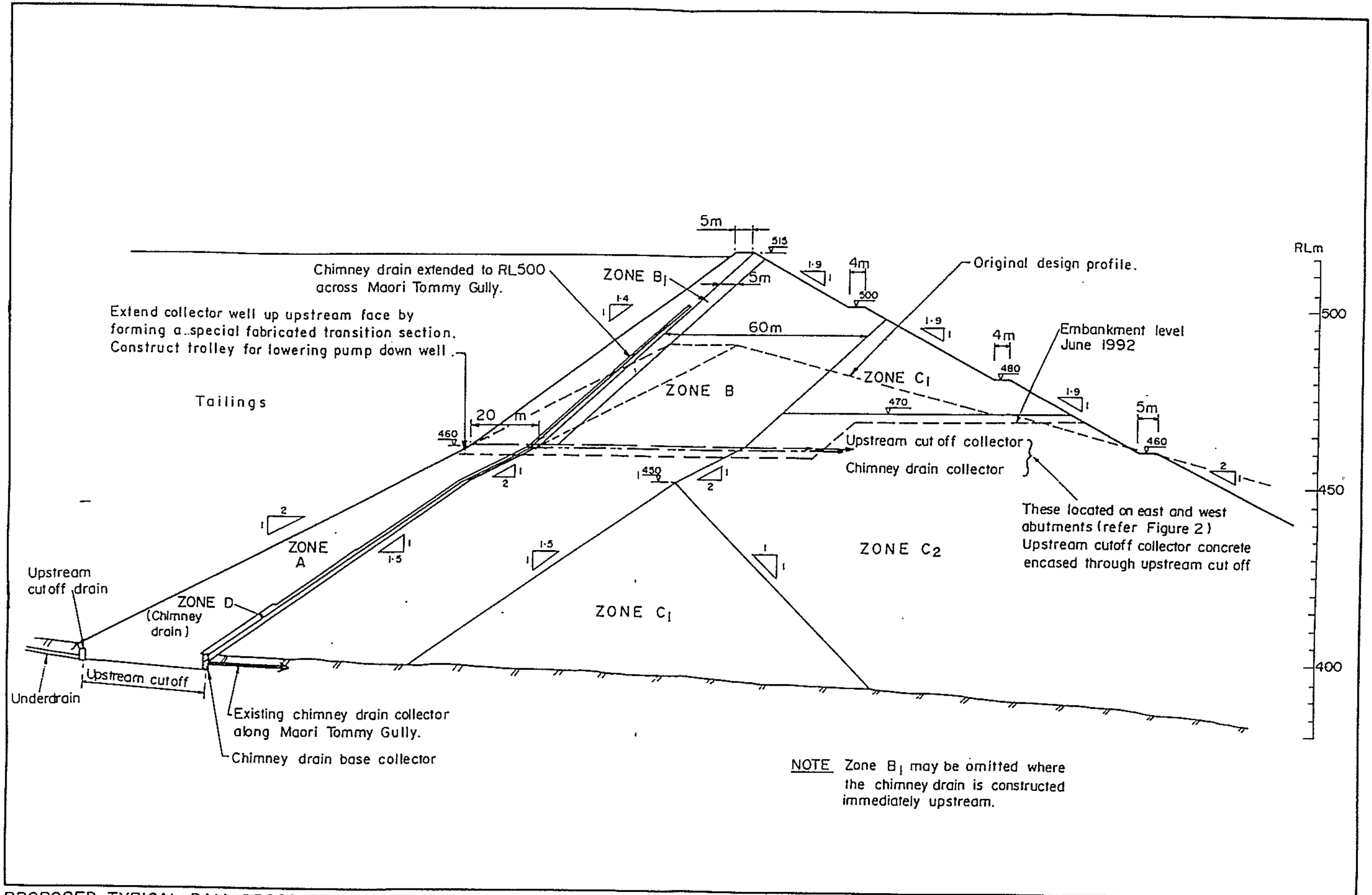
Following completion of the grout curtain, groundwater monitoring wells were installed downstream of the grout curtain. The locations of these wells are shown on Figure 6. Monitoring of these wells is undertaken to detect seepage of tailings fluids into the groundwater system.

Compliance of water permit standards is not required at these detection wells but is required at a further line of wells a short distance from the confluence of Maori Tommy gully with Deepdell creek.

With the detection point some distance from the compliance point, the low groundwater velocities will ensure that there is sufficient time to evaluate the risk to the environment from any detected seepage and to initiate contingency action if required.

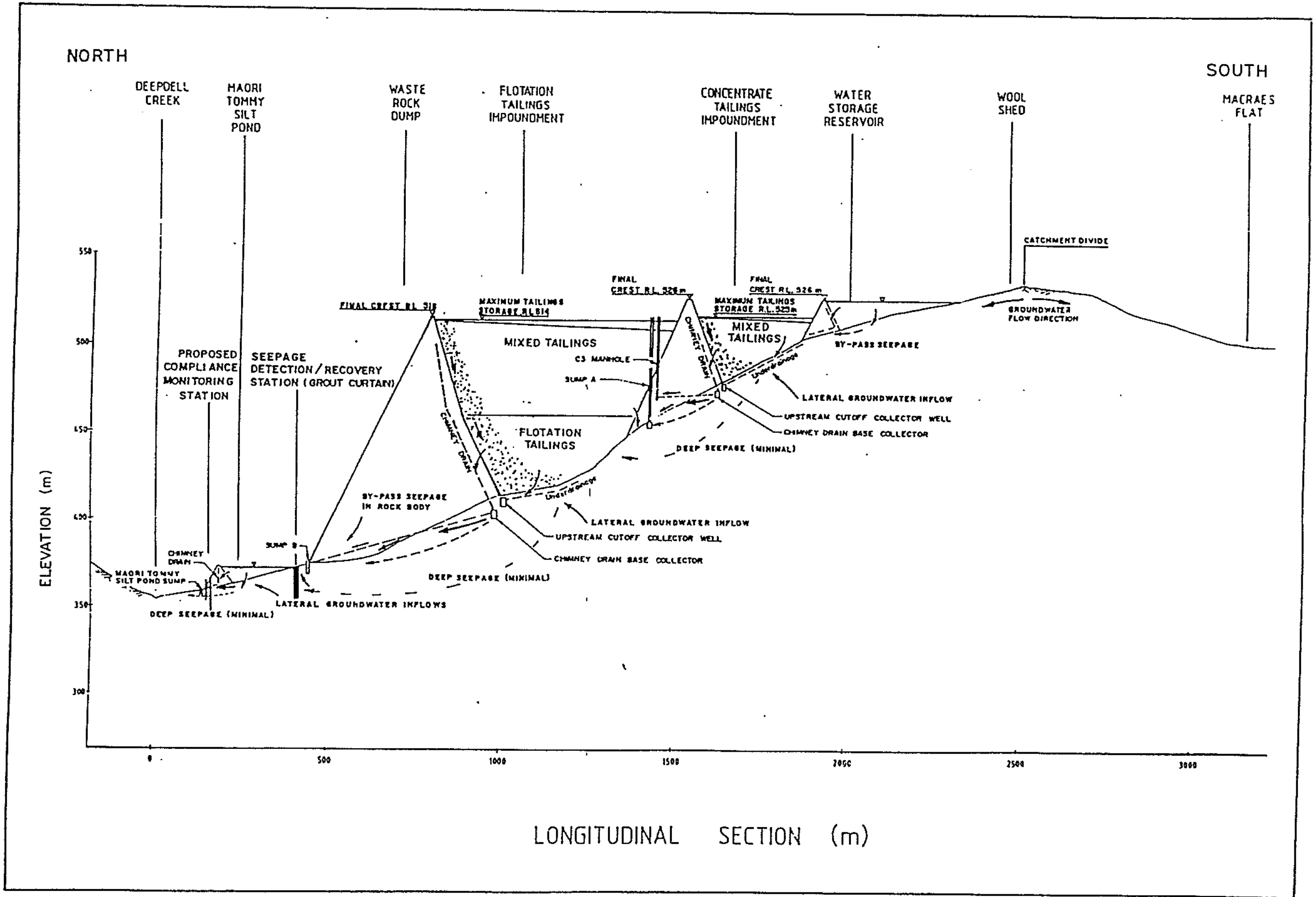
REFERENCES

Ref.1 Matuschka, T., 1993: Evidence Presented at Resource Hearing before Waitakere District Council and Otago Regional Council.



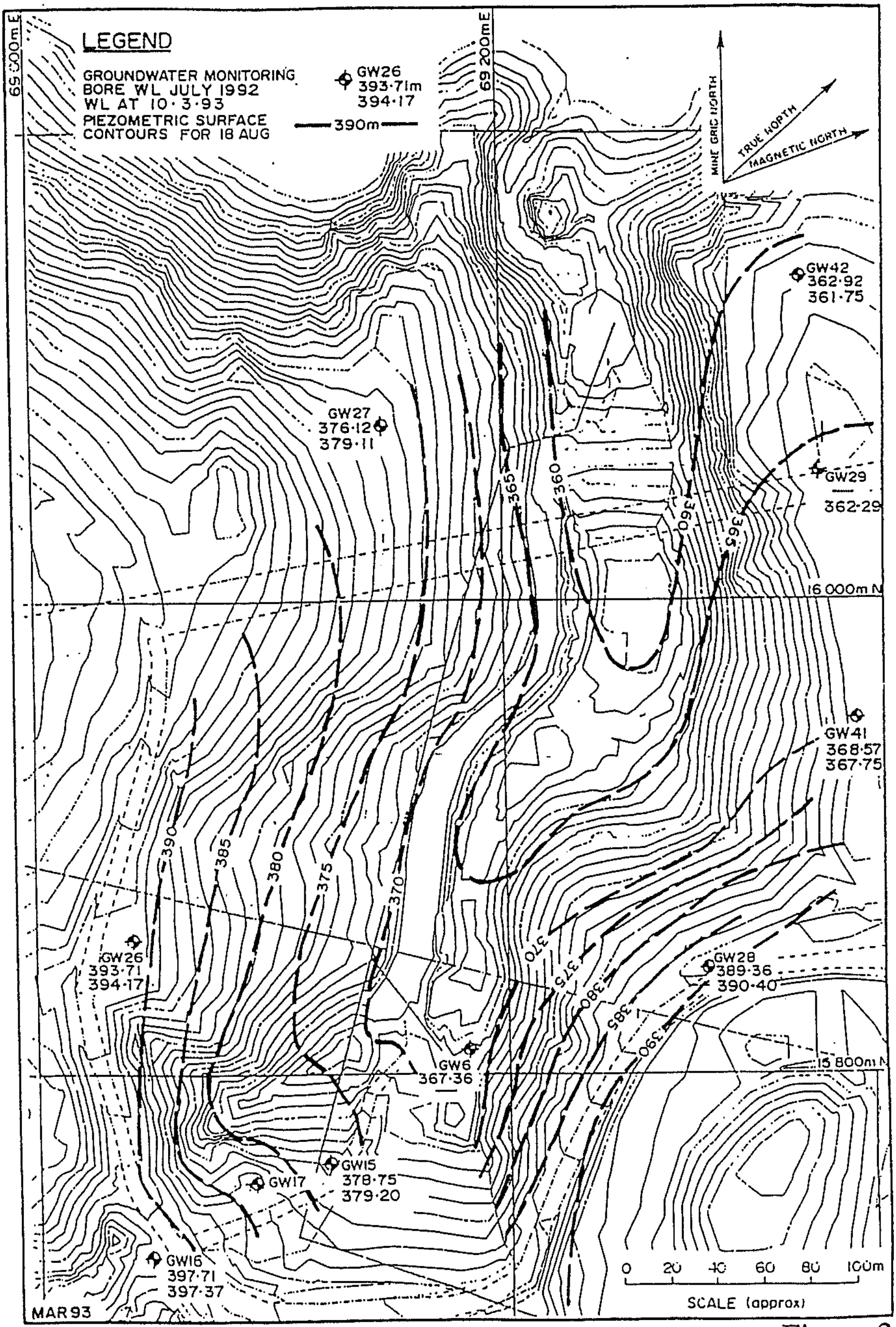
PROPOSED TYPICAL DAM CROSS SECTION
FLOTATION TAILINGS DAM

Figure 1

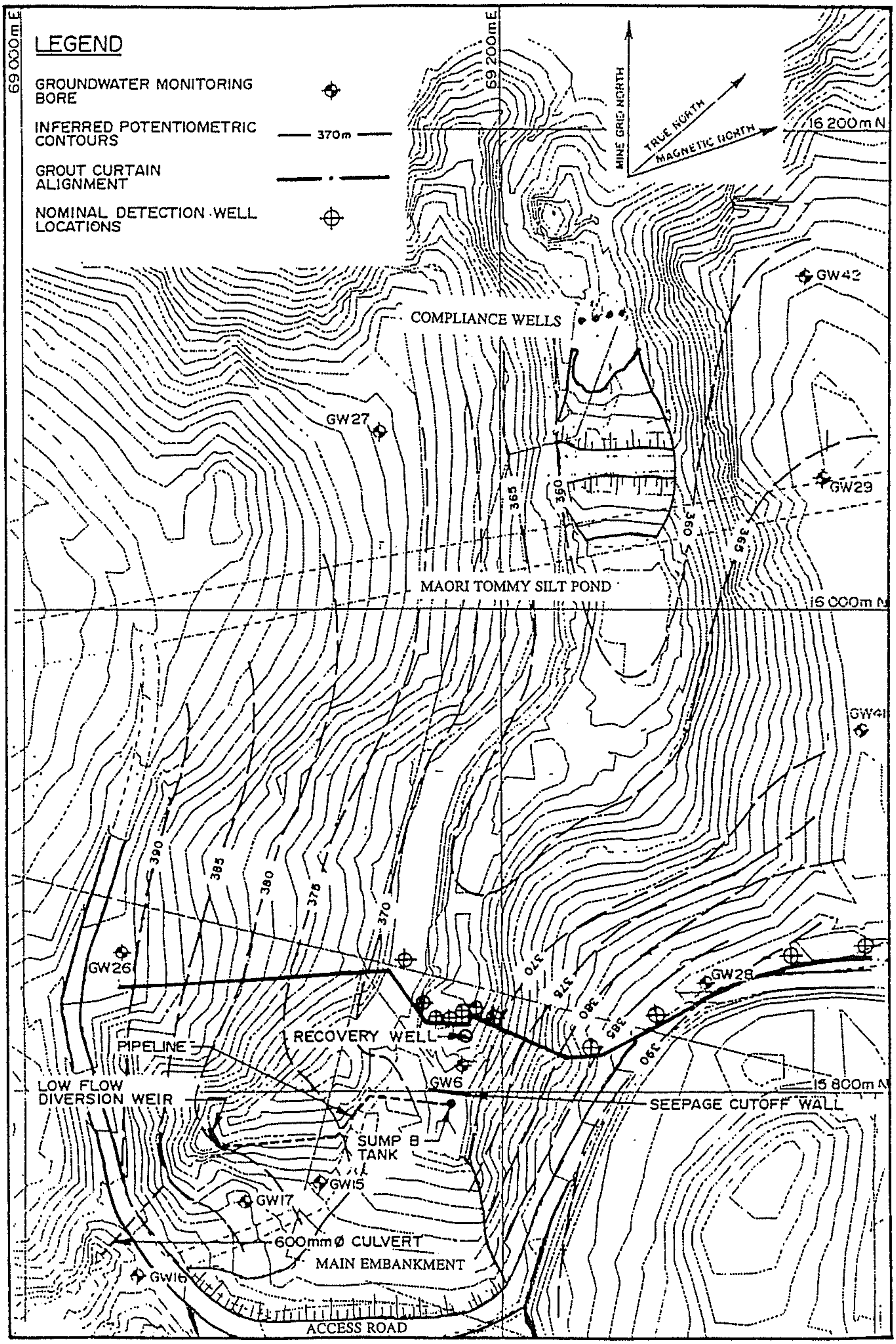


MAORI TOMMY GULLY CONCEPTUAL DOWN-VALLEY SEEPAGE FLOW PATHS

Figure 2



POTENTIOMETRIC SURFACE - MAORI TOMMY GULLY Figure 3



LOCATION OF GROUT CURTAIN & MONITORING WELLS Figure 4

SUMMARY EXTENT OF GROUTING		
LOCATION	DEPTH OF GROUT CURTAIN	
	Top	Bottom
Gully	Ground surface up to RL 390	RL345 where ground surface below RL380
East Abutment	0-8m below ground surface	30m below ground surface
West Abutment	0-12m below ground surface	35m below ground surface

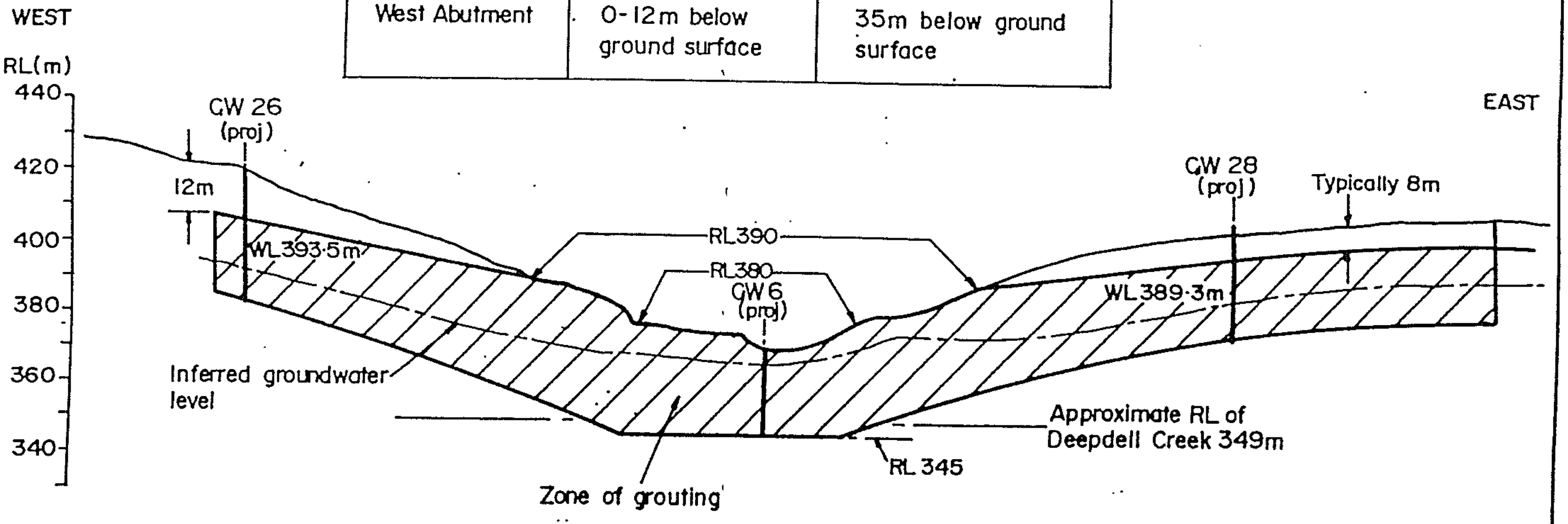
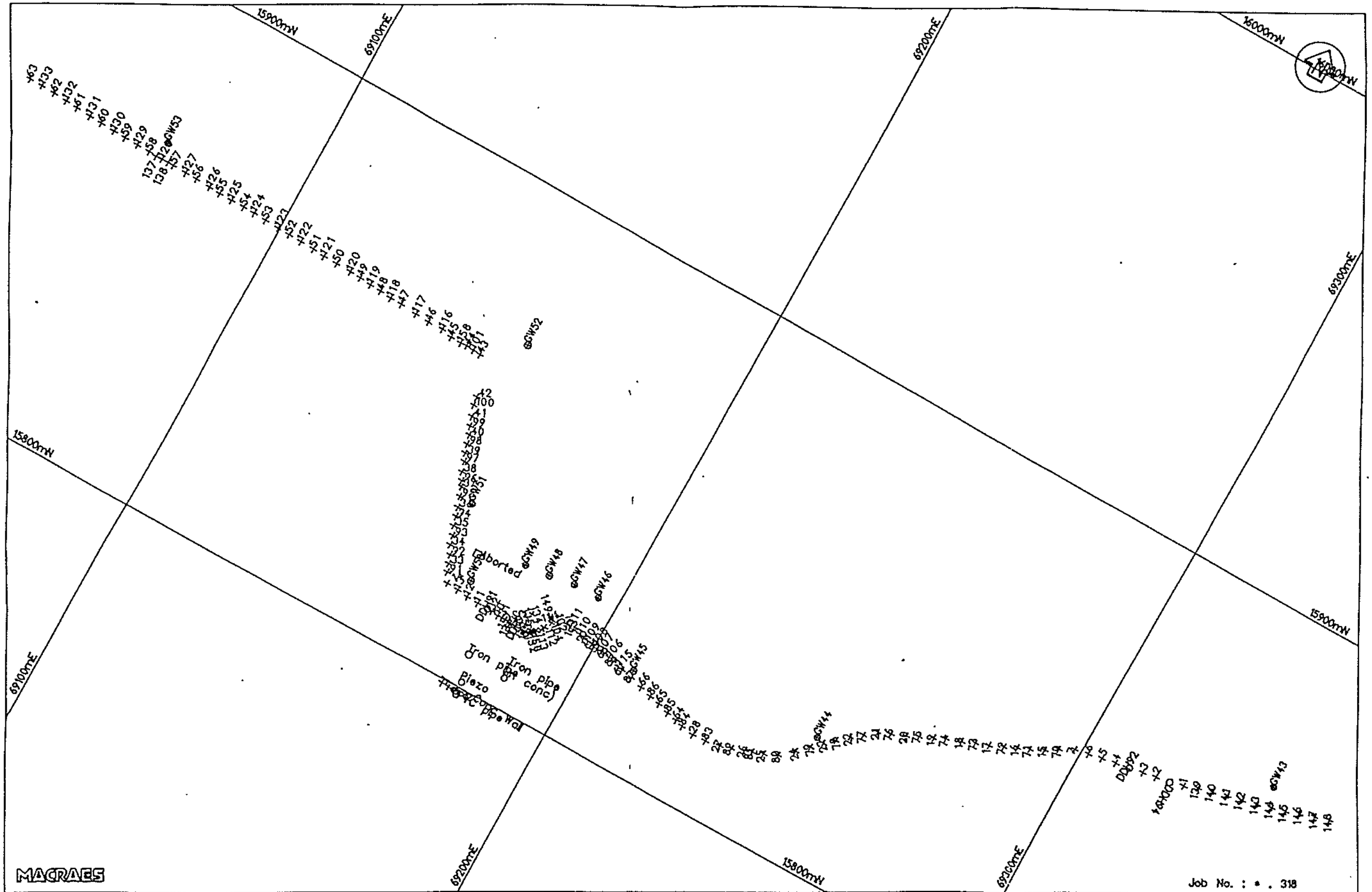


Figure 5



Scale 1:750	MAORI TOMMY – GROUT CURTAIN	Overall Layout	Drawing No. GC-1	Plotted on the 8 APR 1993 Surveyed by: D. C. Wood

Figure 6