PIT SLOPE MANUAL

chapter 4

GROUNDWATER

This chapter has been prepared as part of the

PIT SLOPE PROJECT

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THE PIT SLOPE MANUAL

The Pit Slope Manual consists of ten chapters, published separately. Most chapters have supplements, also published separately. The ten chapters are:

1. Summary
2. Structural Geology
3. Mechanical Properties
4. Groundwater
5. Design
6. Mechanical Support
7. Perimeter Blasting
8. Monitoring
9. Waste Embankments
10. Environmental Planning

The chapters and supplements can be obtained from the Publications Distribution Office, CANMET, Energy, Mines and Resources Canada, 555 Booth Street, Ottawa, Ontario, K1A 0G1, Canada.

Reference to this chapter should be quoted as follows:

Sharp, J.C., Ley, G.M.M., and Sage, R. Pit Slope Manual Chapter 4 - Groundwater; CANMET (Canada Centre for Mineral and Energy Technology, formerly Mines Branch, Energy, Mines and Resources Canada), CANMET REPORT 77-13; 240 p.; November 1977.

FOREWORD

Open pit mining accounts for some 70% of Canada's ore production. With the expansion of coal and tar sands operations, open pit mining will continue to increase in importance to the mineral industry. Recognizing this, CANMET embarked on a major project to produce the Pit Slope Manual, which is expected to bring substantial benefits in mining efficiency through improved slope design.

Strong interest in the project has been shown throughout its progress both in Canada and in other countries. Indeed, many of the results of the project are already being used in mine design. However, it is recognized that publication of the manual alone is not enough. Help is needed to assist engineers and planners to adopt the procedures described in the manual. This need for technology transfer will be met by a series of workshops for mine staff. These workshops will be held in various mining centres during the period 1977-81 following publication of the manual.

A noteworthy feature of the project has been its cooperative nature. Most organizations and individuals concerned with open pit planning in the country have made a contribution to the manual. It has been financed jointly by industry and the federal government.

Credit must be given to the core of staff who pursued with considerable personal devotion throughout the five-year period the objectives of the work from beginning to end. Their reward lies in knowing that they have completed a difficult job and, perhaps, in being named here: M. Gyenge, G. Herget, G. Larocque, R. Sage and M. Service.

D.F. Coates
Director-General
Canada Centre for Mineral and
Energy Technology

SUMMARY

Groundwater influences slope design primarily because groundwater pressure reduces the shear strength of discontinuities. Resistance to sliding is proportional to the normal stress acting through points of contact on the discontinuity. If water pressure acts on the surface, part of the normal stress is transmitted through the water. Less stress acts through the points of contact, and the frictional sliding resistance is therefore reduced.

OBJECTIVES

Groundwater investigations have two objectives:

a. to determine the groundwater pressures for use in slope design;
b. to determine ways of reducing adverse groundwater pressures, through drainage or other controls.

Slope stability analyses must evaluate the influence of groundwater pressure. If this is critical, methods of reducing pressure must be examined and their costs and benefits assessed.

PRESSURE DETERMINATION

Groundwater pressure can be measured directly by piezometers. The simplest is a tube sealed in a borehole. The lower end is open so that water can enter or leave. The level of water in the tube, or standpipe, is a measure of the pressure at the bottom of the tube. More sophisticated piezometers use direct pressure reading instruments sealed into the hole.

Piezometers must be carefully installed if they are to give reliable results. It is important to choose the correct type. For example, the standpipe, which requires an inflow of water to register pressure change, will respond very slowly to pressure changes in ground of low permeability. A more sophisticated piezometer that requires no water flow may be necessary if this delay is unacceptable.

Piezometers alone are not sufficient to determine groundwater pressures for design. First, it is not feasible to install the large number of piezometers required. Second, an important part of groundwater investigation is predicting pressure distribution in future slopes, when not only will mine geometry have changed but sources of groundwater such as streams may also have changed.

In practice, the groundwater pressure distribution throughout a slope is determined by combining field measurements and theoretical

studies. The field measurements determine the slope material properties that affect groundwater flow and ascertain sources of groundwater. Theoretical studies are then used to predict the groundwater pressures throughout the slope. Piezometers provide input data for theoretical studies, and - of most importance - monitor changes in pressure and verify the accuracy of predictions and the efficiency of drainage measures.

ANALYSIS

The key to theoretical study of groundwater pressure distribution lies in groundwater flow. A region of groundwater flow can be represented by flow lines and by lines of equal hydraulic potential. Potential is an important parameter in groundwater flow. It is defined as the elevation at a given point plus the pressure expressed in head of water. Groundwater moves from high to low potential; there is no flow along equipotentials.

Flow Net

Groundwater flow can be represented by a pattern of flow lines and equipotentials called a flow net. The upper boundary of flow is generally known as the water table. The water level in a standpipe installed in a slope rises to the level where the equipotential through the standpipe tip meets the water table.

Thus groundwater pressure can be determined from the equipotentials in a flow net. Strictly, the flow net represents water flow through a uniform porous medium. Flow through rock slopes occurs mainly along discontinuities rather than through the intact rock. However, a flow net can be used to represent average flow through rock slopes.

Permeability

The chief characteristic of the slope material that must be known before a flow net can be constructed is permeability. This is a measure of how much water will flow under a given potential difference. Various techniques are used to measure rock mass permeability in the field.

Surface and borehole mapping and associated core logging and drill records are used to locate fractures which affect permeability.

Falling head tests raise the pressure in a borehole above equilibrium and then measure the drop in this pressure with time. The relationship between flow and pressure is then used to calculate permeability.

Constant head tests measure the flow of water required to maintain a pressure above (or below) the equilibrium pressure. These tests are usually carried out on borehole sections at depth, isolated by seals known as 'packers'. The pressure/flow ratio can be used to calculate permeability.

Well or drawdown tests measure the change in surrounding groundwater pressure as water is pumped from a borehole. At steady state conditions, the rate of withdrawal can be used to calculate the permeability of the surrounding ground.

Flow Net Construction

Once permeability and other parameters such as sources of groundwater have been determined construction of the flow net can begin. There are several possible methods.

Graphical sketching is the simplest and has the advantages of being cheap and straightforward; for simple flow patterns it gives an insight into actual flow conditions. The principle is that for isotropic permeability conditions - permeability equal in all directions - flow lines and equipotential lines meet at 90°. Trial and error is used in sketching a set of lines to achieve this.

Electrical resistance analogues use the similarity between Darcy's law for the flow of water and Ohm's law for the flow of electricity. A network of resistors or a sheet of conducting paper is used to model the slope. The flow of current through the model represents water flow and voltage represents potential.

Numerical analyses using digital computers are the most powerful techniques available. In these analyses, the equations of flow for the slope are solved numerically and the rate of flow and distribution of pressure predicted. The advantage of numerical methods is that varying permeability can be analyzed with relative ease.

All these methods allow various slope geometries and other factors controlling groundwater to be considered. The best method for most purposes is numerical analysis by computer, though graphical sketching is a useful tool for simple cases.

Results of Analysis

Analyses result in a series of flow nets for existing and planned slope geometries, taking into account such factors as stream diversion and variation in rainfall. Flow nets allow groundwater pressures throughout the slope to be predicted; the effect of these pressures on shear strength is included in the stability analyses for design.

The actual pressure distribution in any but the most simple slope is complex. Current stability analysis techniques may require a simplified distribution. This can be done by estimating pressure as the vertical depth below the water table. This is usually conservative because pressure is over-estimated.

If variation of the groundwater pressure distribution is to be considered - for example, because of seasonal variations - this can be done by specifying an upper and lower boundary to the water table.

DRAINAGE

If groundwater pressure is contributing to instability, drainage may be a satisfactory remedial measure.

Before deciding on drainage, careful study of the undrained stability and of drainage methods and influence is required. This means appraising previous stability analyses, performing field tests to determine the potential of drainage and making theoretical analyses of the effects of drainage on groundwater pressure.

The theoretical evaluation of drainage uses the same techniques required to determine groundwater pressure distribution. For example, computer analyses can be used to obtain a flow net for the drained slope. Groundwater pressures can then be determined from this flow net.

It is essential to carry out field trials to verify that a proposed drainage scheme will achieve the desired results. A typical procedure is to install piezometers in a critical zone and then drill several drain holes or sink a well. Pressures before and after drainage can then be compared. It is important both in trials and in actual installations that the water-bearing formations or water-carrying discontinuities be tapped. The volume of water flowing from a well or drain-hole is not a measure of drainage effectiveness; the objective is to reduce pressure.

Drainage Methods

The choice of drainage method depends on many factors including slope height, permeability and economic and operational constraints. Four methods are widely used.

a. Horizontal or near-horizontal holes in the slope face are simple and relatively easy to drill; they require little maintenance and drain by gravity. Holes are usually lined with perforated pipe.
b. Vertical wells drilled behind the slope crest or on the slope face have the advantage of being away from the workings, and can be used for dewatering before excavation begins. Pumps are required, however, with corresponding maintenance costs.
c. Trenches down the slope face or along the slope toe are necessarily shallow and can only drain surface regions. However, where shallow instabilities are critical, trench drainage can be satisfactory.
d. Galleries excavated in the rock mass behind the slope are expensive, but, where large-scale drainage is required, are often the most effective method. They do not hinder operations and can be used for other purposes such as ore evaluation and structural mapping. Supplementary drain holes can be drilled from the gallery.

MONITORING

An essential aspect of groundwater investigation and control is regular monitoring of groundwater pressures by piezometers. Drain discharge, flow in streams and in adits and observation of face seepage are also useful indications of groundwater conditions.

MINE DEVELOPMENT STAGES

A preliminary assessment of groundwater is made in the feasibility stage. Regional characteristics are surveyed and existing information obtained from aerial photographs and maps and from well, stream flow and precipitation records.

During exploration drilling groundwater studies can be carried out at little extra cost. Such studies involve core logging, identification of discontinuities that form flow channels, permeability measurements and installation of piezometers.

Detailed groundwater investigations are required at the mine design stage. All representative exploration holes should also be used for permeability tests and piezometric measurements. Additional drilling specifically for groundwater investigation is usually required.

Data gathering is followed by theoretical analyses required to produce flow nets. These in turn indicate where drainage may be necessary and thus guide further investigations.

During the operating stage, groundwater conditions should be monitored regularly by a network of piezometers and by observing seepage, flow from drains, etc. The information obtained guides the re-investigation and analysis which may be required because actual ground conditions are not as anticipated or because the pit layout has changed.

COSTS

Costs for groundwater investigation vary greatly from mine to mine. The following estimates summarize actual experience for various depths of conical pits. Drilling and drainage costs are not included; 1975 dollars are used.

	Final Pit Depth - ft (m)		
Stage	100(30)	500(150)	1000(300)
Feasibility	$ 8000	16000	24000
Mine Design	$10000	20000	39000
Operating	$ 6000	11000	16000

The approximate time requirements for the various stages are as follows:

	Final pit depth - ft (m)		
Stage	200 (60)	200 - 500 (60 - 150)	500 - 1000 (150 - 300)
Feasibility	4 months	5 months	6 months
Mine Design	6 months	8 months	10 months
Operating	3 months	4 months	5 months

ACKNOWLEDGEMENTS

Roy Sage was responsible for production of this chapter. Address enquiries to him at: 555 Booth Street, Ottawa, Ontario, K1A 0G1.

John Sharp of Golder Associates, with assistance from Gordon Ley and other colleagues, wrote the chapter proper and most of the appendices. In the appendix dealing with permafrost, the experience of the Iron Ore Company of Canada Limited was described by Om Garg, and the experience at Clinton Creek Mine of Cassiar Asbestos Corporation Limited was described by Doug Piteau and Dennis Martin. Roy Sage drafted the appendix on regional groundwater, and the sections linking the groundwater evaluation and design procedures of the manual.

Many organizations and individuals contributed to the chapter, both by providing source material and by providing reviews. The co-operation of Canadian Johns-Manville Limited in providing details of their groundwater experience at Asbestos, Quebec is particularly acknowledged.

The principal contractors have been Golder Associates.

The Pit Slope Project is the result of five years' research and development cooperatively funded by the Canadian Mining Industry and the Government of Canada.

The Pit Slope Group has been led successively by D.F. Coates, M. Gyenge and R. Sage; their colleagues have been G. Herget, B. Hoare, G. Larocque, D. Murray and M. Service.

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INTRODUCTION

PURPOSE AND SCOPE

1. Groundwater is a significant operating factor in many open pit mines. Its influence is particularly important to slope stability because groundwater pressures adversely affect the shear strength of a rock mass and of overburden.

2. Several major slope instabilities have been partially attributed to the effect of ground water pressure, and have underlined its importance (1, 2, 3, 4).

3. The design of soil and rock slopes should therefore always include a study of the influence of groundwater. An evaluation program will include field measurements and interpretation, an analysis of how groundwater affects stability and an evaluation of drainage layouts to control its effect on stability. This chapter explains how groundwater pressure influences slope stability, and how to measure and evaluate its effect.

4. Groundwater evaluation techniques are similar for both soil and rock slopes. Emphasis in this chapter is on rock slopes because of their importance in open pit mining. Principal consideration is given to conditions within or near actual pit slopes rather than to regional groundwater factors; in most cases local conditions are more significant.

5. Dewatering for operating purposes such as to facilitate a particular mining method or to reduce blasting costs is not covered in detail, although many of the principles described in the chapter may be used for that purpose.

6. Guidelines are given for the investigation and evaluation of groundwater and for the implementation of such remedial measures as drainage. A certain degree of interpretation will be required in applying the approaches given to a particular site. An attempt has been made to cover the complex field of groundwater as thoroughly as possible; however, some degree of simplification has been necessary. The information given in the chapter should make it possible to realize when more specialized studies are required. In such cases, specialized assistance, often available within a large mining company itself, should be sought.

GROUNDWATER IN OPEN PIT MINING

7. Before commencing groundwater investigations it is necessary to establish the potential role of groundwater in the mine development. By considering general hydrological conditions, the proposed geometry of the open pit and data from initial geological assessments it is possible to classify groundwater conditions as favourable, marginal or unfavourable. This in turn determines the amount of resources which should be allocated to groundwater studies.

8. Figure 1 shows typical stages of investigation and interpretation leading to the kind of

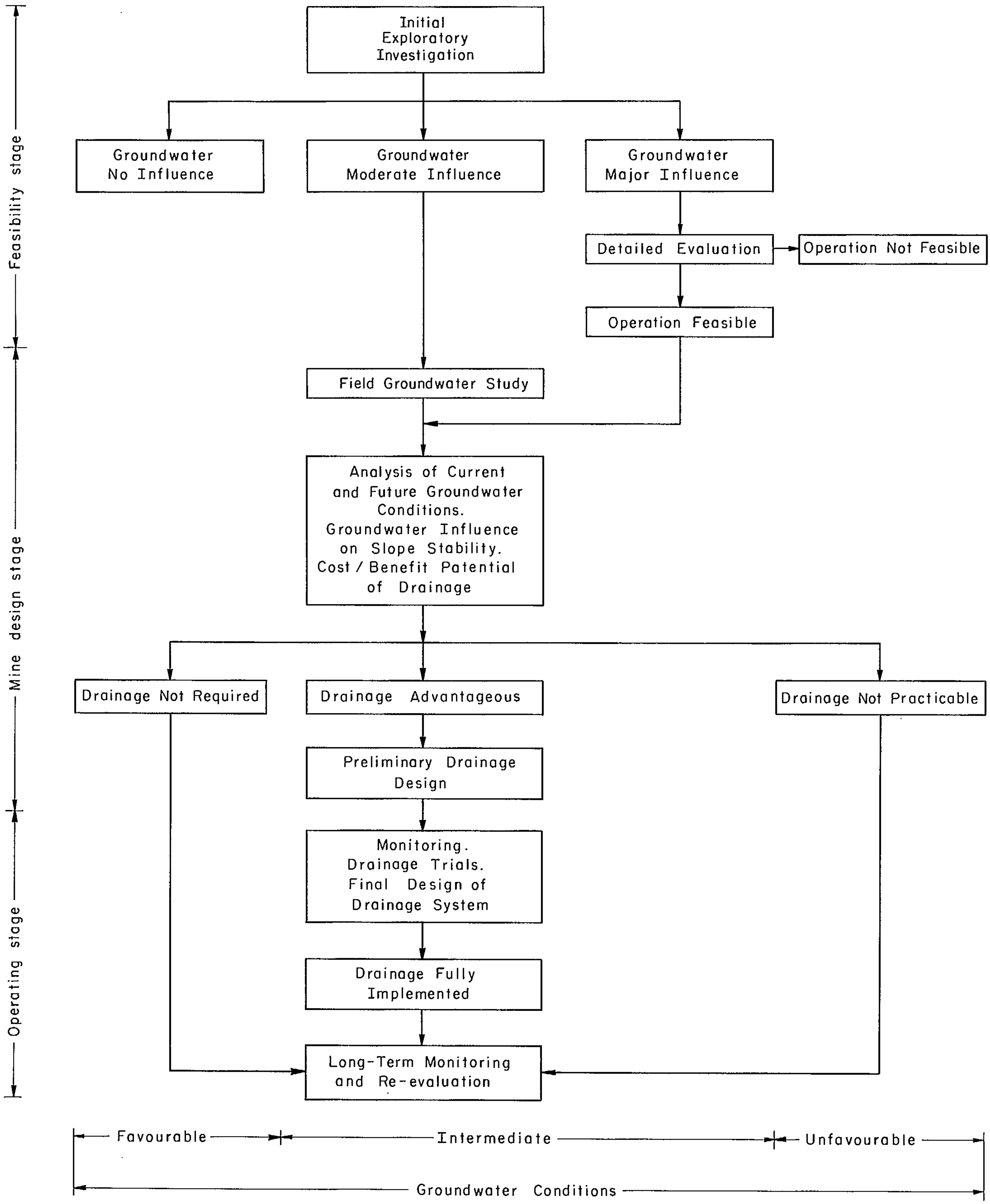


Fig 1 - Groundwater activities in the different stages of mining.

decisions which must be taken at the feasibility, mine design and operating stages of mine development.

9. In the feasibility stage, detailed studies should only be undertaken if groundwater conditions are considered critical to the operation. During the mine design stage, the influence of groundwater on slope stability and the applicability of drainage or other control measures should be determined. During the operating stage, it is possible to establish the interaction of mining and groundwater and the degree of natural drawdown or drainage. Careful monitoring during the initial operating stages will provide the most reliable long term forecasts of the potential influence of groundwater and the need for groundwater control.

ARRANGEMENT OF THE GROUNDWATER CHAPTER

10. Part I outlines the principles of groundwater flow and describes how groundwater pressure influences the shear strength of jointed rock masses. The determination of groundwater pressures within a rock mass is then described together with methods of assessing hydraulic properties. Analytical techniques for predicting overall groundwater pressure distribution at various mining stages are detailed. The concluding section describes the overall influence of groundwater on rock slope stability, and the method of introducing groundwater pressures to the analyses of the Design chapter.

11. Part II describes the practical aspects of groundwater control in open pit operations. The principles and methods of effecting slope drainage are described.

12. Part III is the key element of the chapter from the planning and management points of view. It describes the procedure to be followed in typical groundwater evaluation programs, relating various elements of such a program to the feasibility, mine design and operating stages. Although the treatment is necessarily general, emphasis is placed on the effort required under differing groundwater conditions and at different stages during development. For convenience of the user, much of this part is in the form of a checklist, which can be amplified through reference to other parts of the chapter.

13. Part IV consists of case studies which illustrate the methods and techniques described in the first three parts. The main study is hypothetical to provide a comprehensive illustration of the many aspects of groundwater problems. An example from an actual mining operation complete this part.

14. These four parts of the chapter describe what should be done, and when and why to do it. Practical details of the techniques required are described separately in the Appendices.

15. Figure 2 will assist the reader in judging an appropriate point of entry to the chapter. Those with some background in groundwater engineering are likely to find it convenient to

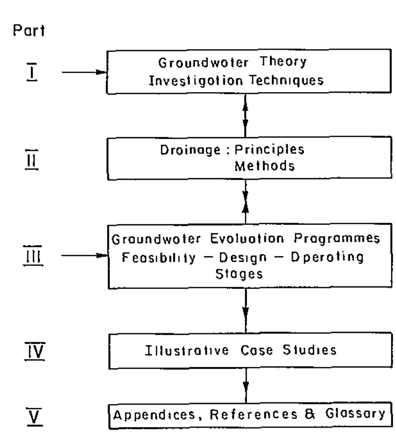


Fig 2 - Arrangement of groundwater chapter.

refer initially to Part III for the preparation of a groundwater evaluation program. Part III provides cross references to the theoretical aspects of various elements in Part I, to methods of slope drainage in Part II, and to procedural details for specific tests and analytical methods in the Appendices. Part IV amplifies the elements of Part III by illustrating them as parts of an overall study.

16. Published material appearing in the list of references is referred to by numbers in parentheses throughout the text. A Glossary is provided, in which terms in the text followed by an asterisk (eg, aquifers*) are defined.

PART I: GROUNDWATER THEORY AND INVESTIGATION TECHNIQUES

GROUNDWATER CHARACTERISTICS

17. Groundwater can be defined as water below the water table, ie, in the zone of saturation. Replenishment of groundwater is termed recharge, its fundamental source being precipitation. Loss of groundwater, eg, that which escapes through springs and evaporation, is termed discharge.

18. Sources of recharge greatly influence conditions in open pits. They include:

a. infiltration of rainfall and melting snow,
b. surface water bodies, eg lakes, rivers, tailings dams, and reservoirs, and
c. water in storage, ie groundwater contained in rocks and soils, which will move towards an excavation.

19. To assess the likely groundwater conditions for a potential open pit, the simplified hydrological balance between precipitation, surface run-off, evaporation, transpiration* and infiltration should be established. The topography and size of the catchment, vegetation type and its density, and the surface soil conditions, as well as the basic climatology of the region affect that balance. This is discussed more fully in para 51 to 56 and in Appendix H. The hydrological balance will indicate the potential for infiltration, ie, recharge, into the mine area. Topography and geology control the movement of water and thus the groundwater conditions.

20. Excavating an open pit below the water table will cause groundwater to flow into the excavation. Groundwater conditions in the pit slopes will vary with time, depending on the development of the excavation and on seasonal climatic variations. The latter may give rise to very adverse transient groundwater conditions.

21. In assessing the degree of recharge into a particular zone, the hydrological cycle, the regional hydrogeology and seasonal variations should all be taken into account.

22. In determining the influence of groundwater on slope stability, it is often most effective to consider the zone containing the slope in detail and determine the sources of recharge with respect to this zone. The loss of water or discharge in the form of face seepage etc, can be similarly considered.

23. The techniques which apply to the study of groundwater aspects of mining, particularly with regard to slope stability, are discussed in this part of the chapter. The theory of groundwater

*indicates term defined in glossary

flow, the interaction of mining operations with the groundwater regime, the effect of groundwater on the stability of slopes and methods for measuring or assessing significant groundwater and material parameters are described.

GROUNDWATER FLOW WITHIN ROCK MASSES

Permeability Conditions

24. The overall pattern of groundwater flow is controlled primarily by the regional topography and geology. Fundamental to any groundwater study is an adequate understanding of geological conditions.

25. Soils and rocks are classified with respect to groundwater flow by their permeability. Permeability, more correctly called hydraulic conductivity, is the rate at which water will flow through a material under a given pressure difference. For homogeneous materials, it is defined by Darcy's law:-

$$Q = Aki$$

where

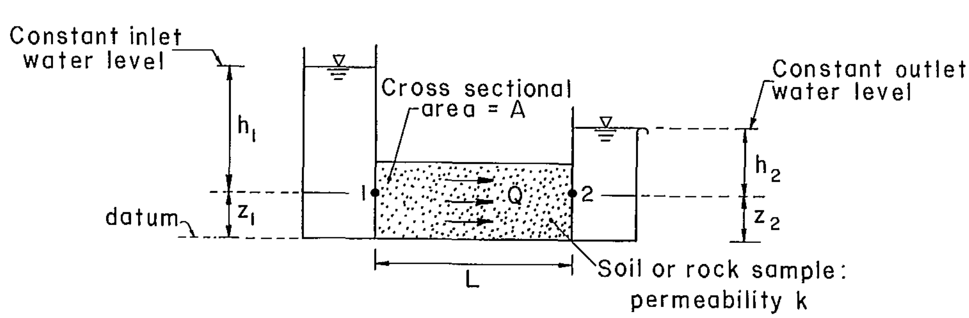
- Q = flow rate
- A = cross-sectional area through which flow takes place
- i = hydraulic gradient
- k = coefficient of permeability (generally a material constant)

26. The hydraulic gradient, i, is the change in hydraulic head, or potential, with distance measured in the direction of flow. The hydraulic head is defined for a given point as the sum of the elevation or height above datum and the pressure head or pressure expressed as head of water for the point concerned. Figure 3 shows a simplified permeability test which illustrates Darcy's law.

27. In soils, the permeability is influenced by both the particle size and thus pore space, and the distribution of particle sizes or degree of homogeneity. Methods for estimating the permeability of granular soils, based on particle size distribution, have been formulated (5).

28. The permeability of intact rock is usually low. In most rock masses, groundwater flow occurs largely through the discontinuities. The nature and orientation of these discontinuities thus determine the permeability of the rock mass. Because the discontinuities generally fall into "sets" having a given orientation, the permeability of the rock mass will vary with direction, ie, the rock mass exhibits anisotropic permeability.

29. In rocks with a defined structure and joints free from infilling, it is possible to define a joint conductivity, k_j, as shown on Fig 4



Hydraulic head at point 1 = $\phi_1 = h_1 + z_1$

Hydraulic head at point 2 = $\phi_2 = h_2 + z_2$

Hydraulic gradient within sample, $i = (\phi_1 - \phi_2)/L$

Flow rate through sample $Q = Aki$ (Darcy's law)

Fig 3 - Simplified permeability test.

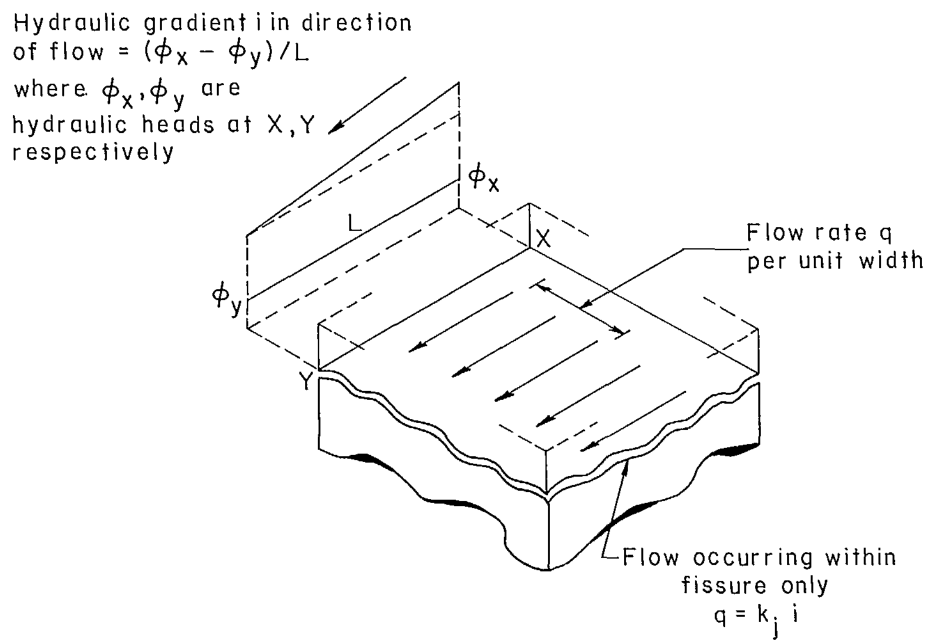


Fig 4 - Definition of joint conductivity, k_j.

(6). Generally, however, discontinuities form irregular and tortuous flow channels and may be infilled with permeable material, such as clay. In some rocks they may be locally enlarged by solution to form discrete flow paths.

30. Therefore, owing to the complexity of most rock types and the practical difficulties of performing detailed measurements in rock masses at depth, the groundwater flow characteristics of the rock must usually be described by a mass or bulk permeability. This expresses the total flow rate per unit area of the rock in a given direction and takes into account the contribution of all discontinuities for that flow direction. Where Darcy's law is used for groundwater studies in rock slope, the mass (or bulk) permeability value is applied.

31. Typical permeability values for some soils and rocks are given in Table 1.

Influence of Geology on Flow Through Slopes

32. Most open pits have both soil and rock slopes. The rock slopes may consist of several different rock types or structural domains and have zones of varying permeability which control the overall flow of water.

33. Commonly, more permeable regions of soil or rock masses, eg, in heavily fractured or porous rock, are termed aquifers, whilst impermeable zones, eg, intact rock and gouge filled faults, are termed aquicludes. The term aquitard is used for intermediate materials. Such terms are frequently applied to sedimentary environments, as illustrated in Fig 5. Here water flow occurs predominantly within the aquifers and results in the formation of perched water tables*.

34. The occurrence of faults in rock slopes can have a dramatic influence on both water flow and pressure. Low permeability faulting often occurs in association with orebody formations and obstructs flow, causing a build-up in groundwater pressure. Alternatively, shatter or breccia zones associated with faults can act as preferential flow paths. The significance of faults is discussed more fully in para 106 to 112.

35. In summary, the behaviour of groundwater in a rock or soil is determined by the lithology and the structural geology. Once rock types have been identified, important information can be deduced from published data on similar types, or from experience, before any permeability measurements are made. This applies not only to the

Table 1: Typical permeability values

Soils (intact permeability)	Permeability, k (cm/sec)
Gravel	$1 - 10^{2}$
Clean sands	$10^{-3} - 1$
Clayey sands	$10^{-6} - 10^{-3}$
Clays	$10^{-9} - 10^{-6}$
Rocks (intact permeability)	
Limestone, 2% porosity	8.5×10^{-8}
Limestone, 16% porosity	10^{-4}
Silty sandstone	2×10^{-6}
Sandstone, 29% porosity	2×10^{-3}
Granite	5×10^{-9}
Slate	10^{-10}
Rock masses (mass permeability)	
With clay filled joints	10^{-5}
Moderately jointed	$10^{-4} - 10^{-2}$
Well jointed	$10^{-2} - 10$
Heavily fractured	$10 - 10^{2}$

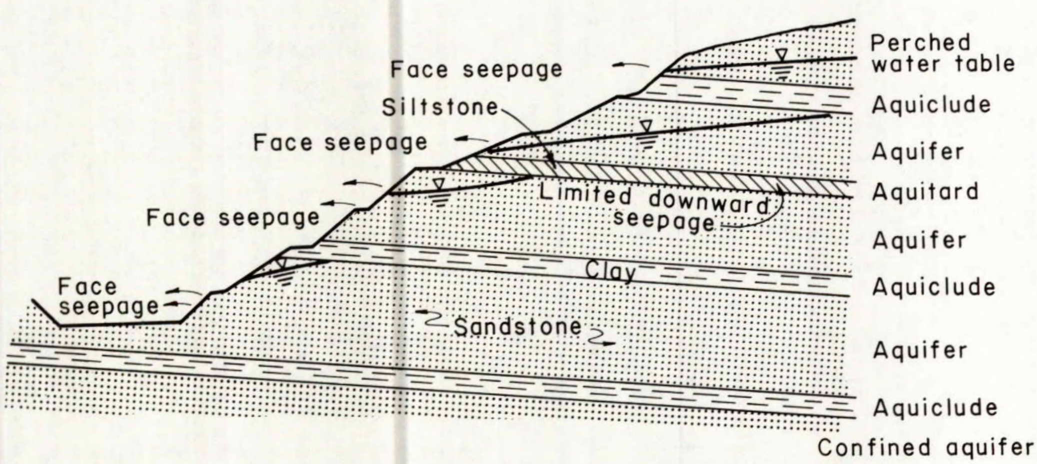


Fig 5 - Groundwater patterns in a stratified slope.

likely range of permeability values in the material itself, but also to any probable anisotropy. Shales can be expected to exhibit strongly anisotropic and very low permeabilities because of their laminated structure. Limestones are likely to exhibit solution effects and may contain water-filled caverns so that permeability will be very variable and sudden changes in flow rates are likely. Sandstones often have both appreciable intact and joint permeabilities and anisotropy will be dependent on the relative frequency of jointing in different directions. The overall hydraulic behaviour of the area can only be properly understood if the structural geology has been correctly defined. The effect of regional dips, major faults and the inter-relationship of aquifers and aquitards will indicate the direction and likely magnitude of flow around the excavation. Thus the knowledge of overall geological structure and permeabilities of the rock units leads to an understanding of the pattern of groundwater behaviour.

Groundwater Pressures Within Slopes

36. The excavation of an open pit causes groundwater to flow into it, setting up hydraulic gradients. The pattern of vertical and lateral variations in hydraulic head is called the groundwater pressure distribution. The flow pattern, and thus the groundwater pressure distribution that will be generated within a slope, will depend upon the following factors:

a. geometry of the slope
b. the permeability characteristics of the slope material
c. recharge from the surrounding rock mass
d. water storage within the slope
e. local precipitation, run-off and infiltration characteristics.

37. The overall flow pattern and pressure distribution in a slope can best be represented by a "flow net" (Fig 6). Flow nets are plots of flow lines (the paths which molecules of water will follow when flow is taking place) and of equipotentials (contours of equal hydraulic potential). In the simplest case of an isotropic material, which has equal permeability in all directions, the equipotentials will intersect the flow lines at right angles. By definition, no flow occurs at right angles to flow lines.

38. To construct a flow net, the groundwater pressure at a number of representative points must either be measured directly or be computed. Methods of doing this are explained later. The construction of flow nets is described fully in Appendix F.

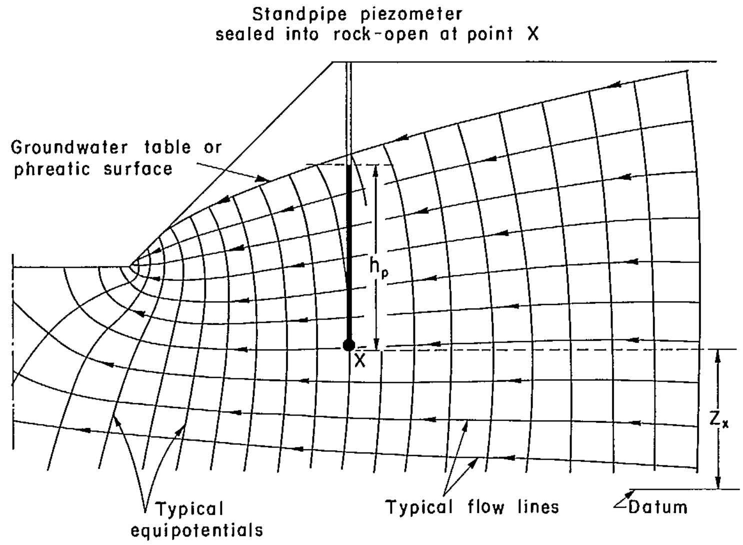
39. From the flow net, the flow through a given zone as well as the groundwater pressure at any point can be determined. The groundwater pressure at a given point expressed as a pressure head is the difference between the hydraulic head and the elevation at that point. The value of any given equipotential is given by its elevation where the pressure head is zero, ie, at the groundwater table or phreatic surface.

40. In homogeneous, isotropic rocks and soils, the groundwater pressure distribution within the mass will depend only on the geometry and flow conditions at the boundaries, and will be independent of the absolute magnitude of permeability.

41. Isotropic conditions are rare in open pits. In non-homogeneous and anisotropic materials the spatial and directional variations in permeability will influence the groundwater pressure distribution. Figures 7(b) and (c) illustrate equipotential distributions for anisotropic slopes.

42. In Fig 7, only equipotentials and the water table are shown. Flow lines would intersect equipotentials orthogonally only in case (a) which corresponds to Fig 6. A fuller discussion of this aspect is given in Appendix F.

43. Only in the case of nearly horizontal flow in isotropic materials can direct measurement of the water table, as determined from water level measurement in open boreholes, provide sufficient data to determine the groundwater flow and pressure distribution. These conditions may apply to granular soils and isotropic rocks in flat topography. They do not apply near natural or excavated slopes where steeper flow gradients exist or in non-homogeneous or anisotropic materials. In these cases, the pressure



Hydraulic potential at point X = elevation of X + pressure head

$Z_x + h_p$

h_p is determined from intersection of equipotential through point X and groundwater table.

Fig 6 - Flow net for seepage through a slope.

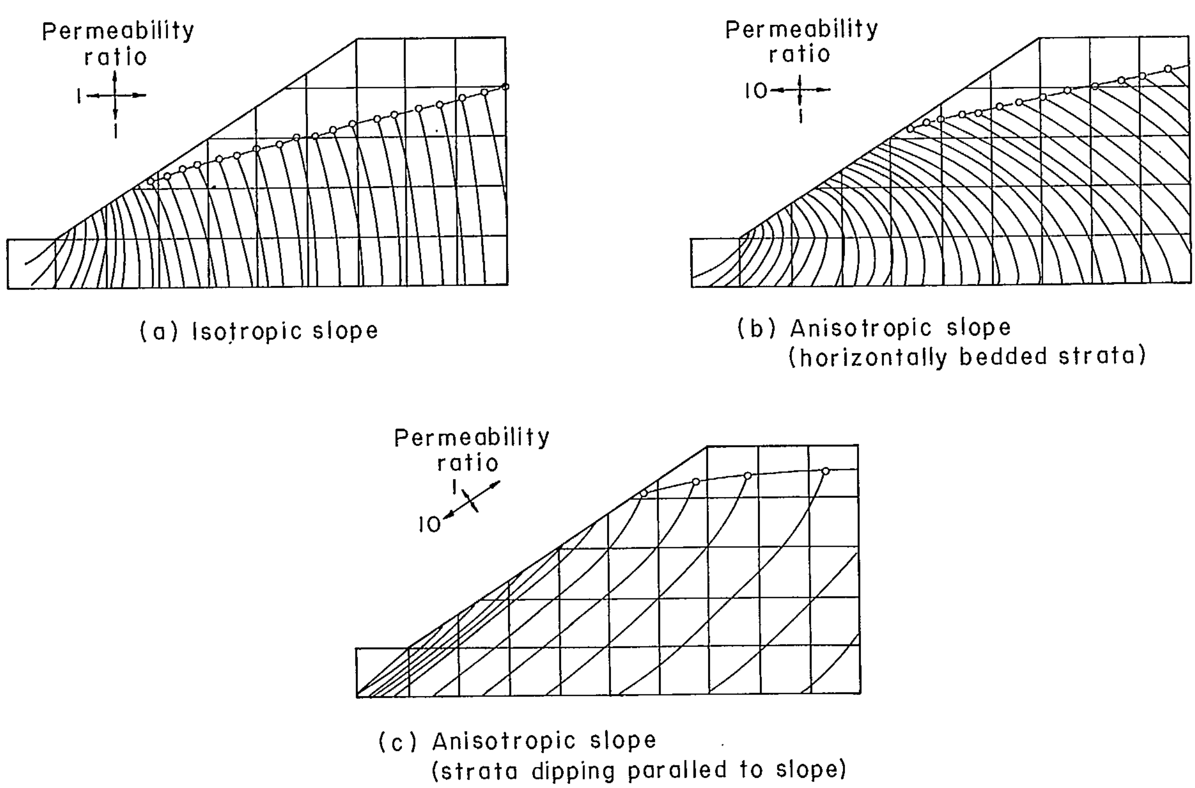


Fig 7 - Equipotential distributions for slopes with varying permeability ratios.

distribution must be assessed by direct measurement at specific points using piezometers or by analytical techniques based on permeability and other data obtained from field studies. These methods are described in para 61 to 70 and 113 to 146 respectively.

INFLUENCE OF GROUNDWATER ON SLOPE STABILITY

44. The primary effect of groundwater pressure in reducing the stability of rock slopes is the resulting decrease in the effective shear strength of discontinuities. This phenomenon is described by the effective stress principle which is fundamental to the understanding of the influence of groundwater on rock slope stability. The principle applies equally to soils and other permeable materials (7,8).

45. The stresses at a given point in a saturated rock mass depend on the influence of gravity, tectonic and other stresses, and on water pressure. Consider a joint, as illustrated in Fig 8(a), which is critically oriented with respect to stability. The stresses acting on a portion of the joint can be resolved into components normal and parallel to the joint plane. In a simple normal stress versus shear strength relationship, ignoring water pressure effects, the magnitude of normal stress governs the shear strength that can be mobilized. This relationship is given by:

$$s = \sigma_n \tan \phi$$

where s is the shear strength.
σ_n is the normal stress acting across the joint.
ϕ is the angle of shearing resistance.

In this case, the normal stress, σ_n, is wholly transmitted through the asperities or contact points of the rock, which in turn account for the frictional resistance of the joint.

46. Consider now a water pressure, u, acting within the joint. The total applied stress across the joint will be transmitted by the rock asperities and by the fluid pressure. The normal stress transmitted by the fluid is equal to the joint water pressure. The stress transmitted through the rock asperities is therefore equal to the total applied stress minus the joint water pressure. The joint shear strength will now be reduced proportionately. The reduced normal stress acting through the rock contacts is termed the effective normal stress, and is given by:

$$\sigma_n' = \sigma_n - u$$

where σ_n' is the effective normal stress.
σ_n is the normal stress.
u is the water pressure.

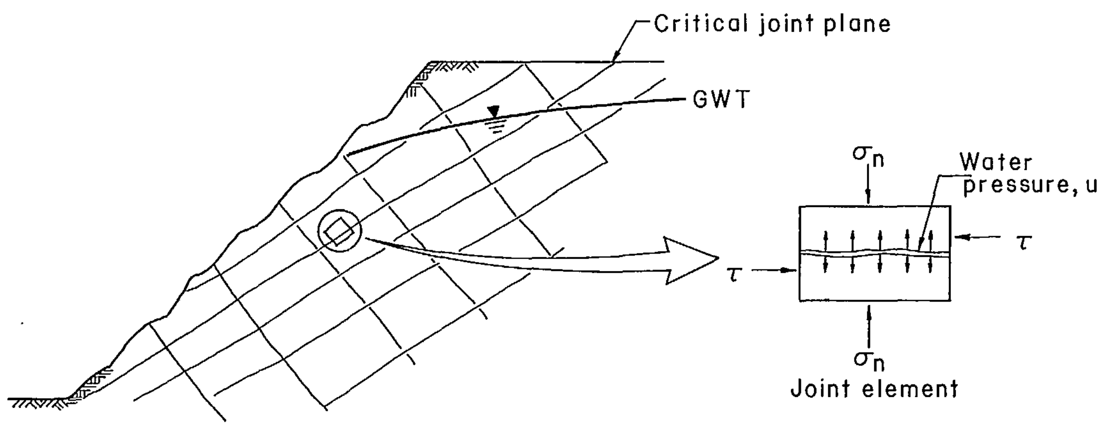
47. The corresponding shear strength that can be mobilized is:

$$s = \sigma_n' \tan \phi$$
$$= (\sigma_n - u) \tan \phi$$

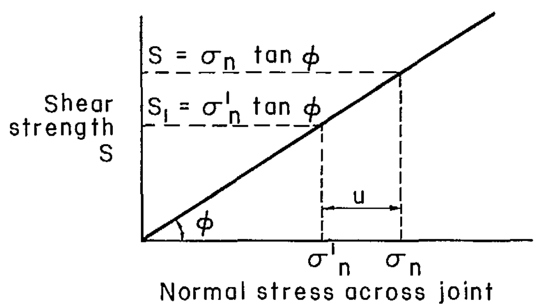
Obviously, when the water pressure, u, is equal to the applied normal stress, the effective shear strength of the joint reduces to zero.

48. In practice, the relationship between shear strength and normal stress for natural rock joints is a function of the geometry of the joint surface as well as of the frictional properties of the material. The geometry of the joint surface depends on the nature of the joint - either shear or tensile - and its prior displacement history. Therefore, the relationship in practice is a curved shear strength envelope, as shown in Fig 8(c). To determine the shear strength in a particular instance, the effective normal stress should first be computed. The shear strength is then determined directly from the shear strength versus normal stress diagram.

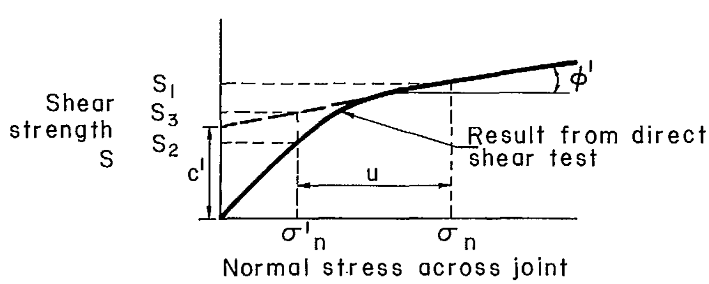
49. The above approach will give the correct value of shear strength provided groundwater pressures and shear strength parameters are correctly determined. However, stability analysis techniques currently available cannot readily determine shear strength from the non-linear relationship between shear strength and normal stress (Chapter 5). An alternative approach is therefore often used. In this, the curved



(a) Stresses on a joint within a rock slope



(b) Simple shear strength/normal stress relationship



S_1 = Shear strength with no water pressure in joint

S_2 = Shear strength with water pressure u within joint

S_3 = Shear strength with water pressure u within joint assuming apparent cohession c' and linear shear strength characteristic

(c) General shear strength/normal stress relationship

Fig 8 - Influence of water pressure on the strength of rock joints.

strength envelope is approximated by a straight line defined by the well-known Coulomb parameters "effective cohesion", c', and "effective angle of shearing resistance", ϕ'. The shear strength is then given by

$$s = c' + (\sigma - u) \tan \phi'.$$

Although this approximation is commonly used, it can lead to errors unless the straight-line approximation corresponds closely to the strength curve for the range of effective pressures that will actually occur. For example, in Fig 8(c), assuming an apparent cohesion, c' and a linear strength relationship, the value S_3 would overestimate the value of the shear strength by (S_3 - S_2). When the Coulomb strength equation is used, the shear strengths determined should be checked against the true strength envelope.

50. A second, and quite separate, effect of groundwater on some rock types, principally those containing layer lattice minerals, such as chlorite, talc and clays, is to reduce the intrinsic frictional resistance of the rock. This will not be discussed further here, as it relates to mechanical properties and is covered in chapter 3.

EVALUATION OF REGIONAL GROUNDWATER CONDITIONS

51. In the early stages of mine planning, a regional investigation is required to delineate in broad terms the geology and hydrology of the zone around the proposed mine. This study should define the geological structure, the likelihood of aquifers affecting the mine and the probable influence of topography and precipitation on surface water and groundwater conditions at the mine site.

52. Such a study will indicate the magnitude of groundwater problems in the mine area, in terms of dewatering requirements and slope stability, and any problems resulting from dewatering, eg, depletion of domestic supplies or settlement under existing structures. It will indicate possible surface and groundwater pollution problems and assist in the siting of tailings ponds and dumps. Finally, and of greatest significance, it will determine the possible scope, scale, cost and timing of more specific additional studies.

53. The key to a successful regional groundwater appraisal is to thoroughly define the regional structural geology and lithology. Published geological data, information from other operations or investigations, and special techniques such as aerial photography and satellite imagery should be employed.

54. Geological mapping and collecting hydrological field data may also be required together with techniques such as tracer tests and geophysics.

55. The precipitation characteristics of the region should be determined from published data or by direct measurement. A simple hydrograph* should be obtained for local rivers or streams to provide information on the hydrological balance, ie, run-off versus infiltration, which will indicate probable seasonal variations in groundwater conditions.

56. Precipitation and geological data should be analyzed and presented on plans and cross sections to show the influence of regional conditions on the mine area. Techniques for regional groundwater evaluation are discussed more fully in Appendix H.

EVALUATION OF GROUNDWATER CONDITIONS WITHIN SLOPES

57. Although regional studies are valuable in the early stages of investigation, engineering solutions to groundwater problems in mining almost invariably require detailed local studies of existing or proposed pit slopes. As shown, the most important groundwater parameter for stability purposes is the groundwater pressure distribution within the slopes (para 44 to 49). This can be obtained in two ways:

a. by direct measurement of pressures using piezometers;

b. by determining pressures from an analysis of the hydraulic properties of the rock mass, eg, geology and permeability characteristics.

58. In practice, except in simple geological environments, it is extremely difficult to measure pressures at a sufficient number of points to enable the distribution of pressures within zones

of potential instability to be determined. Alternatively, predicting pressures from an analysis of groundwater flow based on permeability data and groundwater sources alone is inherently inaccurate except under ideal conditions.

59. The most satisfactory approach is usually to measure groundwater pressures with piezometers at representative locations, and to correlate these data with analytical studies based on a thorough understanding of the geology and on selected permeability or conductivity measurements on representative soil and rock strata.

60. All investigation programs should be carried out in stages to permit re-evaluation of requirements as the investigation develops. Holes drilled for permeability measurements can usually be used for piezometer installations. However, since greater emphasis is placed on piezometric measurements, they will be discussed first.

Measurement of Groundwater Pressures

61. Measurement of groundwater pressure is required not only for stability analyses but also to monitor changes in groundwater conditions resulting from seasonal variations, mining activity and drainage. Systematic measurement of changes in groundwater conditions with time indicates the overall hydraulic properties of the rock mass; during the initial excavation stage such measurements can be used to predict long-term conditions.

62. Except in simple geological environments, hydraulic head within rock slopes varies significantly with depth and from point to point. It should be determined by measuring groundwater pressure with a piezometer at a known elevation.

63. Water levels measured in open boreholes can provide useful data on groundwater table elevations only in isotropic rock masses where there is little flow. Near excavated or natural slopes, or in anisotropic geological conditions open holes will not give reliable indications of groundwater pressure because of the variation in hydraulic head with depth (para 40 to 43). They may also be affected by surface water infiltration.

64. Piezometers are transducers which convert water pressure to some more readily measurable form of output, eg, elevation head, electrical voltage or current. They are generally installed in boreholes and should be sealed into the holes so that pressures are measured at the sampling points only. If the output information is to be of value it is essential that the sealing of the piezometer is completely effective. The problems of sealing in deep boreholes are such that properly designed equipment and experience are necessary to ensure a reliable installation. It is normally very difficult to check the reliability of an installation once completed.

Types of Piezometers

65. A general classification of piezometers listing the most important characteristics is shown in Table 2 and is further illustrated in Fig 9.

66. The most important physical properties required in a piezometer are ruggedness and long-term reliability. Piezometers must be selected to suit the expected permeability of the rock mass. To record changes in groundwater pressure accurately, the volume of water required to operate the device must be readily available from the rock mass without resulting in a significant pressure change. Thus, high-volume demand piezometers or standpipes are unsuitable for measuring rapid changes in groundwater conditions in low permeability materials, although they may be adequate for monitoring long term trends.

67. Full descriptions of piezometer types, selection of suitable tips and installation techniques are given in Appendix A.

68. The response characteristics of various piezometers can be compared by their time lag. This is time for the piezometer to reach equilibrium with the groundwater pressure at the point of installation.

69. The importance of correctly sealing the collector zone has already been stressed. Some installation methods are shown in Fig 10.

70. A cylindrical probe is most often used for measuring the water level in a standpipe piezometer; one type is shown in Fig 11. The

Table 2: Piezometer types - general classification

Broad classification	Class	Relative volume demand	Readout equipment	Major limitations	Major advantages
Open tube (standpipe)	A	High	Water level finder	Longer time lag in most rock types. Tube must be straight for whole length. Difficulties likely in small diameter tubes if water levels significantly below 100 ft, or dip less than ~ 45°.	Cheap. Simple to read.
Closed tube (hydraulic)	B	Medium to low	Usually Bourdon gauge or mercury manometer	Requires readout location not significantly above lowest water level. Therefore not suitable for general borehole use.	Relatively cheap.
Mechanical diaphragm (pneumatic or hydraulic)	C1* C2*	Low to very low Negligible	Specialized pressure transmitter	Hydraulic types require periodic 'de-airing' of monitoring system.	Excellent long term stability. Can be made very small. Simple to install.
Electrical	D	Negligible	Specialized electronic readout	Relatively expensive especially if cable lengths large. Some zero drift possible. Certain types may be susceptible to blast damage.	Ideal for remote monitoring. Installation simple.

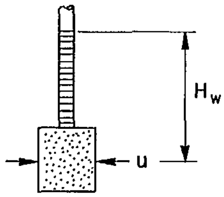
* C1 Class - monitoring input closes bypass valve.
C2 Class - monitoring input opens bypass valve.

probe is lowered down the standpipe by its cable which is usually graduated in feet or metres. When it reaches the water surface, a circuit is completed between the two exposed electrodes producing an audible or visual signal by means of a buzzer or lamp. The depth is then read from the cable. These probes are also used for various borehole permeability tests described later.

Rock Mass Hydraulic Properties

71. The hydraulic properties of rock masses are required for classifying rock types into similar hydrogeologic units. This is for use in analytical methods for deriving groundwater pressure distributions, such as flow nets and numerical analyses, to assess the drainage potential of rock masses, and to design the layout of arrays of piezometers for groundwater pressure determination.

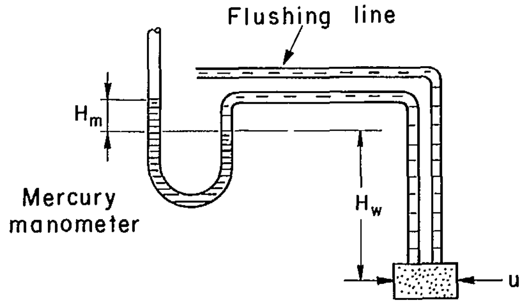
72. The hydraulic property most frequently required for groundwater studies is the mass or bulk permeability, or mass hydraulic conductivity of



(a) Class A

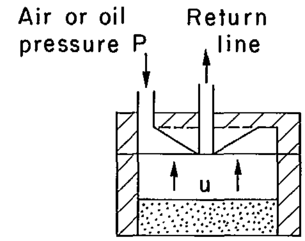
Level measured by water level finder

$u = \gamma_w H_w$



(b) Class B

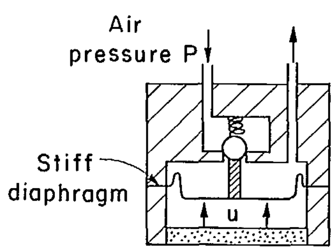
$u = H_m \gamma_m + H_w \gamma_w$



(c) Class C_1

Water pressure against diaphragm seals off outlet. P is increased until valve opens, then

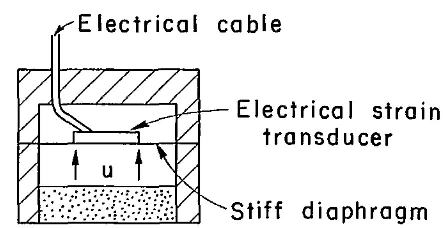
$u = P$



(d) Class C_2

Water pressure against diaphragm opens ball check valve. P is increased until valve closes, then

$u = P$



(e) Class D

$u = f(\text{gauge output})$

Fig 9 - Piezometer classes showing principal characteristics.

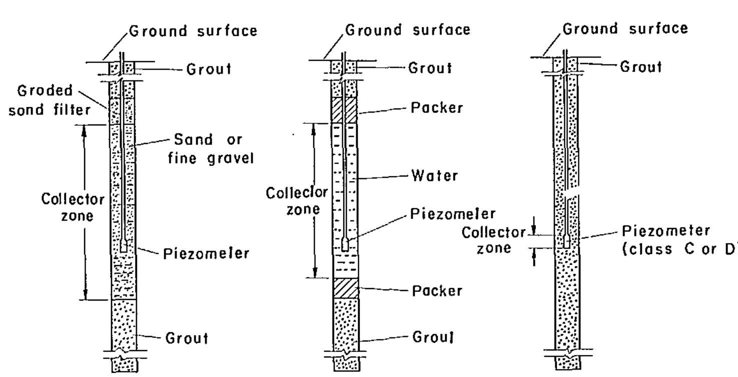


Fig 10 - Possible piezometer installation configurations. Left: filter technique; Centre: packer technique; Right: fully grouted (very low permeability environments only).

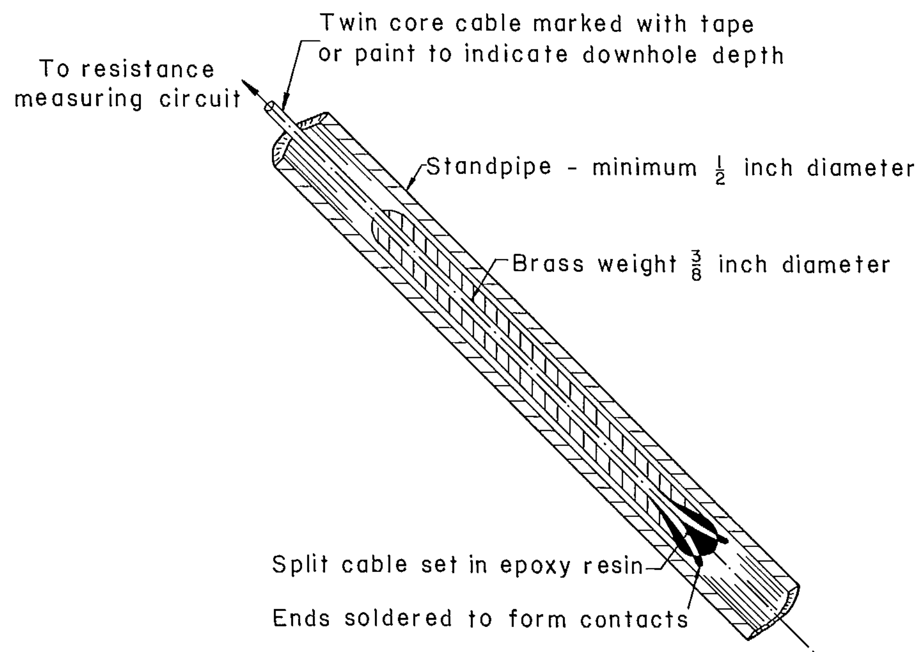


Fig 11 - A simple probe for water level detection.

the rock. The permeability characteristics of the rock mass will, as previously discussed, influence the distribution of groundwater pressures. The accepted methods of permeability measurement use boreholes, wells and large-scale openings and all tests are based on the principle of varying the pressure conditions within a zone of the rock mass and measuring the resultant flow.

73. When assessing the most suitable or cost-effective method of testing permeability, the scale of the test in relation to the scale of the rock mass involved must be considered. Borehole testing techniques are small-scale tests and influence only a very small zone around the borehole; they cannot be considered representative of the rock mass in general unless good geological correlation can be established. Well and adit tests affect large volumes of rock and are large-scale tests; they are however expensive. The choice of test method must depend on local site conditions, the economic significance of the data obtained, and the scale of mining proposed. Additionally, indications of the suitability of one method over another will result from the more general groundwater appraisals carried out earlier in the development, as described above.

Rock Mass Permeability Measurement using Boreholes

Drilling Records

74. Exploration drilling records should be designed to provide groundwater information. The location of losses or gains of drilling water, or of water blowing from percussion holes, should be recorded. Soft or shattered ground or zones where the drill tends to jam should similarly be logged. The borehole water level should be measured and recorded at regular stages during drilling and each morning for a few days after the hole has been completed.

75. From such records, it may be possible to determine the position of particularly free-flowing joints or faults, and some correlation may be possible. Holes which make water can be differentiated from those which lose it, and again some grouping or correlation may be possible.

76. Although such data may not appear significant at the time, later correlation with core examination and subsurface geological cross

sections may enable particular water-bearing joints or thin aquifers to be recognized.

Core Evaluation

77. Detailed studies of core should always precede borehole permeability testing as they permit a qualitative estimate of relative permeabilities.

78. The nature of joints and any infilling observed in good quality core is a guide to the magnitude of flow that can occur through the rock mass. Staining of the joint surfaces generally indicates hydraulic connection with near-surface oxidized zones. Preferential staining or deposition on specific joint sets may indicate a preferred flow direction. Certain rock types, notably limestones, may contain solution channels that are tortuous and irregular compared with the jointing. Silicified zones in metamorphosed rocks can give rise to similar conditions, although usually on a reduced scale.

79. Such qualitative observations regarding relative permeabilities should be combined with geotechnical classifications of the core which indicate relative degrees of fracturing, eg, RQD* and drilling records. In conjunction with borehole location plans and geological cross sections, these data can be used to identify zones of low, medium and high permeability and major water-bearing features, which can be located and plotted on the cross sections. In this way, the basic elements of a groundwater model of the mining zone can be compiled.

80. Another important function of core inspection is to locate and determine the length of test zones. From the groundwater model, test locations can be selected which will give representative measurements of the various hydrogeological units. The length of the test zones will depend typically on the rock type, homogeneity and degree of fracturing or joint spacing; test sections containing an average density of fractures should be selected for each unit to be tested to obtain average values. If packers are to be used, the test location and length should ideally avoid soft or broken rock in the vicinity of the packer positions for the test to avoid seating problems.

Falling Head Tests

81. In the falling head test, a section of the borehole is subjected to an increased pressure head above the static groundwater pressure. This head is then allowed to fall or dissipate to the static value, and measurements of the loss in head with time are taken, using a probe (Fig 11) or a pressure transducer. From this, a relationship between flow and pressure is determined from which permeability is calculated.

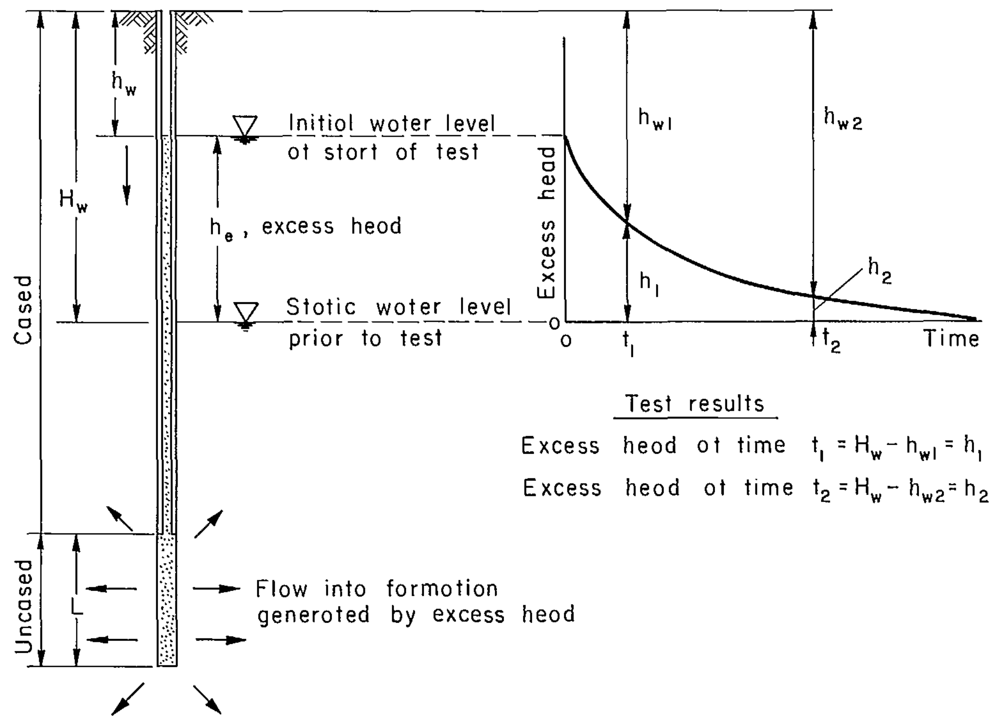
82. The test is carried out on an uncased section of the borehole using the principle shown in Fig 12. The length of the uncased section will often be governed by the type of ground and drilling techniques, but will usually exceed 10 ft (3 m). The test location should be thoroughly cleaned by air or water flushing prior to the test. Drilling mud should not be used.

83. The test is rapid and easy to carry out because it requires minimal interruption of drilling procedures. Stage testing to produce a permeability profile can be included. If, during drilling of an uncased hole, the test is carried out over successively longer sections of the hole corrections can be applied to determine the permeability of each increment. Full details of the test procedure and the analysis of the results are given in Appendix 8.

Constant Head Tests

84. The principle of the constant head test is to apply pressure to a known section of the borehole above, or sometimes below, the static pressure level and to measure the resultant flow into or out of the formation. In its simplest form the test is performed in an open or a partially cased hole. Water is pumped into or out of the hole to maintain a steady level above or below the static level.

85. In deep holes it is usual to isolate a specific test section of the borehole by means of packers and to pump water into or out of that section. If the test is carried out at the end of a borehole, only a single packer is required. If the test is carried out over intermediate sections of the hole, two packers are required, one above and one below the test section. The hole should



Permeability, $$k = \frac{2.3 \log_{10}(h_1 / h_2)}{t_2 - t_1} \cdot \frac{r_c^2}{2L} \cdot \ln(R/r) \quad \text{m/min.}$$

$\ln(R/r)$ may be taken as 7

in which case

$$k = 0.133\ S\ \frac{r_c^2}{L} \quad \text{m/sec}$$

where S is the gradient of the log head time graph (Fig. B-3)

Fig 12 - Principle of the falling head test.

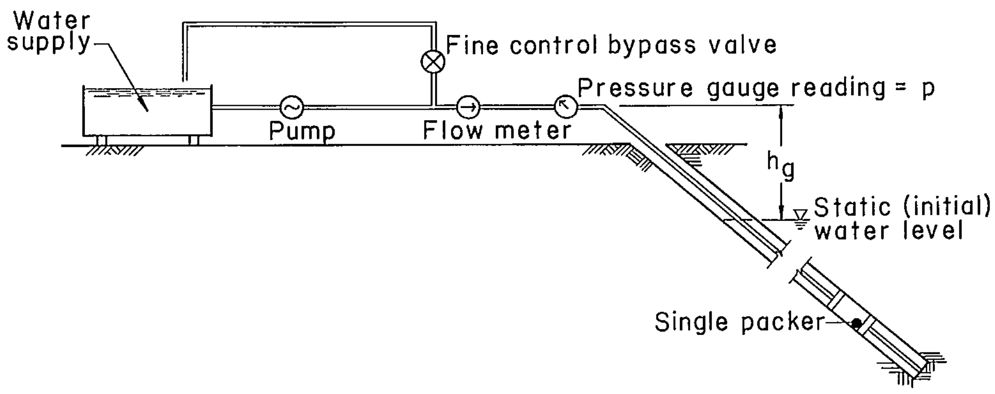
be flushed clean prior to testing. Mud should not be used as a drilling fluid.

86. The arrangement of equipment required is shown in Fig 13, and consists of a pipe from surface terminating in a perforated section and isolated by one or two packers. The pipe should be at least 0.75 in. (2 cm) diameter to prevent significant head losses along its length. The system should be calibrated for head losses before use.

87. Packers are generally made of rubber or a synthetic equivalent and are of two types:

a. mechanical compression
b. pneumatic.

88. Mechanical compression packers, as shown in Fig 14, expand and form a seal when compressed either by a screw fitting or by pressure applied to the drill stem. Pneumatic packers, as shown in Fig 15, are inflated by compressed air through an air line connected to a surface compressor or storage tank. The length of the packer should be sufficient to form an effective seal and prevent short circuiting via open fissures alongside the packer, say a minimum of 6 ft (2 m). The length



p = Pressure recorded on gauge

$\frac{p}{\gamma_w}$ = Pressure head

h_g = Height of gauge above static water level

h_f = Friction head loss (determined by separate test)

$$\text{Excess head on test zone, } h_e = \frac{p}{\gamma_w} + h_g - h_f$$

Fig 13 - Layout of constant head test.

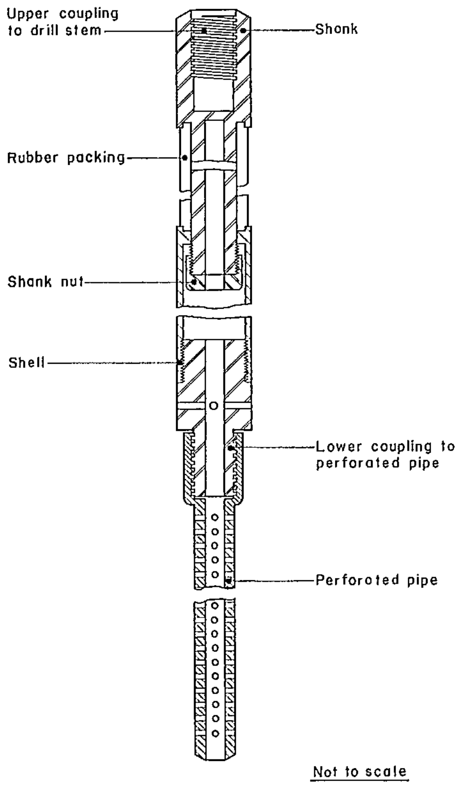


Fig 14 - Single borehole mechanical packer unit.

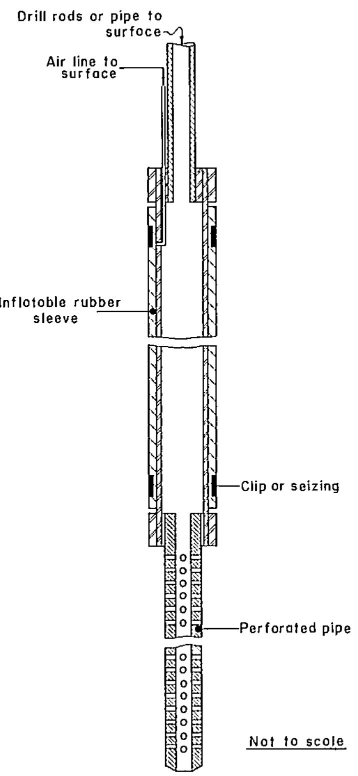


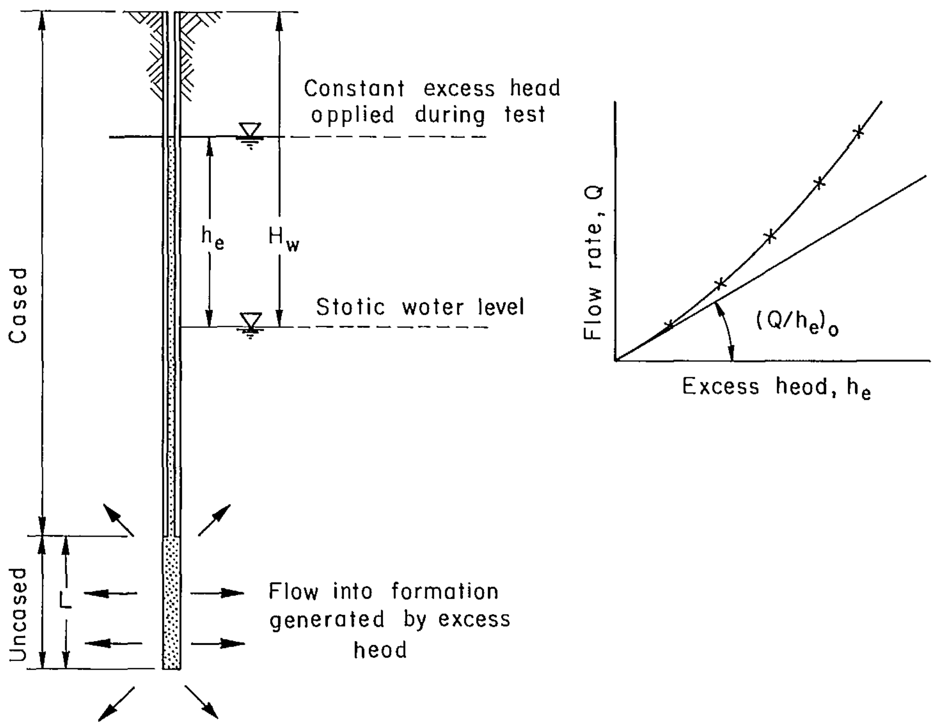
Fig 15 - Single borehole pneumatic packer unit.

of the perforated discharge section should be variable to permit adjustments in the field.

89. The best system for routine testing, eg, to determine a continuous permeability profile, is usually a single mechanical packer operated by drill stem pressure. The hole is tested in stages during drilling, thus ensuring that downhole access is always maintained.

90. Water pressure in the test section may be deduced by taking a surface measurement or by measuring directly with a down-the-hole pressure transducer. Tests are carried out at several pressures, the maximum normally not exceeding the lower of 100 psi (700 kPa) or 50% of overburden stress. The flow is noted at each stage. The permeability of the rock in the test section is then calculated from the pressure/flow relationship, as shown in Fig 16. Full details of the test procedure and analysis of the results are given in Appendix C.

91. Borehole permeability testing requires considerable experience. Structural geological interpretation should always be integrated with the test results. Many refinements to this technique are available for specialized testing to determine anisotropy and other characteristics of



$$\text{Permeability, } k = \frac{1}{2\pi L} \cdot \left(\frac{Q}{h_e}\right)_0 \cdot \ln(R/r)$$

$\ln(R/r)$ may be taken as 7.

If Q is litres/min. and dimensions are in metres,

$$k = \frac{5.833}{\pi L} \cdot \left(\frac{Q}{h_e}\right)_0 \cdot 10^{-5} \quad \text{m/sec.}$$

Fig 16 - Principle of the constant head test.

rock masses (10 - 15).

Rock Mass Permeability Measurement from Well Tests

92. The principle of the well test is to abstract water from a given formation and to observe the resultant change in level in the well and in the groundwater pressure at a number of points within the formation, measured by piezometers in observation holes. The rate of change of groundwater conditions can then be used to determine permeability and other hydraulic properties of the formation.

93. The well test is generally carried out using large-diameter vertical boreholes and is a large-scale test. Difficulties in interpretation arise wherever the permeability distribution is complex. Analysis is thus most straightforward in horizontally stratified, relatively permeable formations or aquifers as illustrated in Fig 17.

94. When water is withdrawn from a well at a steady rate, the water level in the aquifer is drawn around the well and forms a cone of depression as shown in Fig 18. The drawdown is greatest at the well and diminishes outwards. Hydraulic gradients are thus created which result in groundwater flow towards the well. With time, the cone of depression is steadily enlarged until either the reservoir available to the pump is exhausted or until the cone of depression reaches a source of recharge large enough to sustain the

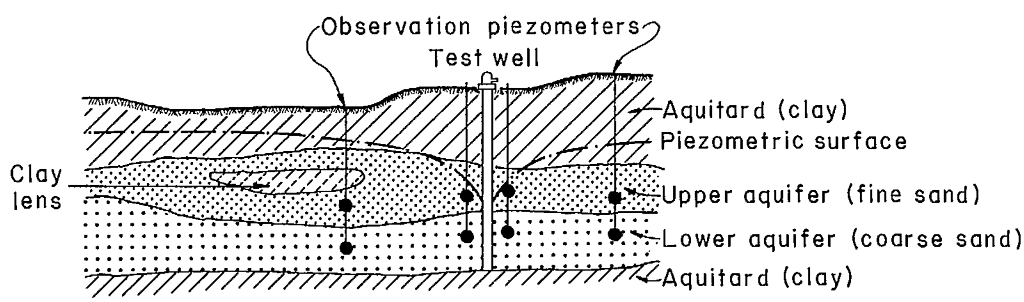


Fig 17 - Typical well test application.

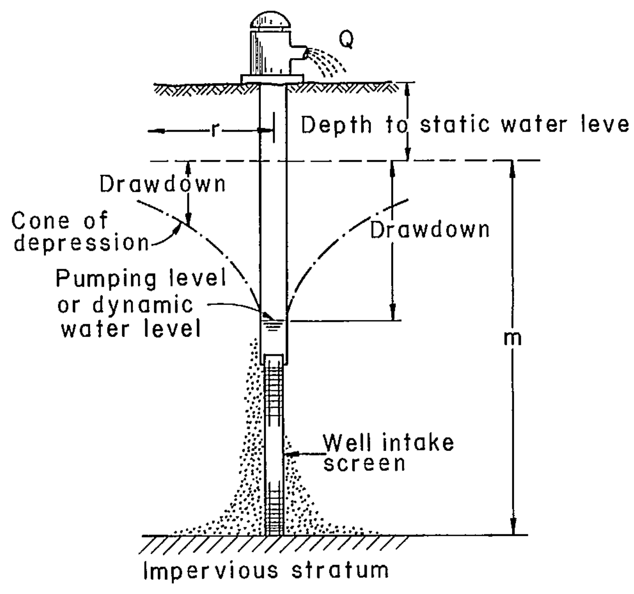


Fig 18 - Measurements related to well tests.

yield of the well and halt further decline of the water level, ie, a state of equilibrium is reached.

95. Observation holes should be located taking into account the geological structure, permeability of the horizon being tested, and the degree of hydraulic connection between the well and the observation points. To facilitate interpretation, at least two observation holes should be installed in a line from the well, spaced approximately logarithmically. The nearest is normally less than 20 ft (6 m) from the well.

96. Except in hard rocks with clean discontinuities free of silt and gravel, the inflow zone of the pumping well should be protected by perforated casing or a well screen. Well screens may be of plastic, steel or other material, and have fine horizontal or vertical slots. For deep wells, techniques exist for perforating casing after emplacement. A gravel pack should be installed around the well screen to act as a filter.

97. The rate of flow of groundwater in response to a given hydraulic gradient is dependent upon permeability of the aquifer. The rates of increase in drawdown and of discharge are used to calculate permeability. Because wells are commonly used for dewatering, the well test is a useful method of assessing drainage potential of a slope.

98. The successful location, execution and interpretation of well tests requires considerable experience; the employment of experienced well drilling contractors and a hydrogeologist familiar with local conditions is recommended. Particular points requiring special consideration are the position and spacing of observation wells, the pumping rate and the selection and aperture size of well screens. Well tests are described in detail in Appendix D.

99. As previously indicated, permeability measurements can be combined with groundwater pressures by a number of means to predict the effect of groundwater on stability. These means are discussed more fully in the next section. The methods of analysis can also be used to predict flowrates for the estimation of pumping requirements.

<u>Observation of Overall Hydraulic Response</u>

100. Two methods of predicting overall hydraulic response have been described. The first combines local permeability measurements at a large number of locations - eg, by falling head tests - with knowledge of the geological conditions. The second uses well tests to measure properties relating to larger rock zones, and predicts long-term response using mass permeability values.

101. In complex geological environments, it is extremely difficult to assess accurately the overall response of a rock mass from tests which apply to limited zones, because of the need to predict quantitatively the effect of faults, changes in joint frequency, sporadic development of clay on bedding planes, etc. A test procedure which will subject a very large volume of the rock mass approaching a sector of the mine in size to changing groundwater conditions is needed.

102. Piezometric observations and recorded discharges can be used to interpret full-scale and long-term conditions. This testing approach can be applied to exploratory adits, to trial open pit excavations or to the initial operations of an open pit.

103. To be discussed later in the chapter are the significance of trial excavations, and of extensive monitoring in the early stages of pit development, for checking design predictions, for firming up design of slopes and drainage methods, and for predicting long-term developments.

104. A large-scale method of assessment which can be carried out during the pre-mining phase is to instrument an exploratory adit prior to and during its development, to measure flows and changes in groundwater pressure. The results may be used to calculate overall "average" mass permeabilities and will indicate the volumes of water likely to be encountered at various stages of mining and show the response of the rock to drainage methods. Groundwater studies using an adit are described in Appendix E, and a practical example is given in reference 12.

105. Overall patterns of hydraulic response are greatly affected by two major geological conditions. The first is differing permeability

characteristics due to different lithologies which have already been discussed and are typified in Fig 5. This is common in sandstone/shale, sandstone/coal/clay and limestone/marl series.

106. The second important factor is the presence of major transgressive geological features, of which the following are typical:

a. gouge-filled faults and seams tending to form impermeable barriers within rock masses;
b. shatter zones and brecciated faults forming highly permeable aquifers within a rock mass;
c. dykes or other intrusive zones having a different permeability than the host rock;
d. solution channels;
e. man-made heterogeneities in the form of grouted cut-offs, stressed foundations, etc.

107. All these features fall into two classes (16):

I. those that are permeable relative to the rock mass, ie are water carriers, and
II. those that are impermeable relative to the rock mass, ie are water barriers.

108. In relation to slope stability, class I structures provide good natural drainage, especially if close to the toe of the slope. However, if they occur close to the ground surface, they will tend to continually recharge the lower slopes, leading to adverse stability conditions and making drainage difficult. Large-scale class I structures may result in a high continuous inflow and require large pumping capacities. In underground work, such structures are usually hazardous, causing high inflows and local instability.

109. Class II structures are unfavourable with respect to slope stability if they are close to the slope face and inhibit groundwater seepage from the face. They may be beneficial in inhibiting seepage from water storage areas or from permeable strata remote from the slope face.

110. Class I structures can be investigated by means of piezometric observations within the permeable feature during changing groundwater conditions eg, during pumping tests.

111. Class II structures are generally difficult to interpret since their impermeable nature over a large areal extent cannot be readily determined from a limited number of drill hole observations. Tests involving adits, wells or the accurate measurement of pressure with depth during drilling are most often required. Because of the scale and cost of such tests, specialist assistance may be advisable for their design and execution.

112. Examples of well and adit tests to determine the continuity of large-scale impermeable structures are illustrated respectively in Fig 19 and 20. Practical considerations relating to adit tests for determining mass permeability and drainage potential of a large rock mass are given in Appendix E.

ANALYSIS OF GROUNDWATER DATA

113. Groundwater analyses are primarily used for determining the groundwater pressure distribution within a slope from a limited number of permeability and piezometric observations. They provide a means of interpolating between or extrapolating from locations where the groundwater conditions are known.

114. The normal approach to analyzing groundwater conditions is by constructing a model or analogue of the slope in terms of the geological structure and the hydraulic properties of the rock units. Known or estimated piezometric heads are then applied at a number of points. The overall potential distribution is determined and used as input to the slope stability analysis to assess the influence of groundwater pressure on stability. They are also used for the design of dewatering or drainage systems.

115. Models can be either physical or mathematical, and represent the flow of groundwater in terms of some physical quantity eg, electric current, or in terms of a mathematical expression (eg Darcy's law).

116. Analytical methods suitable for the study of groundwater flow and pressure in rock slopes are:

a. graphical flow net sketching
b. electrical resistance analogues
c. hydraulic models of rock masses, and

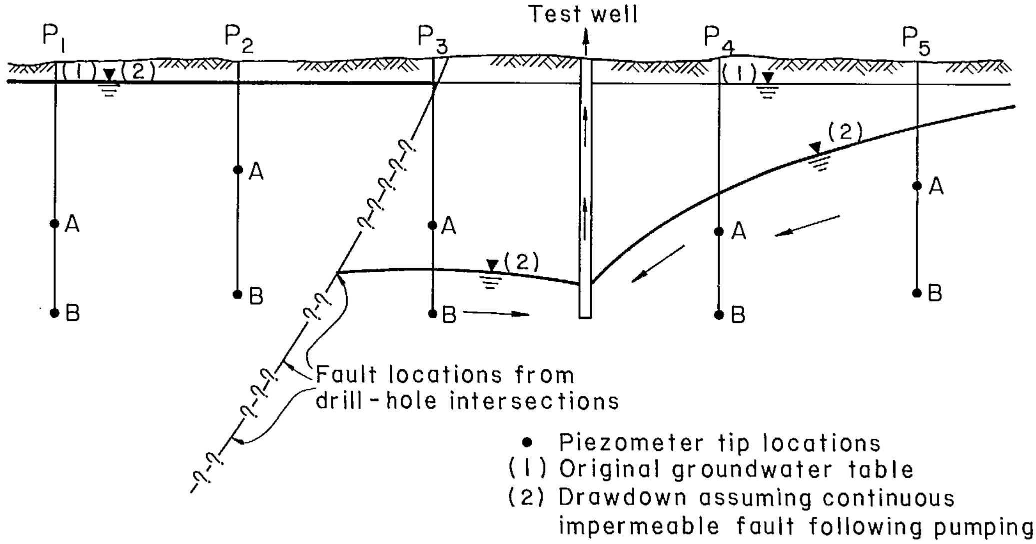


Fig 19 - Well installation to determine the hydraulic characteristics of a fault.

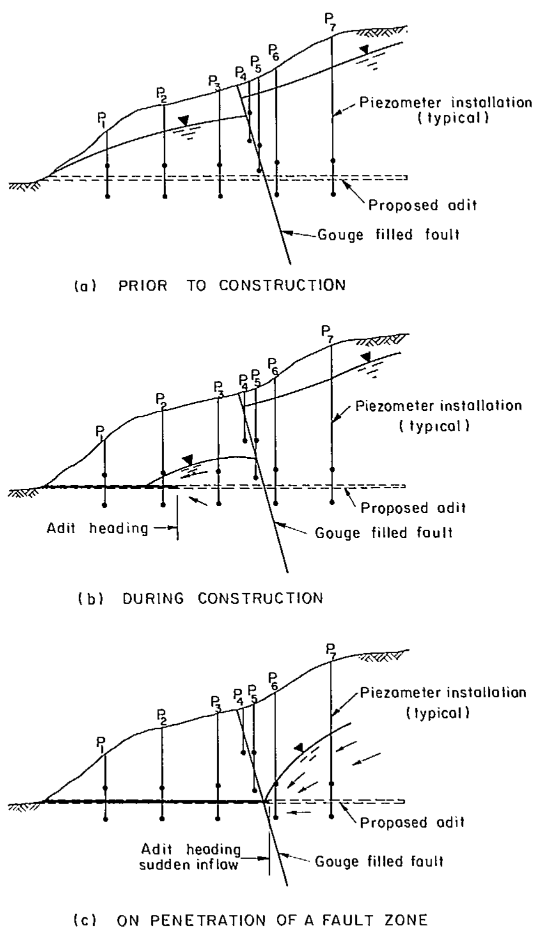


Fig 20 - Piezometric network to determine fault characteristics during adit testing.

d. numerical analyses using digital computers.

117. The majority of cases likely to be encountered involve steady state flow (invariant with time). Methods of solving groundwater flow problems that are time-dependent are available but are beyond the scope of this chapter. Where, for example, a rapid rate of excavation is planned in predominantly slow draining material, time dependent analyses should be carried out, using assumptions on best and worst conditions. These will produce upper and lower bounds to the groundwater conditions with respect to stability. Under such conditions, monitoring during excavation is of primary importance as it indicates the extent to which assumptions conform to actual field conditions.

118. A further important factor to be considered is the transient condition resulting, for example, from heavy surface recharge such as from melting snow. Transient conditions result in flow in zones of the slope which are normally unsaturated and may call for special drainage measures for portions of the slope above the normal groundwater level.

119. Analysis of groundwater data can be carried out with respect to either a horizontal plan or vertical cross sections. Plan modelling is particularly useful in the early stages of investigation, eg, to indicate regional groundwater flow patterns, likely inflows into a pit from different directions, and for special studies involving nearly horizontal groundwater movement. Measured phreatic surface elevations are commonly used as input data.

120. Most methods of slope stability analysis require vertical cross sections to be considered. Results from horizontal modelling may be transformed into vertical co-ordinates or the required data may be obtained directly from an analysis through a particular cross section.

121. The methods used for modelling are equally applicable to either the plan or cross section approach and are considered in the following sections. In summary, the best approach currently available for other than very simple problems is digital computing by the finite element method.

122. All methods of groundwater analysis depend on solving the Laplace equation for two-dimensional fluid flow through porous media, which is derived from Darcy's law (para 25) and the law of continuity of flow, ie, that the net flow into a given volume is zero. Although flow through rocks takes place in discrete channels or discontinuities rather than as uniform seepage, practical considerations relating to the scale of overall groundwater flow through rock masses allow a porous media approach to be adopted.

Graphical Flow Net Sketching

123. As described in para 37 to 39, a flow net is a grid formed by the intersection of flow lines and equipotentials. Graphical flow net sketching is a useful method of analysis in homogeneous isotropic rock masses and can be used to determine both flow rates and groundwater pressures. The method can be applied to inhomogeneous anisotropic seepage studies, but for these more complex problems it is often more practical to use digital computing methods.

124. Some practice is usually required for successful flow net construction. The technique is described in Appendix F. The method is useful for approximate estimates of flow and does not require complex equipment.

Electrical Resistance Analogues

125. Analogues constructed using conducting paper, resistive ink or discrete electrical resistances have been widely used in the solution of groundwater problems. They depend on the analogy of Ohm's law governing electrical flow to Darcy's law, ie,

Current		Potential difference		Conductance	
I	=	V	.	(1/R)	Ohm's law
q	=	i	.	k	Darcy's law
flow per unit area		Hydraulic gradient		Permeability	

Note that $q = Q/A$ (para 25).

Conducting Paper Analogues

126. The simplest conducting analogues use a conducting paper called Teledeltos, which has a uniform conductivity and can be cut to any shape. A typical arrangement is illustrated in Fig 21, and further details are given in Appendix G.

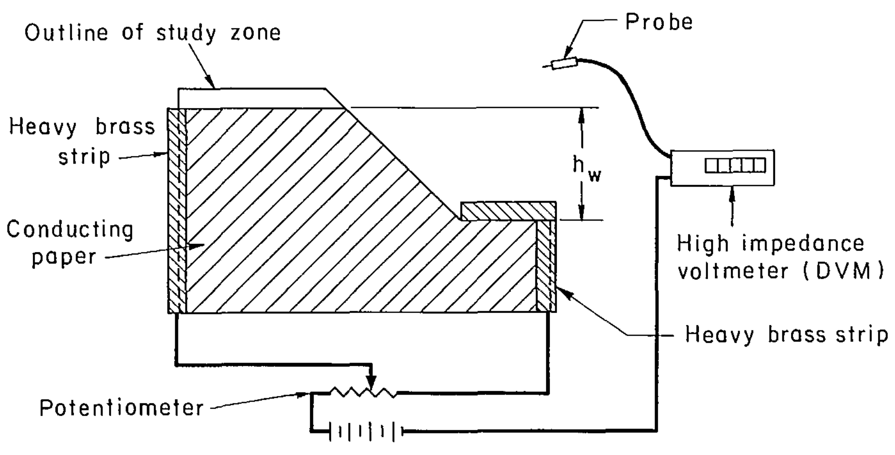
127. The method is best suited to the study of groundwater flow in isotropic materials, although straight-forward anisotropy can be modelled if the cross section is transformed before the paper is cut to shape, as described in Appendix F.

Resistive Ink Analogues

128. Another simple and quick analytical method, which has considerable versatility, uses a resistance grid drawn with ink containing a colloidal suspension of graphite. The grid can be drawn in arbitrary directions and can be readily adapted to the study of flow through jointed rock.

129. By changing the thickness of the lines the resistance of the grid is varied, so that different permeabilities can be modelled. Varying the spacing or orientation of the grid lines allows non-homogeneous conditions to be modelled. The main problem with the method is controlling the resistance or thickness of the lines and some drafting practice is necessary to obtain consistent results. A typical application of this method is illustrated in Fig 22.

130. Once the drafting has been completed, the equipotentials are determined in the same manner as for conducting paper analogues using similar



h_w is the height of the undisturbed phreatic surface

Fig 21 - Conducting paper analogue.

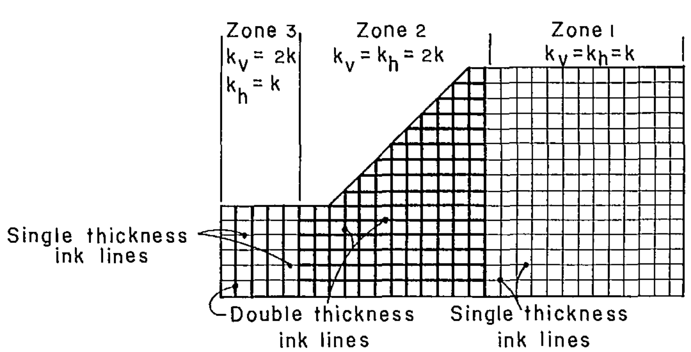


Fig 22 - Resistive ink analogue for non-homogeneous slope.

equipment (Appendix G).

Discrete Resistance Analogues

131. In its simplest form, an electrical resistance analogue consists of a grid of fixed resistances which represent the permeability conditions of the zone under study, as shown in Fig 23. If the solution assumes an equivalent porous media, then the resistance used represents the effective permeability in a given direction of a unit volume of material. If the resistance element is used to model joints directly, then the resistance value must correspond to the hydraulic conductivity of the joint.

132. In constructing a resistance model for flow through rock masses, the resistances are generally arranged on a regular grid within the boundaries of the model. When modelling joints, it may be preferable to use one resistance element to represent several joints near the model boundary. The network spacing should be graded as shown in Fig 24. This enables the most sensitive zone of the grid to represent the important zone immediately behind the slope where the flow pattern changes most rapidly.

133. A given volume of the rock mass is represented by an element of width W and length L, having an equivalent resistance, R, given by:

$$R = C_r \cdot \frac{L}{W}$$

The effective width, W, is derived from the average of the two adjacent network spacings (Fig 24). C_r is a scaling constant, relating resistance to permeability. For inhomogeneous models, the value of C_r will vary with location.

134. For given boundary conditions, the potential distribution is independent of the absolute resistance value. Any convenient resistance range can therefore be chosen. To keep currents to a reasonable value and to prevent excessive heating, an average resistance of

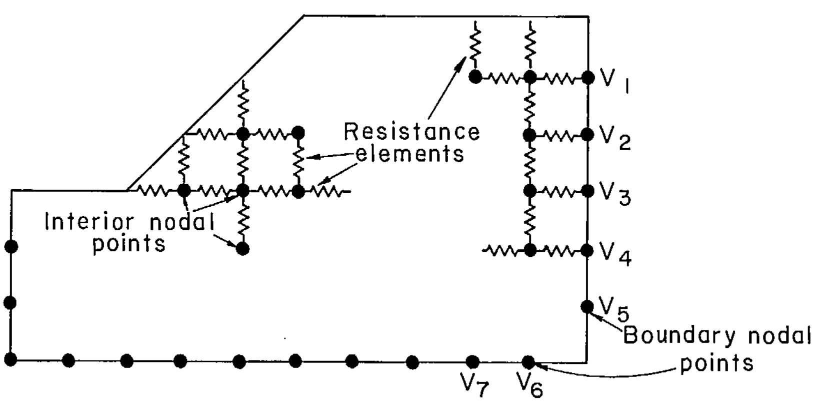


Fig 23 - Basic electrical resistance analogue.

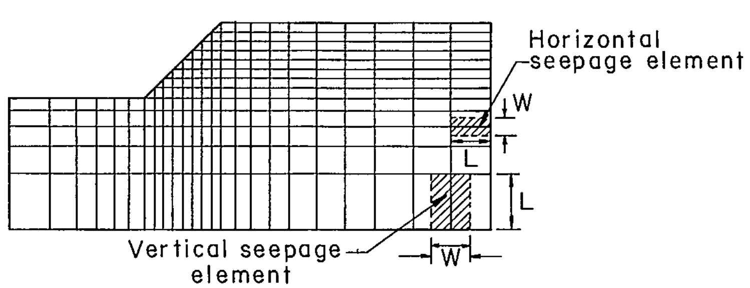


Fig 24 - A simple graded network for an electric resistance analogue.

1000 ohms is desirable for supply voltages within the range 0-10 V. A high impedance potential-measuring device (digital voltmeter) will only be required, since the currents will only be of the order of several milliamps.

135. Once the resistance grid has been laid out, usually on a scaled geological section, test work can proceed. Voltages are applied at the upstream boundary, at the downstream boundary beyond the slope toe and, if required, along the lower boundary of the model to represent the known or assumed water pressure conditions at the boundaries. Known piezometric pressure values within the slope zone can be represented by fixed voltages.

136. The groundwater surface, or water table, is determined by inspection, by successively disconnecting resistances in the upper slope zone until the correct boundary condition is established, ie, hydraulic head is equal to the elevation. The potential distribution within the rock mass is then obtained by determining the voltages at the intermediate nodal points. Flows are computed from the potential difference, equivalent to the head gradient, across an element and its resistance. The flow across a boundary such as the slope face can be determined from a summation of such measurements.

137. In all electrical resistance analogues, drains can be modelled by inserting a very low resistance at the drain positions. The primary advantage of analogues is that they allow the operator to be in direct communication with the problem. Feedback is thus rapid, and continuous trials and developments can be carried out. The primary disadvantage of discrete resistance analogues is that the cost of obtaining adequate versatility and capability of dealing with large complex problems can rarely be justified, except for specialized research and teaching applications. For complex investigations in mining, digital computer techniques, described below, are generally best.

Hydraulic Models of Rock Masses

138. Although used as a teaching aid, hydraulic models are inflexible and cannot readily be adapted to solving practical problems.

Numerical Analysis by Computer

139. Digital computing techniques are widely used in the study of seepage through soil and rock slopes. A reasonable degree of computing expertise is required, and detailed studies need input from both engineering and computing specialists. The approach has few limitations and is particularly suitable for complex geological conditions.

140. Two methods are generally employed, ie, the finite element and the finite difference methods. Although the formulation of each method is different, their capabilities are similar and both can be extended to solve time-variant problems. The finite element method is discussed below, and a description of a computer program is given in Supplement 4-1.

141. The finite element formulation solves the governing field equations for fluid flow by representing the flow region by discrete elements, and expressing the field solution in terms of the solutions at the element nodes. Interpolation functions are used to relate the field with the nodal parameters. The resulting equations can be solved by direct or iterative matrix inversion procedures.

142. The slope is divided into finite elements. The number of elements depends on the complexity of the slope being analyzed. An example is shown in Fig 25. This subdivision is purely mathematical, with the potential at any point within an element being expressed as a function of location and of the potential at the nodes associated with that element. The continuity equation, written in differential format, can be shown to be equivalent to the minimization of an associated integral and it is from this theoretical basis that the finite element procedure is used to carry out an approximate minimization process.

143. The set of equations formulated by this process represents the discretized form of the continuity equations at each of the chosen nodal points. In general, high element densities should be used in regions of high flow rate, ie, high

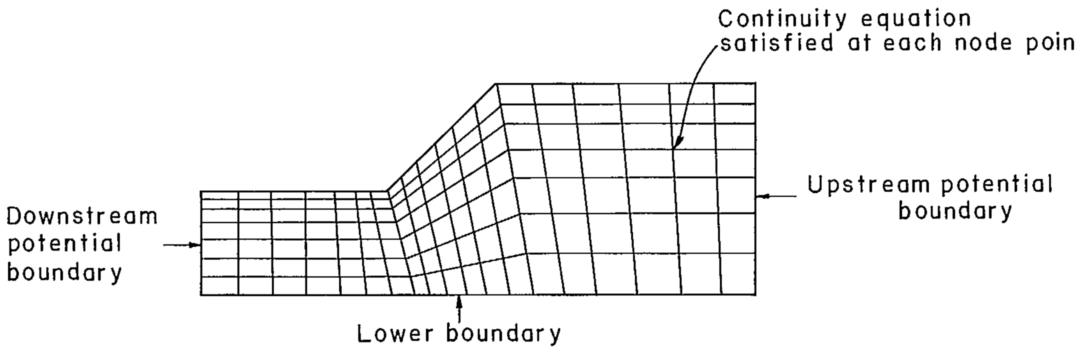


Fig 25 - Typical finite element mesh.

potential gradients, and lower concentrations of elements should be employed in other regions. This gradation of mesh size is a feature which is facilitated by the finite element procedure.

144. To determine the location of the free surface or groundwater table, the nodal points in the upper part of the slope are inspected to determine if the hydraulic potential is greater than or equal to the elevation potential. If this condition is not satisfied, the relevant part of the mesh is removed from the computation and the procedure is repeated until the true location is determined. This procedure is complex in instances where low flow values occur or where there is a nearly horizontal free surface, and considerable care is required.

145. The input to the program consists of the following:

a. mesh geometry, often generated by a separate subroutine;
b. horizontal and vertical permeabilities at given locations in the mesh (other orientations are possible);
c. boundary conditions (upstream, downstream and lower boundary);
d. internal points having a specified potential or flow;
e. drain locations.

146. The output from the analysis is a grid of nodal potential values which can be contoured using a suitable contouring program to give a plot of equipotentials, such as the two shown combined in Fig 26 which represent pre- and post-drainage conditions.

INFLUENCE OF GROUNDWATER ON ROCK SLOPE STABILITY

Modes of Influence

147. The primary influence of groundwater on the stability of rock masses is reduction in the shear strength of joints acted upon by water pressure. The governing principles are explained in para 44 to 49.

148. The shear strength of a potential sliding surface within a slope is estimated from the shear strength of the rock and the distribution of groundwater pressures over the surface. The exact manner of accounting for the reduction in strength due to water pressure will depend on the kind of stability analysis used. Those considered in the design section of the manual (chapter 5) are described in the context of groundwater in para 164-176.

149. In addition, groundwater can also reduce stability by creating hydraulic driving forces within steeply dipping joints where the faces are not in contact such as at tension cracks, and for which shear strength is therefore not a consideration (18,19).

150. The two conditions are illustrated in Fig 27, and care must be taken in slope stability analysis to decide which of the mechanisms is appropriate to any given surface defining an instability.

151. Tension cracks often occur because of poor blasting practice or as a result of overall

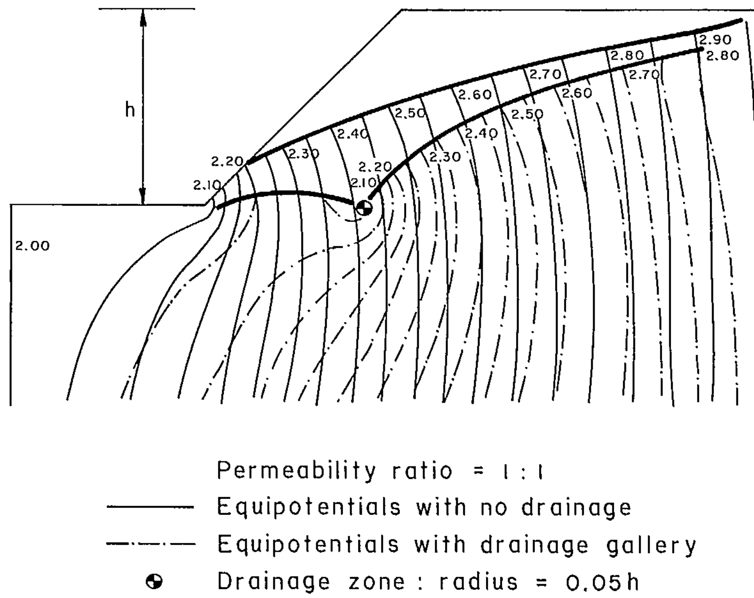


Fig 26 - Equipotential plots determined from finite element analysis.

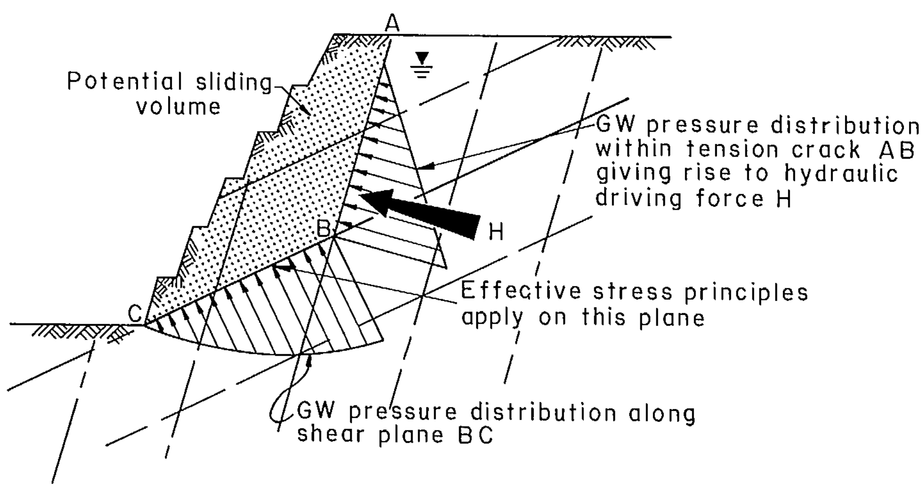


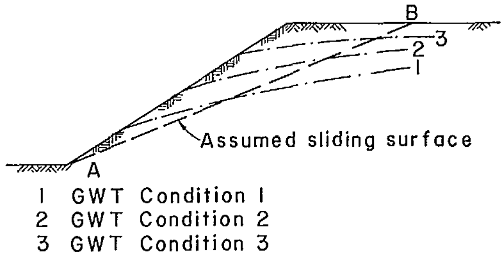
Fig 27 - Influence of groundwater pressure on rock slope stability.

deformation of the slope. Prevention of recharge into such cracks is obviously advantageous and suitable methods of doing so are described in para 266.

152. In assessing the effect of groundwater on stability, the influence of water pressure on the shear strength of sliding surfaces should first be determined. After this has been taken into account, hydraulic driving forces acting across tension features ie, where shear strength is not relevant, should be calculated and allowed for in the analysis. For a given discontinuity only one or other of the effects need be considered.

Effect of Geological Conditions

153. The effect of geology on the overall hydraulic response of a rock slope has been described in para 105 to 109. Examples of the effect of geological structure on groundwater pressure distribution and therefore on slope



Condition	Description	Permeability ratio	Relative stability index
1	Isotropic slope	1:1	1.00
2	Anisotropic slope	10:1	0.93
3	Anisotropic slope	10:1	0.45

Assumed strength parameters on sliding plane $\phi' = 26.5°$, $c' = 0$

Fig 28 - Effect of anisotropy on slope stability.

stability are now given.

Anisotropic Conditions

154. The effect of anisotropy of permeability due to orientation of bedding, for example, on stability is illustrated in Fig 28. Significant variations in stability are evident for various permeability ratios, assuming the same rock discontinuity strength values in each case.

Impermeable Faults/Seams behind Slope Faces

155. The occurrence of relatively impermeable and continuous faults within rock slopes can have a dramatic influence on both flow and pressure conditions. Such low permeability faulting often occurs in association with orebody formations and is thus of particular relevance to open pit slopes.

156. The simplest and potentially most adverse geological condition exists when the fault forms a potential sliding surface within the slope as shown by the idealized example in Fig 29. The stability of the slope in this case is influenced first by the potentially weak nature of the fault material and second by the adverse groundwater conditions that may build up behind the fault itself. The groundwater conditions for the case of the faulted and unfaulted rock mass are compared.

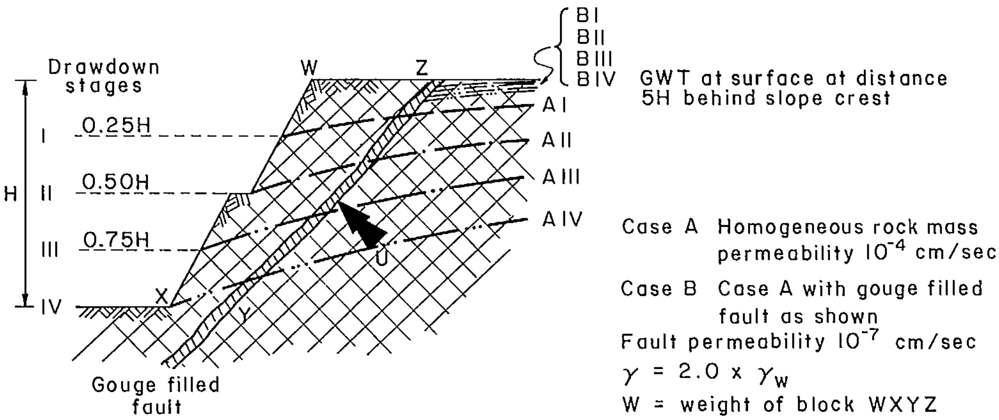
Influence of Stressed Zones within Rock Slopes

157. The hydraulic conductivity of fissures is very sensitive to changes in joint opening (6); the tightness of fissures depends on the applied stresses. Of particular importance to rock slopes is the effect of stress concentrations in the toe area leading to a potential decrease in rock mass permeability and a more adverse overall groundwater condition than in higher regions of the slope. This effect should be considered in the analysis. If shear movements have occurred in a slope, permeability may be increased due to normal dilation across rock joints.

Influence of Recharge and Discharge

158. As already mentioned, transient groundwater conditions can significantly affect the stability of slopes. If the recharge into a slope at a given time is greater than the discharge, a build-up of water pressure will occur with a consequent reduction in stability. To reduce groundwater pressures within rock slopes, the recharge can either be minimized by surface sealing/diversion etc, or the discharge can be increased by drainage.

159. Typical recharge sources into rock slopes are illustrated in Fig 30. Recharge due to precipitation can give rise to near-surface transient pressure conditions which can



Drawdown stage	Hydraulic force acting on plane ZY = U		
	Case A (U_A)	Case B (U_B)	$\frac{U_B}{U_A}$
I	0.58 W	0.83 W	1.43
II	0.43 W	0.80 W	2.35
III	0.12 W	0.79 W	6.58
IV	0.01 W	0.78 W	78.00

Fig 29 - Influence of a low permeability fault on the hydraulic forces developed within a slope.

substantially reduce stability. The other forms of recharge illustrated normally give rise to more steady state seepage conditions. During winter, recharge may be limited due to ice or snow cover. During the spring runoff, however, considerable recharge can be anticipated owing to the quantity of surface water that accumulates, particularly behind the slope crest. Slope instability in the spring runoff period is common.

160. The possibility of sudden instability of rock and overburden slopes should be considered after prolonged rain or snow melt. If necessary, the transient response should be measured using rapid response piezometers ie, having very low volume demand, taking frequent readings during heavy recharge and for sufficient time afterwards to define the peak value of piezometric pressure (Appendix A). Alternatively, approximate solutions can be determined based on assumptions, eg, slope fully saturated, etc.

Influence of Face Freezing and Permafrost

161. Groundwater seeping from the face of slopes may freeze during periods of below freezing temperatures, preventing further discharge of the groundwater. An impermeable barrier is thus formed, causing a build-up of groundwater as shown in Fig 31. As the groundwater overtops the existing ice barrier, it too freezes and the barrier increases in height up the face of the slope. This condition leads to highly detrimental groundwater pressures within the slope and many seasonal soil and rock slides can be attributed to this effect.

162. Permafrost is permanently frozen ground which occurs widely in Northern Canada. Frozen ground has considerable strength; however, stability problems can arise when mining in permafrost because operations may cause local thawing, sometimes with drastic loss of strength and consequent stability problems.

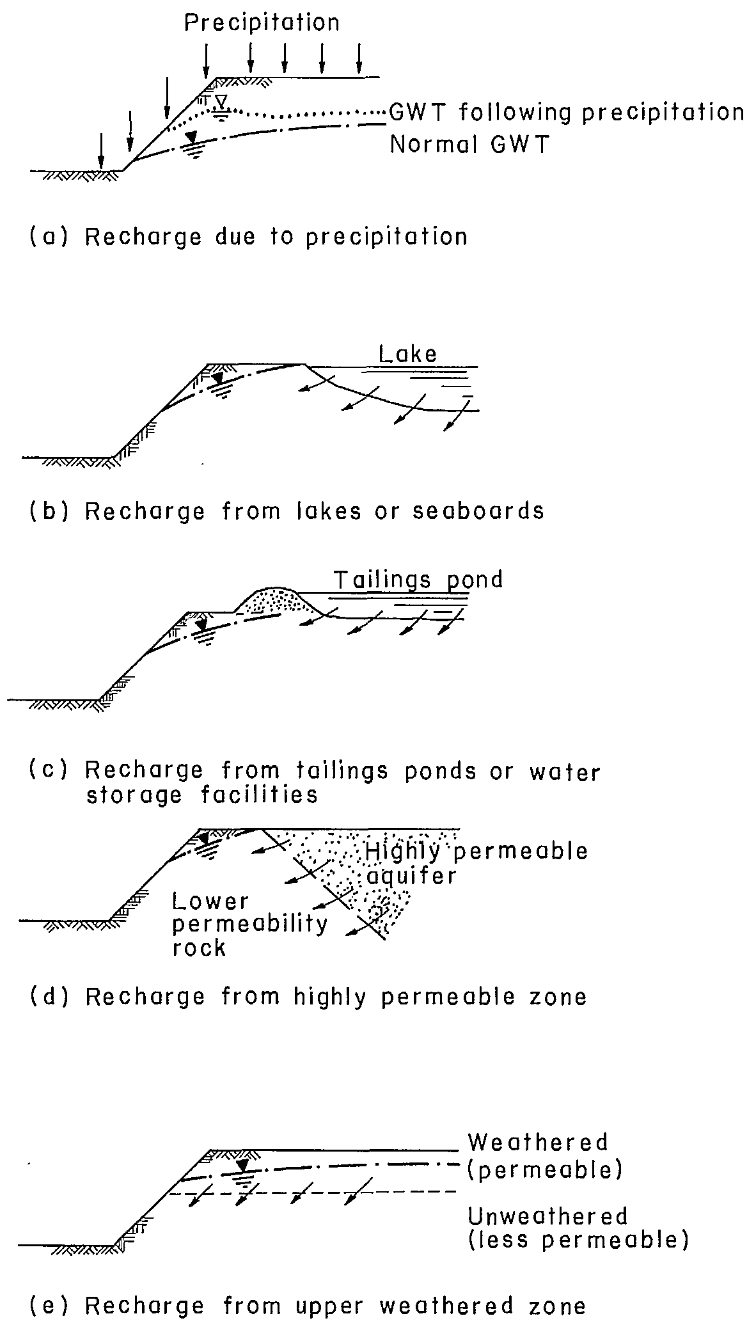
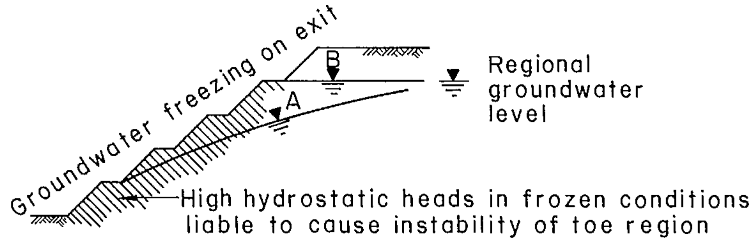


Fig 30 - Recharge sources causing adverse groundwater conditions within slopes.

163. Groundwater flow - and hence groundwater pressures - can be considerably affected by permafrost. This can lead to unexpected drainage problems. There is little documented information to guide mining in general, and stability analysis in particular, in permafrost. The experience of two Canadian mining companies is described in Appendix I.



A-Phreatic surface for natural drawndown or utilizing a protected drainage scheme

B-Phreatic surface caused by face freezing

Fig 31 - Influence of face freezing on slope stability.

GROUNDWATER INPUT TO DESIGN

164. The purpose of groundwater investigation and analysis is to evaluate the effect of groundwater pressure on slope stability. The analysis of slope stability includes the influence of groundwater either as a driving force (para 149) or more importantly as a factor reducing shear strength (para 44-49).

165. The various instability modes and appropriate methods of analysis are described in the Design chapter. The methods all require the determination of water pressure on the potential surface of sliding. This water pressure is used to determine either the hydraulic driving force or the reduction in normal stress, and therefore the reduction in shearing strength.

166. The various instability modes and the nature of the required groundwater input are described below.

Rotational Shear

167. Fig 32 shows schematically the rotational shear instability mode. The stability of the sliding mass is analyzed in two dimensions by assessing the forces tending to produce sliding

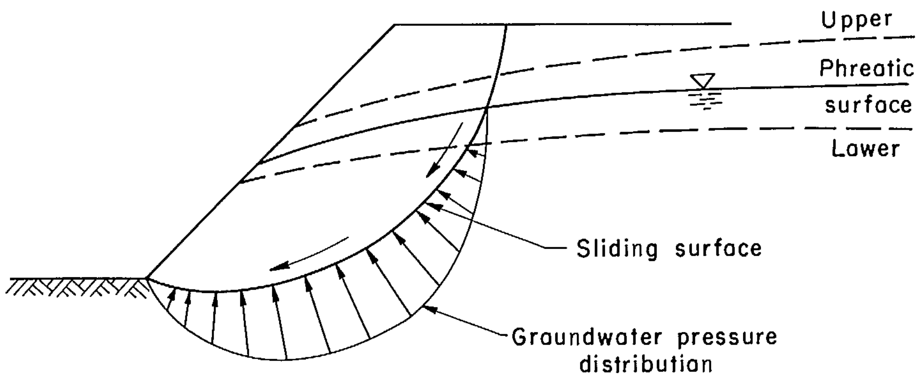


Fig 32 - Groundwater input to analysis of rotational shear sliding.

and the shear strength on the surface of sliding. The groundwater pressure distribution is determined from analysis of the groundwater conditions; in the method given in the Design chapter, the pressure is approximated by the vertical depth from the phreatic surface to the sliding surface. A probability analysis is carried out; this requires an estimate of the range of peak phreatic surface levels. That is, the range of phreatic surfaces to be considered in design is determined. Typically, slope design might be based on the highest groundwater level occurring during the year; this usually occurs following spring runoff. If the slope is designed for, say, 20 years, 20 potentially different peak levels should be considered. The engineer must determine the highest and lowest values for these peak phreatic surface levels, by considering ranges of precipitation, permeabilities and other natural factors. He may also allow for inaccuracies in measuring and analyzing groundwater flow.

168. The highest and lowest estimates of the peak phreatic surface are determined using the field investigation techniques and subsequent analyses to obtain flow nets, as described above and in the appendices. The input to the design process consists of these phreatic surfaces, defined by a sequence of points on the surface.

Simple Plane Shear

169. Figure 33 shows the simple plane shear instability mode. A two dimensional analysis is used; the driving forces, eg, weight, and the resisting forces or shear strength on the surface of sliding are determined, and used to assess the stability of the sliding block. As in the rotational shear analysis, a probability analysis using the highest and lowest estimated peak phreatic surfaces can be carried out. Input to the design process consists of the coordinates of points on a mean phreatic surface, with the variation expected above and below the mean at each point.

Complex Plane Shear

170. A typical wedge or complex plane shear mode is shown in Fig 34. Groundwater pressures act to reduce stability in this mode as in all others, but determining the pressure distribution is more complex than in the 2-dimensional analyses described above.

171. Two distinct modes are considered in the wedge analyses of the Design chapter - the case with a tension crack and the case without - as shown in Fig 34(a) and (b). The groundwater pressure distributions assumed in the analysis are shown for the two cases in Fig 34(c) and 34(d).

172. For the simple wedge, a bi-linear phreatic surface is assumed with peak water head at the mid-point of the line of intersection (the junction of the two planes defining the wedge). Alternatively, an impermeable face - implying a build-up of water pressure - can be considered as shown by the broken line in Fig 34(c). The location of the phreatic surface is determined by estimation from the flow net prepared in the groundwater investigation.

173. The analysis for stability with a tension crack assumes water may be present in the crack. This determines the average pressure distribution

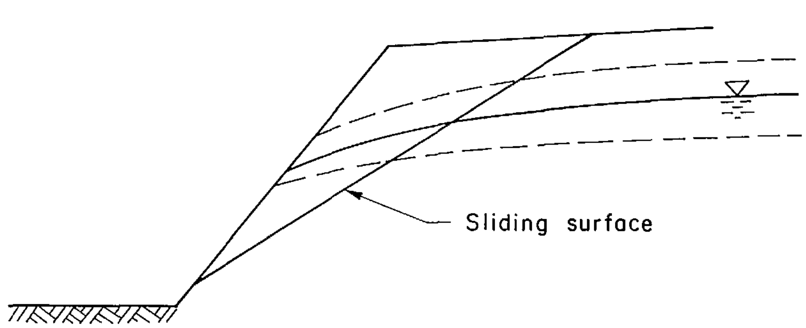


Fig 33 - Groundwater input to analysis of plane shear sliding.

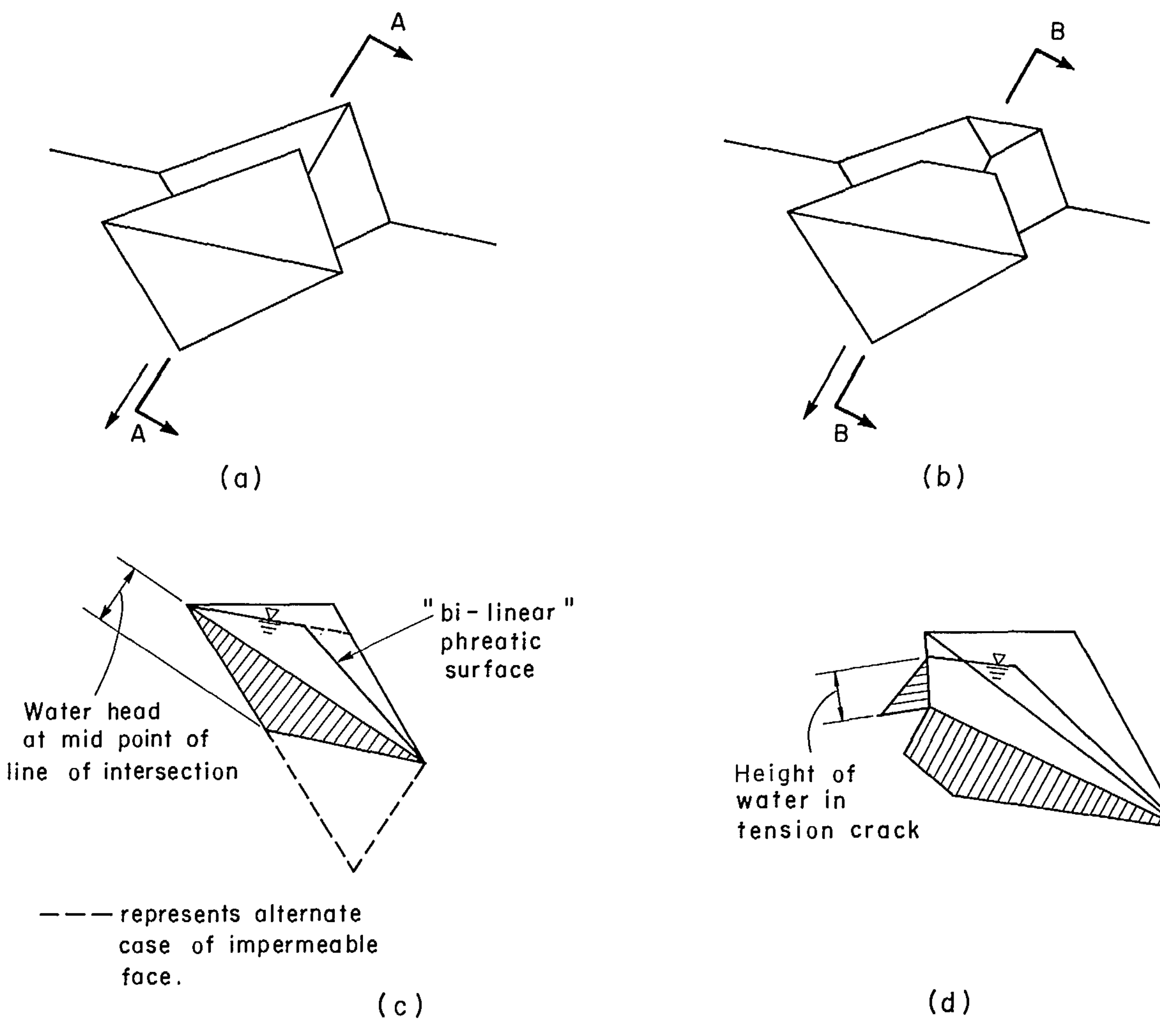


Fig 34 - Groundwater input to analysis of wedge sliding.

on the wedge, as shown in Fig 34(d). The height of water in the crack is determined from measurements in the field and from the flow net prepared in the groundwater investigation.

174. Variation in water pressure - for probability analyses - can be included by specifying the upper and lower limits of the phreatic surface, or of the water in the tension crack.

Multi-Block Instability

175. Multi-block instabilities are analyzed similarly to wedge instabilities (Fig 35). A groundwater pressure distribution beneath and between the wedges is required, and is estimated from the flow net and, if relevant, from water levels in tension cracks.

176. An important consideration in stability analysis is that, as soon as movements have occurred, permeability conditions in the zone of instability may change, and the groundwater pressure distribution will alter. A review of the results of the groundwater investigation may then be necessary.

GROUNDWATER MONITORING

177. The most effective method of groundwater evaluation and prediction is the observation of conditions within pit slopes by a network of piezometers. Results from the initial excavation can often be used as a model for later slope designs.

178. The drawdown characteristics for the

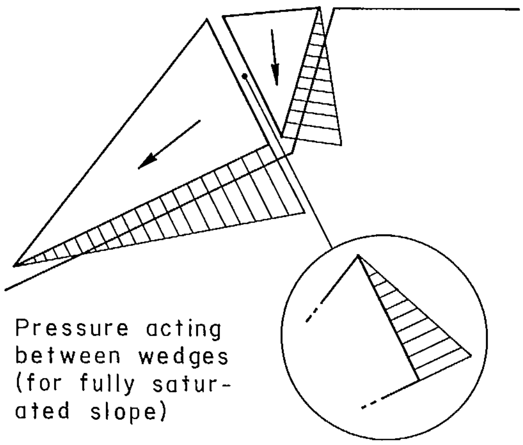


Fig 35 - Groundwater input to analysis of multi-block sliding.

various excavation stages are predicted from initial measurements of hydraulic properties and groundwater pressure conditions. A preliminary slope design is usually based on these predicted values, which must be verified by monitoring as pit development takes place and drawdown occurs. Results from the monitoring program are used to update preliminary designs to form the basis for detailed, long-term designs, and to evaluate slope drainage requirements.

Piezometric Networks

179. Layout for piezometric networks will depend on the geometry and geology of the excavation. The design and use of piezometers have been discussed in para 61 to 70. Piezometers should be located on representative sections around an open pit so that they measure groundwater pressures within and behind the potential instability zones. The number and layout of holes will depend on the following factors:

a. relative importance of groundwater to slope stability
b. complexity of geology
c. scale of the open pit
d. drilling considerations (depth of piezometer installation).

180. The sections should be selected to monitor those zones which, from consideration of regional and other early studies, are considered to be most critical to stability, eg, in the major areas of recharge, in the weakest rock types, where the steepest or highest slopes are planned, and where major geological features which would be detrimental to groundwater pressure distributions are anticipated.

181. A series of holes drilled from behind the pit crest and preferably inclined towards the pit bottom should be adequate for initial observations. These can then be supplemented by additional holes on the same radial sections both in and out of the pit to determine the variation in groundwater conditions throughout the section.

182. As early installations are lost through development, a program of replacement should be followed to ensure continuity of monitoring. Sufficient long-term installations should be planned to ensure this continuity throughout the pit life. New installations should be made in adequate time to establish background conditions before pit expansion affects the readings.

Flow Measurements

183. Overall flow measurements from open pits or specific flow data from benches, drain holes, tunnels etc, are useful measures of the permeability of the rock mass and of how groundwater conditions change with seasons and with pit depth. Consistent measurements with time, corrected for surface effects, are thus required. These can be correlated with climatic conditions and piezometric levels. Flow measurements can be carried out by:

a. localised measurements using a calibrated volume and timing device,
b. pumping rates, both instantaneous and overall,
c. weirs, V notch or venturi flumes.

Overall Surveys

184. Overall surveys of piezometric and flow data should be carried out routinely. Typically, such surveys are used to produce piezometric contour plans of the whole excavation and piezometric distributions within slopes. These data can then be used to establish trends for predicting future conditions and to indicate where additional monitoring installations are required.

PART II: PRINCIPLES AND METHODS OF DRAINAGE

185. The adoption of drainage implies that a slope is required to stand at an angle approaching the maximum for the soil or rock mass concerned. Drainage, in reducing the detrimental effect of groundwater, can either increase the stability of existing slopes or allow slopes to be steepened while maintaining the same degree of stability.

186. The cost of a drainage scheme must be considered against the benefits of drainage. These are of two types: benefits from increased stability through reduced groundwater pressures, and benefits from operating factors such as reduced blasting cost with dry holes. The benefits from increased stability can be appraised by determining the expected distribution of groundwater pressure, and using this in the design process. The benefits are then determined by cost-benefit analysis (Chapter 5). Benefits from operating factors must be assessed directly by the operator.

187. There is growing recognition of the importance of cost-benefit appraisal of drainage in the design of all slopes. However, at present, drainage is used predominantly as a remedial measure in slopes showing obvious signs of instability. The detection of an unstable condition implies that some deformation of the slope will have occurred and that strength properties of the material will have been reduced. If the need for drainage is recognized, and a drainage scheme is adopted before instability develops, the maximum benefit from reducing water pressures within the slope will be realized, because the rock mass will still possess its undisturbed strength.

188. Before developing a drainage system, sufficient information on the groundwater characteristics of the slope material must be available to allow selection of the optimum system. The cost of the preliminary investigations required to obtain the necessary information is usually justified if the overall slopes are critical to the safety or profitability of the mine. Clearly, in common with other stabilization methods, the absolute stability of the slopes must be defined within close limits to allow the effect of the drainage measures to be fully understood. Although this section concentrates exclusively on the benefits of slope drainage to stability, other operational factors are extremely important and should not be overlooked.

189. When an open pit excavation cuts through the natural groundwater table, groundwater flow

towards the excavation occurs and pit dewatering will generally be required. The resulting drawdown within the slopes leads to natural drainage, which in itself tends to increase stability. The degree and rate of development of this natural drainage should be evaluated by piezometric measurements before any decisions are made regarding additional drainage measures.

190. The design of a drainage system should commence with evaluating the hydraulic properties and groundwater pressure distribution in the rock mass. This is followed by theoretical analyses to evaluate the effects of various drainage designs from which a target design is selected. This target design should be validated by field trials, and the design modified if necessary, before proceeding with the final full-scale installation.

PRINCIPLES OF DRAINAGE

191. The reduction of slope stability caused by adverse groundwater conditions is primarily influenced by water pressure on potential sliding surfaces within the rock mass. The primary object of drainage is to reduce water pressures within the critical zones as far as practicable.

192. To ensure that efficient drainage can occur, adequate hydraulic connection must exist between the slope material and the drainage system. The response time of the groundwater pressures to a drainage installation depends directly on the permeability of the material. In low permeability materials, periods of a year or more may elapse before a steady state "drained" condition is reached. The scale of the drainage measures must also be directly related to the scale of the slopes to be drained.

193. Because flow takes place predominantly through discontinuities in almost all rock types, the efficiency of a drainage system is directly related to the number of discontinuities which it intersects, as illustrated on Fig 36. The efficiency of the drainage system influences both its cost and the degree of improvement in stability, and should obviously be maximized.

194. In some rock types, particularly those susceptible to erosion, the flow is not widespread throughout a given fissure, but is confined to particular flow channels within the fissure. These materials are generally difficult to drain and are characterized by highly variable drain discharges.

195. Drainage of a particular zone is effective only if the discharge capacity of the drainage system exceeds the recharge into the drainage zone. The discharge capacity of a given installation is limited by the effective surface area of the installation.

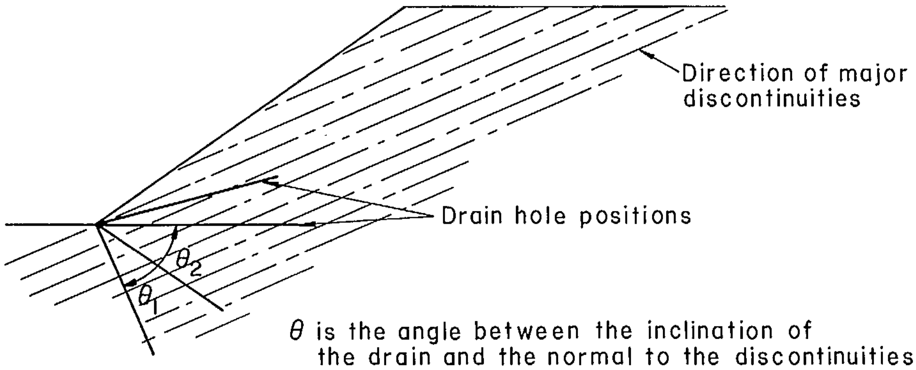
196. Where the slope is non-homogeneous, the drainage system must be adapted to suit the permeability conditions. A drainage system can be rendered inoperative by features such as impermeable strata or faults that have not been perforated by the drains. Therefore, an important design criterion is identification of the position, extent and hydraulic properties of lithological variations and transgressive features.

197. Because the build-up of groundwater pressure within slopes occurs as a result of recharge, all possible precautions should be taken to minimize recharge into the slope zone. Surface water should be collected by means of lined and properly graded interceptor ditches, and diverted or pumped away from the slope. Subsurface water can be prevented from reaching the slope zone by means of impermeable cut-offs or well curtains. These measures are particularly relevant to highly permeable near-surface deposits, such as alluvial overburden or badly weathered surface rock.

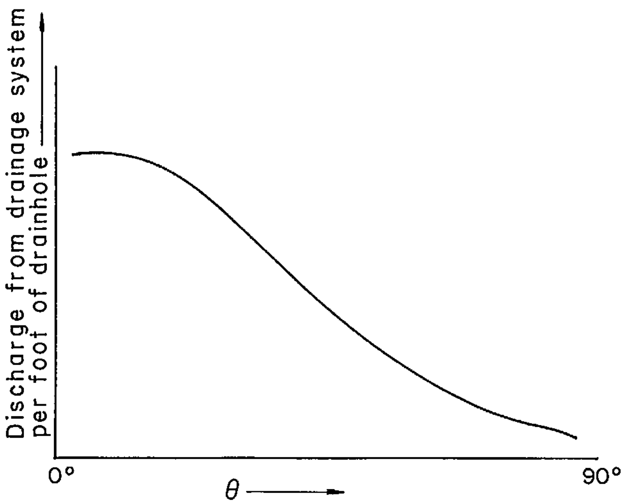
SLOPE ZONE REQUIRING DRAINAGE

198. To define the extent of the rock mass requiring drainage, the most probable instability zone, itself a function of the groundwater conditions, must be determined. A typical zone that can be used for initial comparative studies is shown in Fig 37.

199. The overall height of the slope and the depth to which drainage is required will determine the scale of the drainage system. The practical radius of influence of a given installation depends on the effective discharge area of the drain, the time allowed for drainage to occur, and the recharge and hydraulic characteristics of the



(a) Variation in drain inclination



(b) Effect of drain inclination on discharge

Fig 36 - Efficiency of a drainage system.

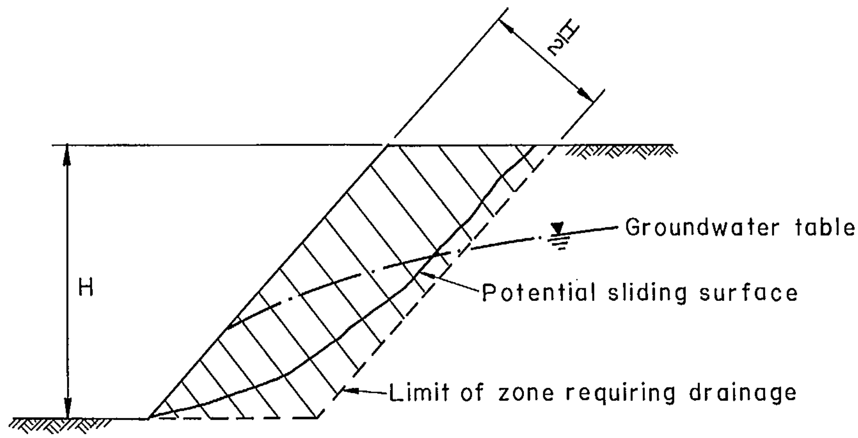


Fig 37 - Definition of slope zone requiring drainage.

rock mass itself. For example, for large slopes it might be necessary to use several hundred small horizontal drain holes instead of two or three levels of galleries.

200. In many rock masses, the permeability or hydraulic conductivity decreases significantly with depth because of closure of joints under increase in stress, and a general increase in joint spacing. In this case, the amount of recharge from the overlying strata to the interior slope regions is relatively high compared with storage capacity of the rock mass. Deep interior drainage will therefore be of limited value because discharge from the drainage system will be small in comparison with the available recharge (Fig 38(a)). Similarly, drainage near the toe of the slope will only provide a limited degree of drawdown owing to the potentially high near-surface flow (Fig 38(b)). Probably the best approach in such cases is to minimize recharge into the main slope region by providing a shallow drainage curtain behind the slope crest. This can be supplemented as necessary by drainage installations on the lower face (Fig 38(c)).

FACTORS AFFECTING DRAINAGE

Permeability and Hydraulic Connection

201. The amount of discharge from any given drainage unit will depend on the hydraulic connection between that drainage unit and the rock mass, as shown in Fig 36.

202. In general, for given hydraulic properties, the better the connection with the rock mass discontinuities and zones of preferential flow, the greater the pressure dissipation at any given distance from the drain. Thus drainage facilities must always be designed to take account of local permeability conditions and to intersect the major flow paths. In practice, many rock masses have such low permeability that drainage is uneconomical. In this respect, the most important items to be considered are the time to achieve, and the scale of, the required drainage.

203. It is difficult to predict the effectiveness of a drainage system before installation; the degree of success depends on the extent to which representative hydraulic parameters for the rock mass have been accurately defined. However, drainage is likely to be successful in rock masses with well developed sets of joints, devoid of infilling and with an equivalent mass permeability of 10^{-5} cm/sec or greater. In these types of material, the response time to a drainage system over a zone of the order of one hundred feet (30 m) in dimension will be less than one month.

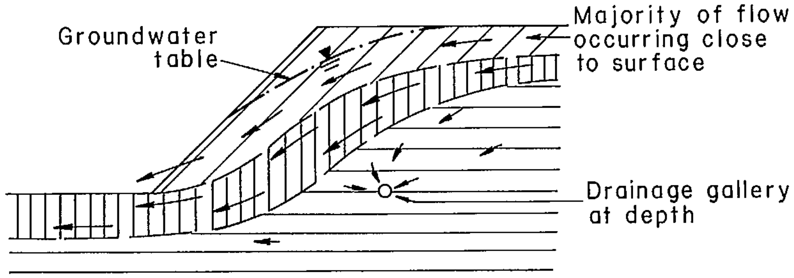
204. In other less permeable rock masses, or where flow occurs through specific channels only, more detailed investigation and some sort of drainage trial will be required before the feasibility of drainage can be established. Although methods exist for draining low permeability materials - typically electro-osmosis and vacuum techniques - the scale at which they can be applied is generally too limited for the stabilization of major soil and rock slopes. In materials with poor drainage characteristics, therefore, it is usually necessary to resort to shallower slope angles to achieve the desired stability.

205. Except in fairly simple geological environments, the potential effectiveness of drainage will need to be assessed by combining measurements of local hydraulic properties (para 57 et seq) with large scale tests or trials of the proposed drainage system on the rock mass to indicate its drainage potential (para 92 et seq).

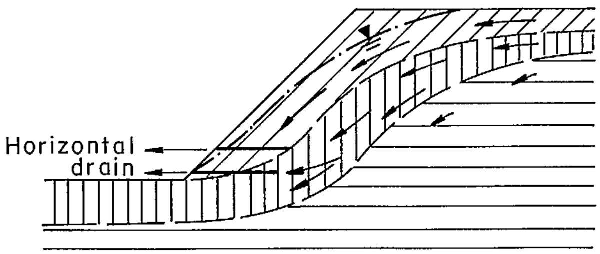
Major Structural Features

206. In geologically complex rock masses containing major structural features such as described in para 106-111, particular attention should be paid to the design of the drainage system. If such features are permeable, they should be adequately intercepted by the drainage system. If the features have a low permeability relative to the surrounding rock mass, they should be penetrated by the drainage system to relieve excess groundwater pressures that may develop behind the structures. Typical examples of good and bad drainage practice are shown in Fig 39.

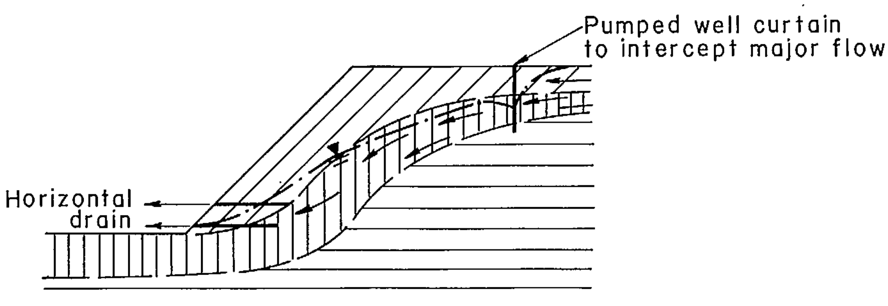
207. When rock adjacent to excavated slopes is subjected to blast damage, the permeability of the rock mass increases significantly and the rock



(a) Poor drainage practice



(b) Poor drainage practice



(c) Good drainage practice

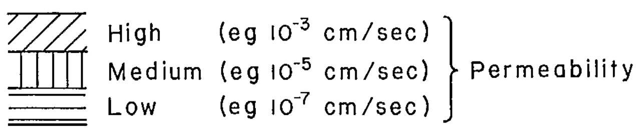


Fig 38 - Influence of non-homogeneity on slope drainage practice.

becomes free-draining. The extent to which fractures open up adjacent to exposed faces is difficult to determine and will vary according to rock type and blasting techniques employed. In addition to providing greater freedom of drainage, blasting will also result in increased infiltration and a loss of strength along joint planes.

NATURAL AND INDUCED DRAINAGE

208. During the initial excavation stages, drawdown due to natural drainage will probably be sufficient to ensure stability because:

- the working slopes may not be as steep as the final slopes;
- the slopes will not be as high as final slopes;
- the groundwater table will be at some depth

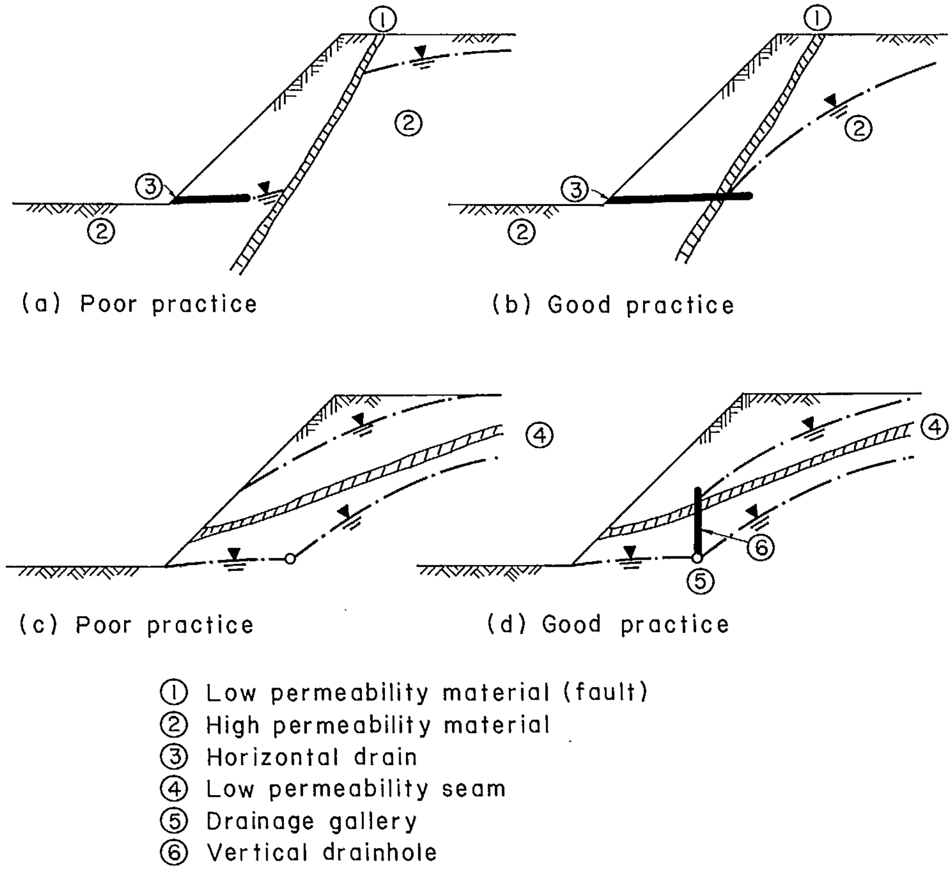


Fig 39 - Examples of drainage procedures for non-homogeneous slopes.

below surface and groundwater pressures developed in the slopes will be low.

209. Conditions become more critical as excavations are deepened and at some stage natural drainage might be insufficient to maintain the required degree of stability. Before this condition is reached, in situ piezometric observations should be carried out to determine the drainage measures required before final slope development, as described in para 177-182.

210. For maximum benefit, drainage of the slopes before excavation is required. In some cases this will determine the method of drainage to be employed. For example, vertical wells or adits are the only possible method for large-scale drainage of an area of low topographical relief prior to excavation.

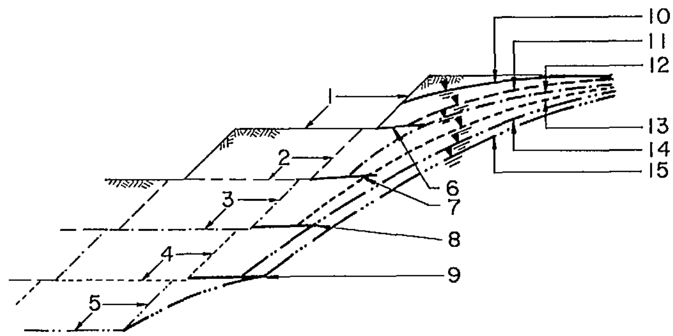
211. If the slope is composed of low permeability materials, adequate time must be allowed following installation for the required conditions to be established. This may involve planning the excavation in stages for the most effective results. An example of this type of development using horizontal drains is shown in Fig 40, from which the time lag in the development of drawdown after excavation is apparent.

DRAINAGE AND DEWATERING METHODS

212. The method of slope drainage most appropriate for a particular set of conditions will depend on the height of the overall slopes, the permeability of the slope material and economic and operational considerations. Slope drainage systems that are currently most widely used are shown in Fig 41:

a. horizontal or nearly horizontal drain holes drilled into the slope face;

b. vertical wells drilled behind the slope crest



Development carried out	Groundwater conditions	
	Initial	Eventual
(1) Excavation stage 1 : horizontal drain (6) installed :	GWT at (10)	GWT at (11)
(2) Excavation stage 2 : horizontal drain (7) installed :	GWT at (11)	GWT at (12)
(3) Excavation stage 3 : horizontal drain (8) installed :	GWT at (12)	GWT at (13)
(4) Excavation stage 4 : horizontal drain (9) installed :	GWT at (13)	GWT at (14)
(5) Excavation stage 5 :	GWT at (14)	GWT at (15)

Fig 40 - Staged pit drainage development.

or on the slope face;

c. drainage galleries or headings excavated in the rock mass behind the slope, with or without supplementary holes drilled from the gallery;

d. drainage trenches constructed down or along the slope face.

213. Methods such as well point dewatering, filter blankets and filter fabrics can be used for near surface drainage of soils and overburden slopes, and have specialized applications for water pressure relief in foundations and tailings. They are not often applicable to large-scale drainage of slopes and will not be considered here.

214. Methods a. and d. are primarily suited to the drainage of medium and small-scale slopes and for dealing with localized instabilities. Method d. provides only a limited depth of drainage and applies only in particular geological circumstances, as discussed more fully later. Methods b. and c. are generally applicable on a large scale where drainage of slopes higher than 300 ft (90 m) is required. Each method will now be considered in more detail.

Horizontal Drain Holes

215. The main advantages of horizontal drain holes are that they are relatively quick and simple to install, they rely on gravity drainage, little maintenance is required and the layout is flexible and can readily be adapted to changes in geology.

216. The primary disadvantages of horizontal drain holes are that they have only a limited drainage influence and cannot be installed until the slope has been excavated.

217. For slopes with a groundwater table some 100-200 ft (30-60 m) above the toe elevation and isotropic permeability conditions, holes drilled in from the toe to a depth approximately equivalent to the slope height with a maximum of about 300 ft (90 m) should provide effective drainage. Typically, a reduction in water pressure of sufficient magnitude to give a significant improvement in slope stability could be achieved by a row of 3 in. (8 cm) holes spaced at 20 to 50 ft (6-15 m) intervals along the face. For groundwater tables more than 200 ft (60 m) above the toe, additional rows of drains should be installed at approximately 100 ft (30 m) intervals above the toe so that the upper level of drains is not more than 200 ft (60 m) below the groundwater table.

218. For slope heights greater than several

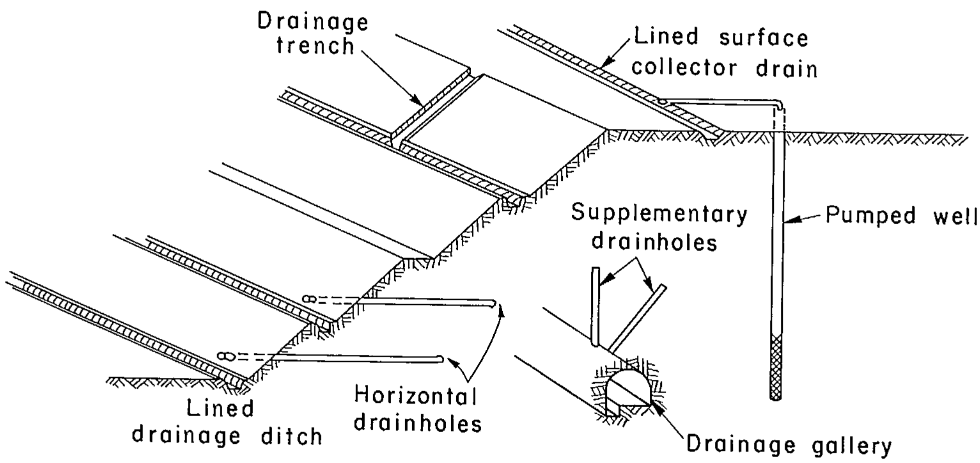


Fig 41 - Slope drainage systems.

hundred feet (100 m), the length of holes required generally makes horizontal drain holes uneconomic compared with larger-scale methods unless shallow drainage only is required; in this case horizontal drain holes can be installed on every second or third bench.

219. These guidelines are applicable only to relatively homogeneous slopes of limited height. In common with other drainage installations, the effect of the drains should always be checked by a trial installation to ensure that the required reduction in grounwater pressure is achieved. It should be noted that the volume of flow issuing from such drains gives an indication only of the permeability of the material - not of the adequacy of the drainage system.

220. Many examples exist where horizontal drain holes have been used successfully for slide control. The primary advantage of this approach is that drain holes can be installed relatively quickly in a localized area. Where slides occur in hard competent rock, the unstable zone may be relatively permeable owing to dilation during shearing. In these instances, drains intersecting the instability zone will provide the maximum reduction of water pressure at the critical location. Once hydraulic connection has been established with this permeable region at the base of the slide, good drainage within the instability zone can occur. Flow rates of up to 100 gal/min (450 l/min) have been observed from drains intersecting such zones; the equivalent flow from the unsheared rock would be less than one tenth of this value. The installation and maintenance of horizontal drains is considered later.

Pumped Vertical Wells

221. The main advantages of pumped vertical wells over horizontal drain holes are that they can be installed and drainage can begin before any excavation takes place, and that, whether installed before mining commences or not, their installation does not interfere with the mine production. Pre-mining drainage may have substantial benefits - in some instances the cost of well pumping can be offset by reductions in the cost of blasting and haulage. The pumped water, which is usually clean, is available to the mill and for other mine services.

222. Their main disadvantages compared with horizontal holes are the capital cost of pumping equipment - an individual pump and power supply are generally needed for each well - and the fact that power failure results in increasing groundwater pressure in the slope.

223. The depth and spacing of the wells must be such as to relieve, as far as practicable, groundwater pressures affecting the zone of potential instability.

224. The main effect of pumped wells in slope

dewatering is to cut off the flow toward the excavation; this causes a lowering of groundwater pressures downstream of the well line, ie, between the well line and the slope face. Wells may also be drilled from the pit bottom to control potential bottom heave problems, such as are commonly experienced in coal and some other sedimentary deposits.

225. Because of the rise in the groundwater table between adjacent wells, the depth of hole required for overall slope dewatering may be greater than the slope height - up to 1.2 x slope height. This is also the case where it is desirable to combine slope drainage and pit dewatering facilities; the wells will usually be installed to a greater depth to promote flow away from the base of the excavation.

226. An alternative approach to large-scale drainage, using several levels of wells, is shown in Fig 42.

227. The necessary well spacing depends primarily on the rock mass permeability and the geological structure. The importance of understanding the rock structure and its influence on groundwater flow is again emphasized. In many rock masses, the flow occurs predominantly within relatively permeable, nearly vertical joints. It would thus be possible to drill a vertical hole parallel to one of these joints without intersecting it. In such cases it would be preferable to install a well at a slight inclination to intersect an increased number of permeable joints. In general, the type of relationship shown in Fig 36 exists between well direction with respect to discontinuities and discharge capability.

228. To determine the position and spacing of wells around a pit, the regional flow characteristics of the area should first be determined and the sources of recharge defined. Potential aquifers and aquitards should be determined. Techniques for carrying out these investigations have already been described.

229. Once a well design has been completed a carefully monitored trial installation is necessary. The results of such a trial will invariably lead to modifications to the original layout. This trial and error approach is the principal method of designing dewatering systems for deep excavations in rock; it also reflects the current state of knowledge concerning complex rock masses, in comparison with relatively uniform sedimentary aquifers whose properties are considerably easier to define.

230. The studies of regional hydrology and well layout should include consideration of the effect of well dewatering on the groundwater environment,

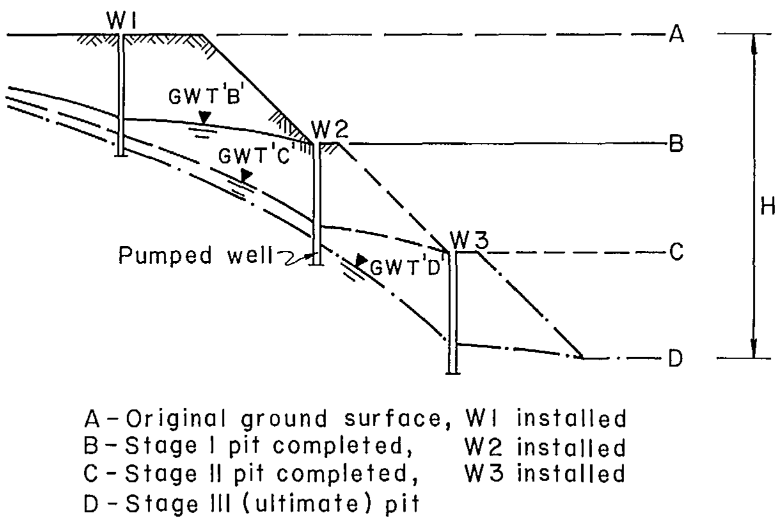
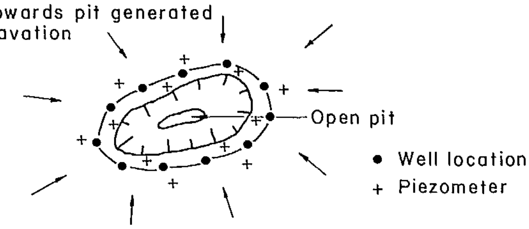


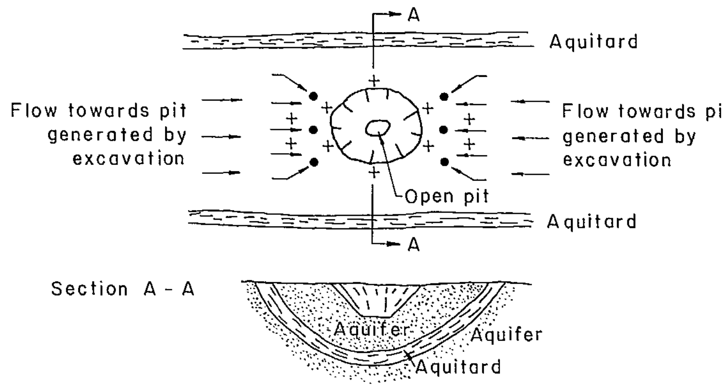
Fig 42 - Staged pumping facilities for high slopes (1000 ft - 300 m).

in particular the risk of aquifer depletion affecting domestic or industrial water supplies and the possibility of settlement over lightly consolidated aquifers. In extreme cases, some system of controlled recharge of the groundwater system outside the well line may be appropriate.

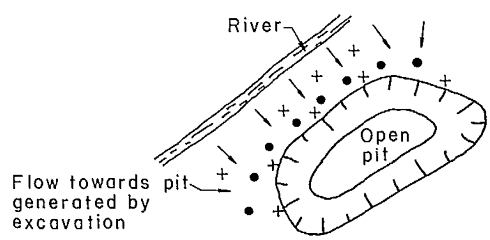
231. In low permeability materials, the draw-down ie, cone of depression around each well may be very steep and of limited extent, resulting in a substantial rise in the water table between adjacent wells. In these conditions, the spacing of wells required to effect an overall reduction in groundwater pressure may be so close that the system becomes uneconomic. Such environments may be impossible to drain effectively and slope stability may be affected accordingly, eg relatively conservative slope geometry may be necessary to attain the required stability. When data have been collected, an analytical study of the potential effects of peripheral wells on groundwater conditions within the slopes can be made using empirical or theoretical methods, such as flow net construction (Appendix F), or numerical methods (Supplement 4-1). As a result of these studies, an optimum well layout can be determined. Typical layouts are illustrated in Fig 43.



(a) Uniform areal recharge source
Uniform layout of interceptor wells



(b) Excavation located within synclinical basin
Wells placed to cut off flow parallel to synclinical axis



(c) Excavation located adjacent to river system
Wells required on river side of pit only

Fig 43 - Layout of wells around excavations in different geological environments.

Drainage Galleries

232. Drainage galleries or headings excavated in the rock mass behind the slope face are often economically justifiable as a drainage measure for large slopes. In comparing the costs of drainage holes or wells and galleries, it must be remembered that for large slopes, because of the limited drainage capacity of drilled holes, a very large number might be required.

233 Only one or two levels of galleries are usually required for any given slope. Galleries are relatively inflexible with respect to layout although they can be used to drain final as well as working slopes. Galleries are usually driven parallel to the slope, although examples exist where adits driven normal to the slope have been used for drainage. This latter approach is particularly useful for draining localized zones and can be supplemented by cross-cutting behind the slope or by fan drilling. The choice of orientation must be such that the galleries intersect major aquifers or water-carrying features, and is therefore influenced by geology.

234. Some of the advantages of drainage galleries over pumped wells or drain holes are:

a. They have greater drainage potential owing to a larger cross-sectional area, giving rise to better hydraulic connection with water bearing fissures. Their potential can be readily extended by means of additional holes drilled from the gallery.

b. They are more reliable for long term operation because gravity drainage is usually employed.

c. They provide excellent facilities for determining rock properties and subsequent drainage performance. Drainage galleries can sometimes be combined with ore evaluation studies around the pit margins.

d. Little or no interruption occurs at the slope surface.

e. They are more suitable for low temperature applications because the underground location inhibits freezing of the system.

235. The optimum layout and size of drainage galleries can be studied theoretically, using the same methods as apply to well design. The limitation of this approach is the accuracy with which the permeability characteristics of the rock mass can be defined. The influence of additional drainage holes can be studied in the same manner. Typical results of such analyses, determined in this case from computer studies, are given in Fig 44 and 45.

236. Where the rock is more permeable in a vertical direction, eg owing to predominant vertical jointing, gallery drainage is efficient

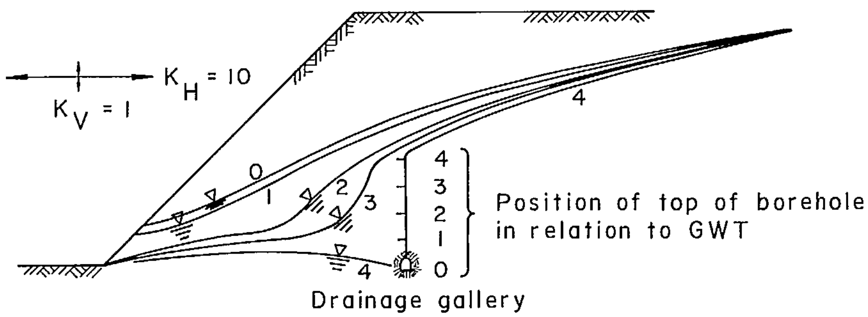


Fig 44 - Influence on drainage of supplementary vertical boreholes drilled from a drainage gallery in a slope with horizontal to vertical permeability ratio of 10:1.

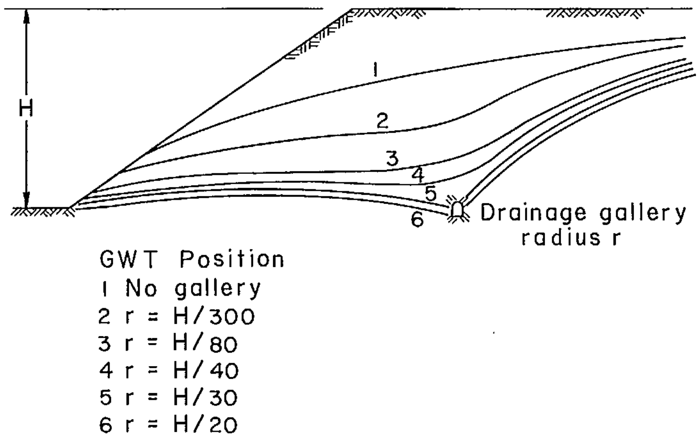


Fig 45 - Influence of drainage gallery radius on drainage of a slope with equal horizontal and vertical permeability.

and results in steep drawdown. If the ratio of horizontal to vertical permeability is high, as in the case of horizontally bedded sedimentary rock with relatively impermeable layers interbedded between permeable horizons, the water flow is predominantly horizontal and will tend to by-pass the drainage gallery. In this case drainage can be improved by drilling vertical boreholes from the gallery to intersect some of the horizontal flow. Figure 44 shows the effect of supplementary holes in tapping the horizontal flow. The spacing of the holes along the gallery should provide a continuous cut-off to the flow.

237. In rock masses where a considerable variation in directional and spatial permeability exists, or where the spacing of discontinuities of the rock mass is so large that adequate hydraulic connection with the gallery does not exist, attempts should be made to improve drainage efficiency by additional drainage holes drilled from the gallery. Ideally, these should be oriented in a direction normal to that of maximum permeability to intersect the predominant flow paths.

238. The effective diameter of the permeable zone around the gallery is an important consideration in the design of this type of drainage system. Fig 45 shows the influence of the effective gallery radius on the drawdown conditions in a 45° slope. For large slopes - 500 - 1000 ft (150 - 300 m) - the diameter of the gallery required to effect substantial drawdown may be obtained by drilling a fan of holes outwards from the gallery.

239. An alternative approach for large slopes is to construct galleries at two or more levels, avoiding extensive underground drilling. Each gallery can then be used to drain a 200 to 300 ft (60-90 m) lift of the slope.

240. Sometimes, thin impermeable seams of considerable continuity divide a rock mass into a series of separate reservoirs, each bounded by a barrier of low permeability relative to the contained zone. To ensure effective overall drainage, holes should be drilled from the gallery to pierce all zones in which water pressure might be detrimental to stability.

Drainage Trenches

241. The use of backfilled trenches is one of the oldest methods of slope stabilization in soil and soft rock slopes. Because of the limited depth to which trenches can readily be excavated, the method is limited to cases where shallow drainage only is required.

Slope Trenches

242. In certain geological formations, primarily those of sedimentary origin, slope trenches can be used to advantage for the relief of groundwater pressures at shallow depth. A typical example is that of an opencast coal operation having a footwall composed of bedded strata dipping into the excavation, as shown in Fig 46. During the final excavation stages, a footwall composed principally of rock with thin seams of relatively impermeable coal or clay can remain. Because of these aquicludes, high groundwater pressures can cause relatively shallow instabilities involving the sliding of slabs of rock on weak seams. Trenches excavated and backfilled down the dip of the slope can be used to relieve adverse groundwater pressures behind the weak seams forming the aquicludes. The trenches must extend into the underlying aquifer to provide hydraulic connection.

243. The possibility of using shallow drainage in certain stratified materials illustrates the need to appreciate the influence of geological structure, permeability, and drainage methods on stability. In some instances where potential slides would occur at a relatively shallow depth, deep drainage with galleries or deep wells would be both unnecessary and inefficient.

Horizontal Trenches

244. Where mining is being carried out in horizontally bedded materials composed of seams of different permeability, the use of trenches at the toes of working slopes can often be beneficial to the stability of the slopes. Figure 47 shows a backfilled horizontal trench used to relieve confined groundwater pressure, first to improve the stability of the existing cut, and second to prevent heave during deepening of the pit.

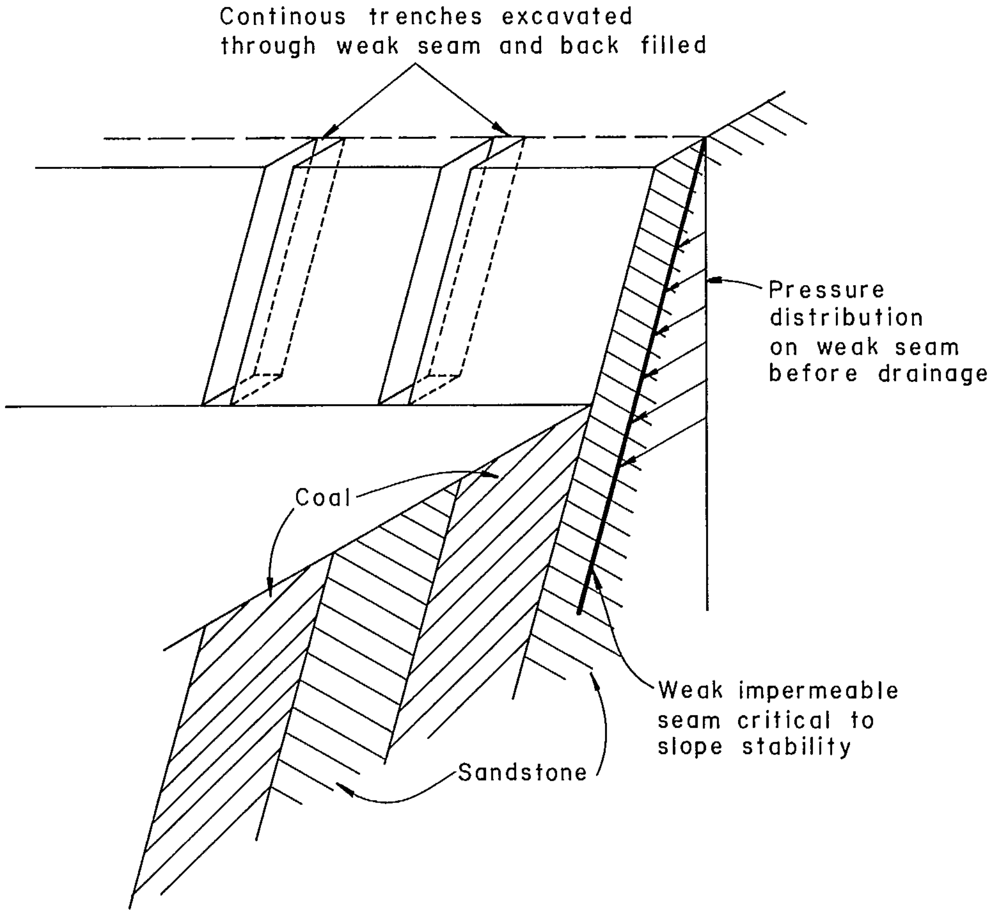


Fig 46 - Typical application of drainage trenches.

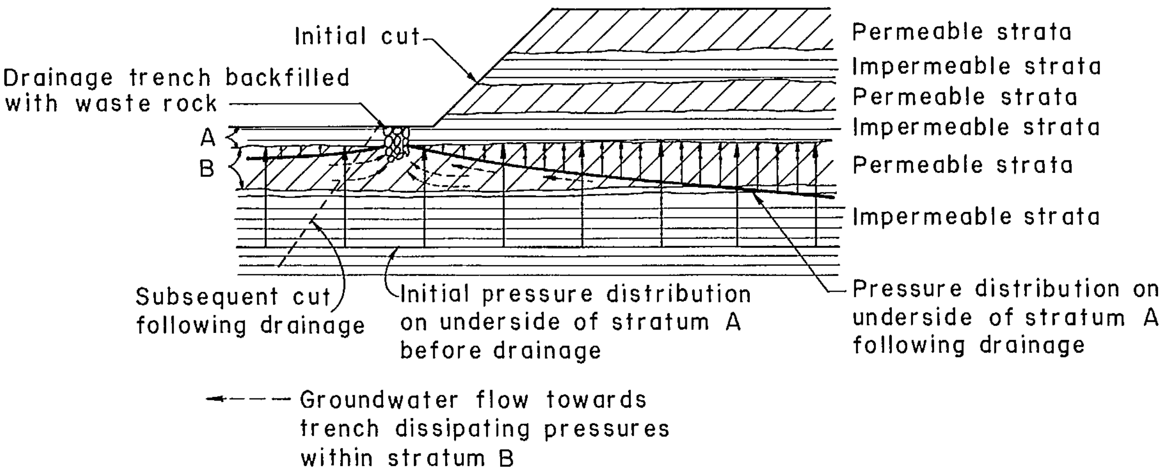


Fig 47 - Use of drainage trenches in horizontally bedded strata.

SELECTION OF DRAINAGE SYSTEMS

245. The choice of a particular drainage system will in general depend on the following criteria:

- geological characteristics of the slope material,
- scale of drainage required,
- cost of the installation,
- climatic considerations.

246. The geological characteristics will define the potential instability mechanism for the slope and hence the depth to which drainage will be required. This in turn will influence the choice of drainage method. For example, for relatively shallow drainage, horizontal drain holes or trenches might be best, whereas for deep drainage, wells or galleries could be more effective. The method of drainage will be further affected by the differences in rock types if the slope is non-homogeneous or anisotropic.

247. The scale of the slope to be drained will determine whether relatively localized drainage using small diameter horizontal drain holes will suffice or whether it is necessary to adopt large diameter holes or underground galleries, possibly augmented by drainage wells, to achieve a wide zone of drainage influence within the rock mass.

248. The cost of installation will be determined by local factors and cannot readily be predicted. The selection of drilling or boring equipment will depend largely on the extent of drilling required, and may be influenced by the type of equipment available at the mine.

249. Climatic considerations will partly determine the amount of recharge to the slope, which in turn will affect the extent of drainage required. Where prolonged periods of freezing are anticipated, operational convenience may suggest the use of drainage galleries in preference to wells or horizontal drains.

INSTALLATION OF DRAINAGE SYSTEMS

Horizontal Drain Holes

250. Many techniques have been developed for rapid installation of horizontal drains in softer overburden materials. A typical track-mounted drilling rig using a "drop-off" drilling bit and a slotted plastic pipe which is installed through a drill casing is shown in Fig 48.

251. In soft and weathered rock, perforated casing may be installed using a casing shoe, or specially perforated drilling rods may be used.

252. By comparison, no specialized techniques have been developed for installing drains in rock. In sound rock where holes will remain open, standard percussion machines capable of drilling to about 100 ft (30 m) are usually used. After drilling, the rods are withdrawn and a perforated drain pipe installed. This method is cheap but is unsuitable for weak rock or long holes.

253. Under difficult drilling conditions, diamond drilling or rotary tricone drilling equipment with a casing system may be required. Using coring equipment, a good assessment of the rock mass conditions can be made although costs are high.

254. It is to be expected that powerful rotary rigs for drilling horizontal holes, similar to existing blasthole rigs, will become available in the near future. Such rigs would lead to improved efficiency for drilling of long, large diameter holes, and would make horizontal drains more practicable for the drainage of very large slopes.

255. After completing the drill hole, it is advisable to install a drainpipe to ensure the uninterrupted flow of water towards the face throughout the life of the installation. The pipe should be slotted or perforated at the inner end and left plain towards the slope face to ensure that the water is carried clear of the slope face and collected. The length of the perforated section required depends on local conditions, but will be about 50% of total length.

256. Plastic pipe made specifically for drain applications is available. This pipe is inexpensive, light-weight and easy to install. The pipe is pre-slotted in a number of sizes and is usually constructed from rigid PVC which can be coupled together using "push fit" glued couplings. Alternatively, screw-coupled galvanized iron water pipe, black iron pipe or welded steel casing can be used, perforated on site as required.

257. Much of the benefit of slope drainage will be lost if the water draining from the holes is

Fig 48 - Track-mounted drilling rig for installing horizontal drains.

Fig 49 - PVC Sheeting used to prevent water infiltrating a slope.

not prevented from recharging the bench below. A properly constructed and maintained collector drain should be installed to intercept drainage and carry it to a suitable disposal point.

258. In severe climates, the flow from drains may prove insufficient to prevent freezing. If such problems occur, the steel drainpipe should be extended outside the hole and the pipe covered with an insulating rockfill barrier.

Vertical Wells

259. Techniques for drilling large diameter wells in overburden and soft rock materials have been well established by the water supply industry.

260. For drilling wells in rock, powerful rotary rigs are usually employed using tricone or eccentric cutting bits. Such rigs are capable of drilling large diameter holes, typically up to

15 in. (40 cm) in diameter and in excess of 1000 ft (300 m) deep. For many applications of pit dewatering, particularly where the drill hole will remain open, blast hole rigs can be used for well drilling, although available air pressure may limit the depth to 200-300 ft (60-100 m).

261. In weaker rocks, where casing and well screens are required, specialist equipment will usually be required. Well screens consist of various perforated or slotted casings which prevent fine fissure material or soft rock from entering the hole and impeding pumping. The screens in slope dewatering wells should be located in the most permeable zones of the rock mass.

Drainage Galleries

262. The construction method and cost of drainage galleries will depend on the rock type and the availability of equipment. If a rail mucking system is used during excavation, it is usual to adopt the minimum possible cross section to reduce cost and support. If a rubber tyre loader system is used, the gallery will be somewhat larger. In some instances the use of specially constructed tunnelling machines may prove advantageous, particularly if the gallery is long.

263. From the drainage standpoint, a tunnel excavated by blasting is to be preferred because of the free draining zone created around the periphery by blast damage.

264. Careful gradient control is required for ensuring gravity flow of the drained water. Depending on the amount of flow within the gallery, it may be possible to channel the flow in a ditch along the side of the tunnel to maintain access through the remainder of the gallery for groundwater monitoring etc. The flow from drainage galleries can be measured by V-notch weirs at the gallery portal and at intermediate stations. For measurements at other locations, a pump and flowmeter can be used in conjunction with a sump. The pumping rate is adjusted until a constant level is achieved in the sump and the flow is then measured.

265. If supplementary drain holes are required from the gallery these can be installed using a percussion drill/jack leg arrangement. In large diameter tunnels and where a significant number of holes are required, the use of a fan drill, which can drill holes at different vertical angles from the same set-up, is generally more economic.

MAINTENANCE AND IMPROVEMENT OF DRAINAGE SYSTEMS

266. The effectiveness of drainage installations can be improved during the life of the system by a number of methods. A primary method is to minimize infiltration into the slope surface. This requires particular attention to surface drainage measures and the diversion and collection of water before it percolates into the slope. Under some circumstances, particularly where the slope is showing signs of distress and where open cracks are visible, it is advantageous to use specific waterproofing measures, such as placing impermeable membranes over the top and crest of the slope. If any movement of the slope is expected, the membrane should be flexible (heavy gauge butyl rubber sheeting is ideal for this purpose). For stable slopes, asphalt or other more rigid sealants may be applied. Fig 49 shows a rock slope treated in this manner using PVC sheeting overlain by wire mesh anchored to the slope.

267. Horizontal drain holes and well casings can be maintained by periodic cleaning using a high pressure water jetting tool and wire brushes. Horizontal drain holes drilled on a slight up-grade will usually be self-cleaning and will require a minimum of maintenance. Wells should also require little attention provided the initial installation, particularly with regard to gravel packs and well screens, is properly executed.

268. Under certain geological and groundwater conditions, deposits of carbonate and other substances may lead to decreased efficiency of the drainage installation. Such accumulations can be controlled by acidic additives during water jetting at intervals during the life of the installation.

269. In competent rock masses, the hydraulic

connection between a drain hole or well and the surrounding rock can sometimes be improved by controlled blasting to fracture the rock in the inflow region. Specially prepared "shaped charges" can also be used to perforate well casings or drain pipe at specific locations after the initial installation has been carried out.

270. Another method of improving connections to the rock mass is to use the hydraulic fracturing technique commonly employed in the oil industry for improving reservoir performance. In this technique a selected zone of the well or drain hole is isolated with packers and water under high pressure is injected into the zone. At a sufficiently high pressure, opening of the fractures will occur or new fractures will be created. The fractures can then be "propped" open by injecting sand, which results in a considerable increase in hydraulic conductivity. Such methods should be applied with extreme caution where stability conditions are critical since a temporary reduction in stability may occur during the hydrofracturing process.

OTHER GROUNDWATER CONTROL MEASURES

271. Drainage is a term that implies the removal of water from an area which may be critical with respect to stability. In some cases, it may be more expedient to prevent the water from reaching the critical zone, by creating an artificial barrier within the ground. Such a barrier, commonly called a cutoff, dams or retains the water behind it and allows natural drainage to occur within the slope, as shown in Fig 50.

272. Cutoffs can be used only where there is a well defined stratum of permeable, water-bearing material. They can be used to isolate excavations from river channels or to seal off specific horizons behind slopes, eg, buried pre-glacial channnels. These measures are expensive, and a thorough investigation of all alternatives is necessary before deciding on a cutoff.

273. Cutoffs can be constructed from sheet piling, by excavating trenches and backfilling with concrete or by injecting cement or bentonite grout. The last method is most commonly applied to slope problems and is called a grout curtain or grouted cutoff; it is usually formed by grouting a series of primary holes at a spacing calculated according to material properties and experience. A typical layout is shown in Fig 51.

274. The effectiveness of the initial row of grout holes is determined either by piezometric observations on either side of the curtain or by permeability testing in intermediate holes. If the primary holes are spaced too widely to give an

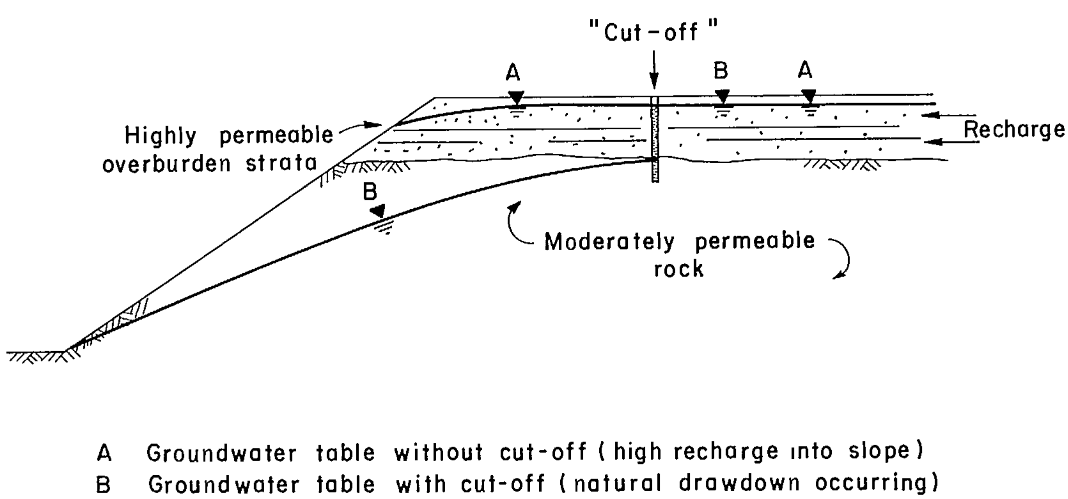


Fig 50 - Use of cutoff to control seepage into a slope.

adequate reduction in level across the curtain, secondary holes are grouted between the primary holes, or are slightly offset upstream. If necessary, a third set of intermediate holes can be grouted. (Fig 51).

275. The exact layout and spacing of grout holes will depend not only on the material properties but also on the required efficiency of the system. Grouting procedures and the type of grout used are dependent upon the materials encountered, and specialist knowledge is frequently necessary to determine the best method.

MONITORING OF DRAINAGE

276. To check that a satisfactory reduction in groundwater pressure results from drainage or other groundwater control method, monitoring is necessary. The monitoring system should be installed before construction so that the initial conditions and the response to drainage or to a groundwater cutoff can be monitored. The monitoring system is usually a network of piezometers and equipment for measuring drain discharges. Monitoring is discussed in full in para 177 to 184 and in Part III of the chapter.

277. If a good seal can be achieved around specific drains - by grouting much of the hole around the pipe - they can be used to measure groundwater pressures. A valve can be attached at the drain outlet and a pressure gauge mounted on the upstream side. The pressure can then be measured directly by shutting off the valve temporarily. The measurement of flow is a poor indication of drain performance and should only be used as an auxiliary measurement to piezometric monitoring.

DRAINAGE TRIALS

278. The final design of a drainage system will usually be decided following an initial trial of limited scale which should be carefully monitored. The results of such trials can be used to determine the required drain hole spacing, well spacing or gallery location. For the installation of horizontal drain holes, a network of piezometers will generally be laid out as shown on Fig 52. Initially, a wide drain hole spacing is adopted as indicated. The spacing is gradually reduced by drilling intermediate drains until the required piezometric response is achieved. In geologically complex rock masses, such simple trials may be inadequate and a more detailed evaluation will be required using the techniques already described.

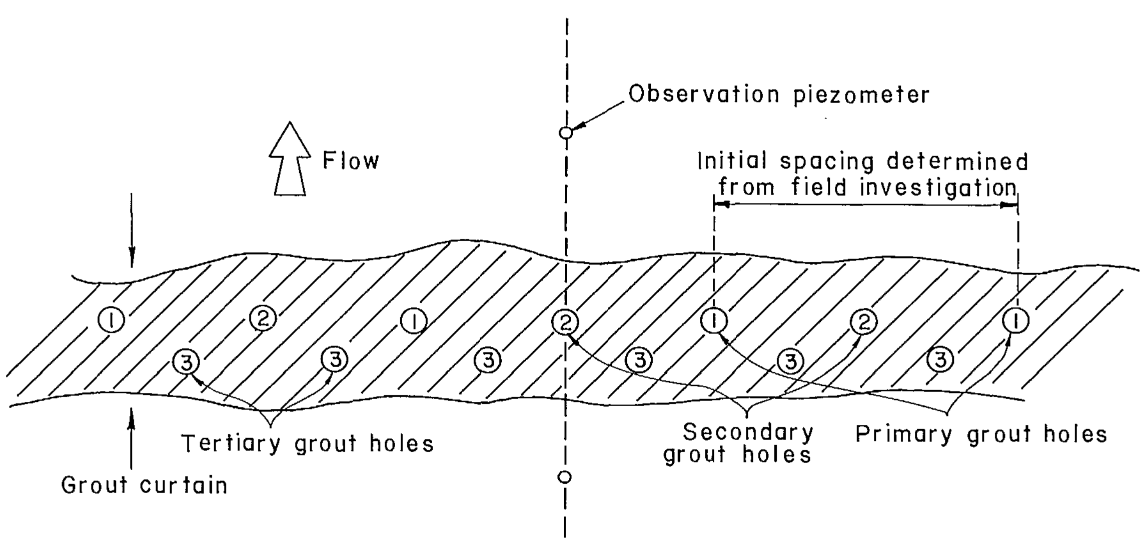
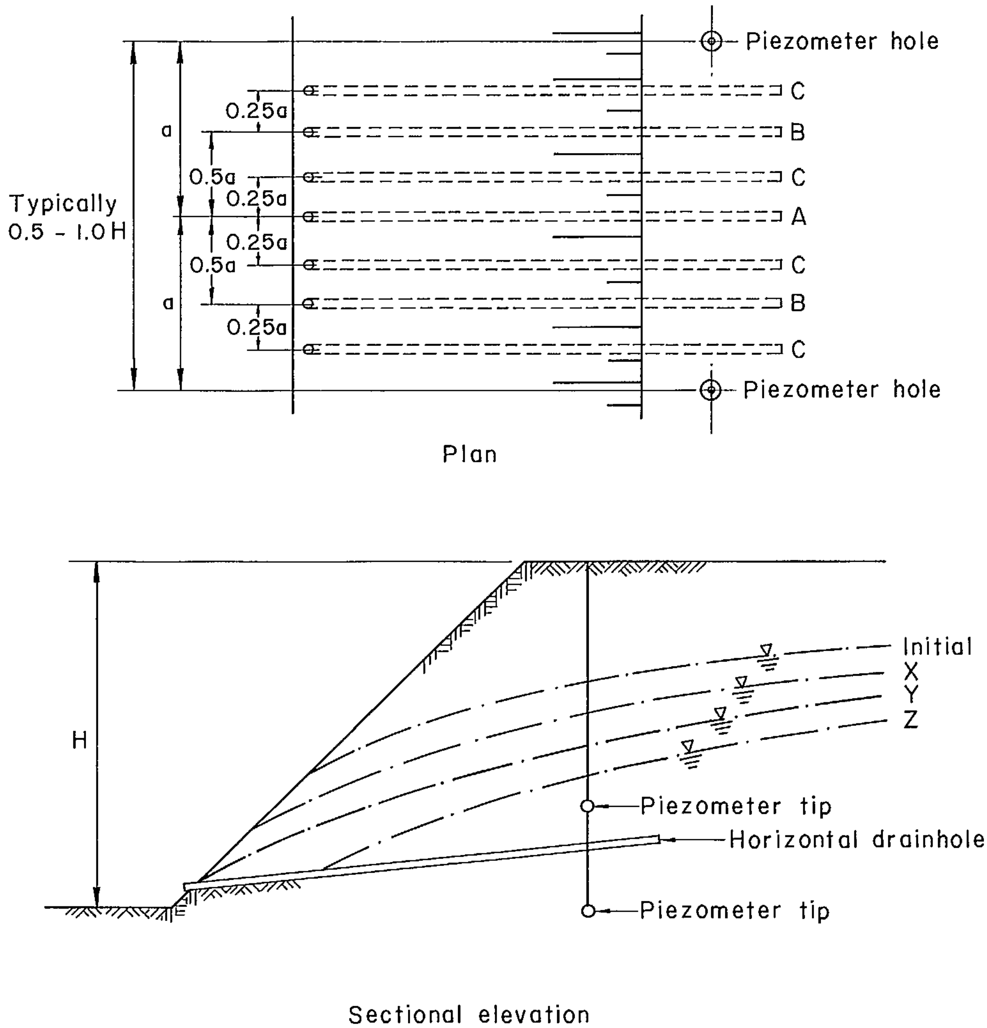


Fig 51 - Plan view of staged development of a grout curtain.



A-drain series 1 Groundwater table at piezometer section : X
B-drain series 2 Groundwater table at piezometer section : Y
C-drain series 3 Groundwater table at piezometer section : Z

Fig 52 - Typical trial layout of horizontal drainholes.

PART III: GROUNDWATER EVALUATION PROGRAMS

279. Groundwater investigations are required at all stages of mine development. Generally, they are performed concurrently with other geotechnical investigations. Their main objective is to provide information on the role of groundwater in slope stability and for designing and monitoring drainage measures.

280. The scope of groundwater investigations depends upon the relative importance of groundwater to safe and economic mining and to the stage of development the mine has reached. Figure 1 shows this relationship. It is important to establish the effect of groundwater on slope stability at the earliest possible stage.

281. The individual components of the evaluation program will vary from site to site, as will their relative importance. Figure 53 shows how these components fit into the overall groundwater study, during the feasibility, mine design and operating stages. To make the program outlines which follow as clear as possible, descriptive material has been minimized; cross references to the detailed description of the techniques are given.

FEASIBILITY STAGE

Initial Exploratory Investigation

282. The main purpose at this stage is to make an early and thorough assessment of the potential role of groundwater in the planning and operation of the mine, ie, determining whether groundwater conditions are likely to be favourable or unfavourable to mining. This is achieved by an initial exploratory investigation, which is principally a desk study to collate and interpret all available information, augmented by limited studies such as aerial photography and measurements on site. The activities to be carried out are as follows.

283. Obtain available topographical maps of the mine region at various scales. Obtain aerial photographs of the mine region, including stereopairs and special emulsion photographs, eg, infra-red if possible (Appendix H). From these photographs, with specialist assistance if necessary, determine:

- the pattern of surface drainage (rivers, streams, lakes etc).

		PARA.	FEASIBILITY STAGE: INITIAL EXPLORATORY INVESTIGATION	FEASIBILITY STAGE: DETAILED EVALUATION	DESIGN STAGE: FIELD EVALUATION	DESIGN STAGE: ANALYSIS	DESIGN STAGE: DRAINAGE	DESIGN STAGE: MONITORING	OPERATING STAGE: EVALUATION	OPERATING STAGE: DRAINAGE	OPERATING STAGE: MONITORING
LOCAL STUDIES	PUMPING RATES	183							●		●
	FACE MAPPING	312							●		●
	OTHER LOCALISED DRAINAGE	313								●	●
	MAIN DRAINAGE INSTALLED	313								●	●
	FINAL DRAINAGE DESIGN	212								●	
	DRAINAGE TRIALS	278, 313								●	
	DETAILED DRAINAGE DESIGN	212					●				
	PRELIMINARY DRAINAGE ESTIMATES	198		●							
	STABILITY ASSESSMENT	147,		●		●			●		
	GROUNDWATER PRESSURE DISTRIBUTION	61, 164		●		●			○		
	SEEPAGE ANALYSIS	113-146		●		●			○		
	ANALYSIS OF FIELD DATA	113	●	●	●	●			●		●
	LOCAL IN PIT MONITORING	312,									●
	REGIONAL MONITORING PIEZOMETERS	315, 61,						●			●
	SLOPE MONITORING PIEZOMETERS	65,						●			●
	INITIAL PIEZOMETER INSTALLATIONS	177	●	●							
	ADIT TESTS	104		●	○						
	WELL TESTS	92		●	○						
	PERMEABILITY CONSTANT HEAD TESTS	84		●	●				○		
	PERMEABILITY FALLING HEAD TESTS	81	●		●				○		
	PERMEABILITY CORE EVALUATION	77	●	●	●				○		
	DRILLING RECORDS	74	●	●	●				●		
	ROCK MASS STRUCTURE	71	●		●				●		
	MAJOR STRUCTURES	105	●		●				●		
	GEOLOGY	153	●						●		
REGIONAL	AERIAL SURVEYS	51-56	●								
	HYDROGEOLOGY	51-56	●								
	HYDROLOGY	51-56	●		●				●		

● RECOMMENDED PROCEDURES

○ RECOMMENDED PROCEDURES TO BE CARRIED OUT IF CONDITIONS ARE UNFAVOURABLE

Fig 53 - Reference chart for groundwater activity at the various mining stages.

- surface signs of groundwater conditions (spring lines, marsh areas, dry zones, etc).
- vegetation which may indicate groundwater or geology variations.

284. Obtain available precipitation records relevant to the site, preferably for a considerable number of past years. Where information is limited, a rainfall gauging station should be set up as soon as possible. If the mining zone is close to a river or large stream, obtain hydrographs if available, or take measurements of flow and correlate these with precipitation.

285. Consider fully all the information on regional geology available from the geology task. Complete a composite geological/hydrological map of the mine region - typically a zone of about two miles square around the proposed excavation. This composite should summarize what is known of the topography, geology - main rock type descriptions and structural features such as faults - and hydrology of that zone. Draw representative cross sections.

286. From the plans and sections, consider what can be deduced about likely future conditions at the mine, including:

- surface water run-off problems which may arise around the mine, particularly if in a basin.
- the effect of any large-scale surface water features, eg, lakes or rivers which are close enough to recharge the mining zone directly.
- the likely relative permeabilities of the main rock types, eg, low permeability shales and slates, high permeability sandstones and limestones, etc.
- the influence of overburden deposits and degree of weathering of the rock on groundwater behaviour.
- the effect of major structural features, eg, faults, shatter zones, on the continuity of groundwater movement.
- which rock units are likely to be aquifers, and whether the geological structure is such that they will form sources of recharge to the excavation.
- whether the combination of geological structure, lithology and topography is likely to result in artesian conditions within the mining zone.

287. Monitor exploration drill holes for information on water occurrences. Groundwater levels in these holes should be recorded on the map and sections during and after drilling. Drilling crews should be assisted in observing groundwater occurrences, cavities, water losses etc. Where holes are flowing, this should be noted together with the depth at which the flow was encountered. This indicates a source of groundwater and should be marked on the map. Whether the hole flows continuously or ceased to flow after a certain period should be recorded.

288. Obtain standing water levels in existing boreholes which will give some preliminary information on groundwater tables. Carry out some falling head tests in existing boreholes to obtain representative permeabilities of the main rock types, preferably around the margins of the orebody where final pit slopes are likely to be established (Appendix B). Install a limited number of piezometers in existing exploration holes to measure piezometric pressures in the main rock units, particularly in suspected confined aquifers (Appendix A). Install piezometers in relation to major structural features. Monitor piezometric conditions regularly to determine seasonal variations.

289. Plot the above data on plans and sections of the proposed mine layout. Depending on complexity, it may be possible to make preliminary approximate assessments of the effect of known groundwater tables, etc, on the stability of the proposed slopes. In all cases, once this initial stage has been completed, a qualitative if not quantitative assessment of the influence of groundwater on mining can be made. This may be categorized as of major, moderate or no influence.

Detailed Investigation

290. If groundwater is anticipated to be an important influence, an early detailed evaluation is necessary, so that it can be quantified and financial implications can be assessed.

291. The objectives are to define water tables, piezometric pressures and permeability values.

The need to define these parameters with greater accuracy involves a larger number of tests and observations, together with the use of large-scale testing techniques and specialist assistance. The principal activities are as follows.

292. From the initial exploratory investigation, make an assessment of the areas in which additional information is necessary. These will include the zones within which large-scale intermediate and final pit slopes will be developed, major geological features important in the context of groundwater, and major sources of recharge.

293. Within these zones, determine the type of information required, ie, permeability, flow, or groundwater pressure data, and set up a program to evaluate them. A typical program might include:

- continued monitoring of precipitation, drilling records etc.
- geotechnical logging of exploration cores (RQD, fracture spacing etc) to correlate with permeability; this will allow extrapolation of permeability values to zones not tested directly.
- detailed permeability measurements using constant head or pressure tests (Appendix C), to fully define the hydrological properties of rock units.
- compilation of continuous permeability profiles of selected drillholes representative of the main geological features and tests at specific important horizons ie, in fault zones, etc.

294. These data must now be used, together with input from other tasks, in preliminary stability assessments and mine design. A hydrogeological model should be constructed using plans and sections, and the model analyzed at various stages of development. The sensitivity of the results to various input parameters should be ascertained; if there are critical areas where insufficient or unreliable data exist, additional field testing should be carried out.

295. The procedures used for this preliminary design are described in para 303. The detailed evaluation should examine various options of slope geometry, methods of drainage etc., which can be optimized and analyzed financially. A decision must be made on whether the proposed operation is feasible or not. It is rare for groundwater conditions alone to be the deciding factor; however, unfavourable groundwater conditions which will mean additional mining costs may combine with other factors such as limited ore reserves, ore-dressing problems, transportation difficulties, etc, to make the operation an unfavourable proposition.

296. In summary, individual circumstances dictate the level of effort which must be expended on groundwater evaluation during the feasibility stage. It must be sufficient to provide reliable answers to two questions. First, to what extent are anticipated groundwater conditions likely to be unfavourable to mining? Second, if they are unfavourable, will drainage and other measures be effective enough to allow safe and economic mining?

MINE DESIGN STAGE

297. If the feasibility of the proposed operation has been demonstrated, detailed evaluation and design of the mine follow. At this stage, sufficient data collection must be carried out to construct a reliable hydrogeological model and the data analyzed to provide a cost-optimized slope geometry and drainage design for implementation during the operating life of the mine. The scope of the investigation will be influenced considerably by the amount of work carried out at the feasibility stage.

Field Groundwater Study

298. The data collection component at the design stage is a field groundwater study. This involves the same types of study as described above for the detailed evaluation at the feasibility stage (para 290 to 296).

299. Requirements for boreholes around and beyond the initial pit limits should be considered, for both permeability evaluation and as long term monitoring locations. The number of holes depends on the scale of the proposed mine and the complexity of the local hydrogeology. Typically, about ten holes are required. The bottom of the holes should be of about the same

elevation as the final pit bottom. Inclined holes behind the proposed slopes will usually provide more representative data. The drilling program should be fully integrated with other slope stability studies. Specific studies related to the delineation of the geology, major structures and rock mass structure should be carried out in conjunction with the groundwater evaluation program.

300. Detailed permeability testing using boreholes should be carried out (falling head tests, Appendix B, and constant head tests, Appendix C). Other testing techniques such as water sampling, tracing, etc, should be reviewed in relation to the project. If anomalous or adverse conditions are encountered in any area, further test work should be carried out.

301. Large scale hydraulic testing using exploratory adits or wells should be carried out if adverse groundwater conditions are encountered. These tests should be designed to produce data relevant to the design of possible drainage systems.

302. If a detailed evaluation has been carried out at the feasibility stage, there may be no need for a field groundwater study, or it may be on a reduced scale and designed to infill areas of information not fully covered previously.

Current and Future Groundwater Conditions

303. From a study of all the field data, an analysis of current and future groundwater conditions must be carried out as follows:

- construct a detailed hydrogeological model of the proposed pit, drawing plans and sections.
- from the model, develop an analogue (Appendices F and G) or a numerical model (Supplement 4-1) and derive groundwater pressure distributions for each stage of pit development. Best and worst anticipated conditions should be determined, bearing in mind the accuracy of the input data and seasonal variations in groundwater conditions.
- use the resulting pressure distributions to study the influence of groundwater on slope stability by incorporating this data with input from the structural geology and mechanical properties tasks in analyses of slope stability. A relationship showing this influence at various development stages and different excavation sequences should be determined. If critical areas emerge, further field studies and analysis should be carried out.
- estimate groundwater seepage rates for various stages of pit development to determine pumping requirements.

Cost/Benefit of Drainage

304. If groundwater is critical to slope stability, the benefits of drainage should be examined. Further groundwater analyses are required to determine the effect of various drainage layouts. The cost of the various layouts and their effect upon slope stability must be determined using the techniques of the design chapter. The cost of drainage and its influence on stability can then be used in cost/benefit analyses to determine the potential benefits of drainage.

305. The practical details of drainage must be examined at this stage in relation to expected operating conditions. It is now possible to select the preliminary drainage design.

Monitoring

306. The groundwater monitoring carried out during the design stage usually involves the installation of piezometers in orebody evaluation drillholes for monitoring during the early operating stages of the pit. They should be located so that long-term operation is reasonably assured, and where they will monitor conditions adjacent to the initial pit operations. In addition, several regional locations should be selected to monitor long-term and seasonal changes. Adequate long-term protection of the borehole collars is essential.

307. Monitoring requirements for the operating stage should be assessed and planned. These will depend on the influence of groundwater at the various stages of development predicted by the design study and on the location and extent of any

drainage systems.

OPERATING STAGE

308. Direct observations during the initial operating period are the most reliable means of predicting long-term groundwater conditions. Groundwater evaluation during this stage of development has two objectives: first, monitoring to determine how accurately the design predictions are working out in practice, and second, providing adequate data and experience to enable the design of the late and final stages of pit development to be optimized.

309. The level of effort required during this stage is thus entirely dependent upon the accuracy of the design forecast. If substantial differences from the design assumptions are indicated, re-design using the procedures described in para 303 is necessary. If major changes in the design target arise because of, for example, financial constraints or the discovery of additional ore, further field evaluation as outlined in the design stage will be required (para 297 to 305).

310. Both short-term and long-term development planning is necessary. The need for groundwater data input at any stage will depend on their relative importance to the overall mine development.

311. Frequently, groundwater conditions may be known to affect final slopes seriously, but be of little significance for the interim slopes planned for the first few years. In this case, groundwater investigation may cease or carry on at a much reduced level. Piezometers should, however, still be monitored. Checks on up-dated development plans should be carried out to determine at what stage groundwater will be significant to mine design. This will indicate when and if groundwater investigation should be resumed or increased to provide the necessary data for detailed design of the late stages of mine operation. The importance of continuity of monitoring is again emphasized.

Monitoring

312. Monitoring includes the observation and collation of all relevant groundwater data that can be obtained during the early stages of mine development. The data is used to produce reliable, long-term forecasts. A typical monitoring program will include:

- continuing the precipitation, piezometric pressure and other measurements initiated in the feasibility and design stages.
- collecting data relating to geology, structure and permeability from all ongoing drilling in representative areas.
- using the initial excavations below the groundwater table for the prediction of long-term conditions. They should be monitored using local, short-term piezometer installations in addition to the long-term monitoring measures already adopted.
- carrying out in-pit mapping of seepage areas as excavation proceeds and investigating any zones of anomalous seepage.
- recording pumping rates to determine variations in pumping rate with depth, exposed rock types, etc.
- maintaining and up-dating long term monitoring installations.

Drainage Trials

313. A thorough evaluation of any proposed drainage measures must be completed before a critical stage, as predicted by the design, is reached. This evaluation will include drainage trials with associated monitoring to determine the efficiency and practicality of a proposed method. The final design of a drainage system, with particular emphasis on special requirements for local areas of the pit, and its scheduling in relation to pit development, should be determined only after satisfactory trials have confirmed the validity of the design derived by analysis or analogue studies.

314. Long-term drainage measures should be implemented in stages, and each stage should be monitored and evaluated to ensure that the performance is as predicted and that the required improvement in stability is being maintained. If not, re-evaluation must be carried out to produce a modified design.

Long-Term Monitoring

315. As previously emphasized, monitoring of groundwater changes in relation to pit development is of fundamental importance:

- in assessing whether predicted stability conditions are being maintained
- to determine if drainage measures are required or those installed are operating correctly
- to provide data for future designs.

316. The location of monitoring installations will depend on excavation geometry and the schedule of development. Optimum use should be made of deep holes by installing several piezometers at different elevations, based on geological conditions and mine geometry. The positioning of piezometers so that they lie on sections radiating from the approximate centre of the pit and orthogonal to the face facilitates the drawing of cross sections for interpretation.

317. In-pit monitoring using piezometers installed with mine drilling equipment should be used to check conditions immediately behind the slope surface and in the vicinity of the toe. Particular attention should be applied to areas which have been identified as critical with respect to groundwater.

318. Continuity of information is essential. Therefore, new installations should be phased in before existing installations are destroyed by pit development.

Re-evaluation

319. The monitoring data should be plotted continuously and used to carry out regular reviews of conditions, typically every six months or every year, to check the validity of the design. These reviews indicate what investigation and analysis should be carried out in relation to the next phase of development.

320. If a stability problem develops during the operating stage, it is highly probable that the ensuing investigations and remedial measures will require groundwater information. The extent of previous investigations will determine the level of activity necessary, but pressure of production may mean that the investigations have to be intensive. Installation of piezometers, stability analyses and design of remedial measures may need to be completed within a short time. The general approach is as follows:

- assemble all groundwater and geological data and plot them on the mine plans and sections
- survey and map the problem area
- determine areas where groundwater information is required
- organize drilling and installation of monitoring instruments
- add the additional data to the plans and sections
- analyze for stability, using back analysis if instability has occurred
- liase with the mine engineering section to design remedial works
- continue data collection and monitoring throughout the period of remedial action
- on completion of the remedial work, set up a system to monitor groundwater behaviour.

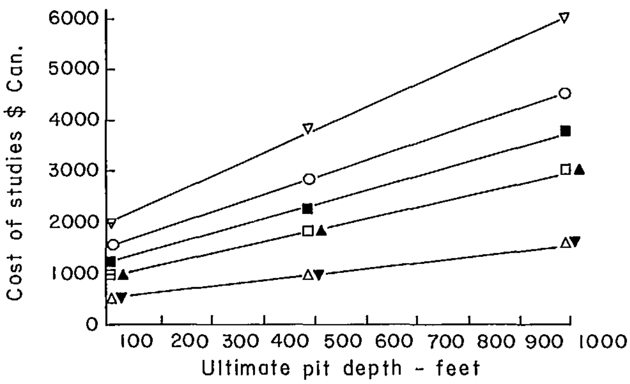
321. Groundwater investigations during this stage of mine development will be oriented to solving problems that could affect production, and to collecting piezometric and flow data to check against design assumptions. Marked departure from anticipated conditions will then require the appropriate re-design effort.

COST AND TIME GUIDELINES

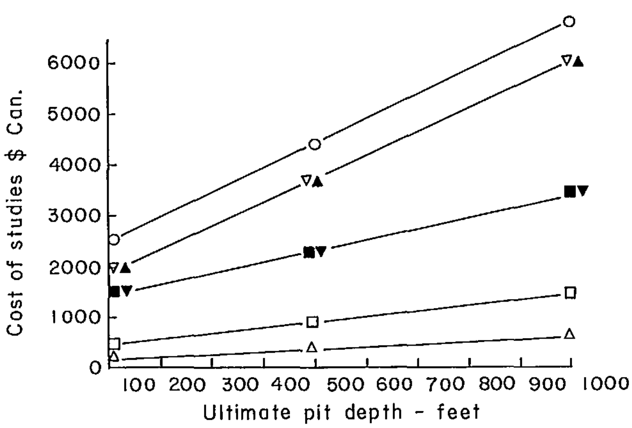
322. Precise cost and time guidelines for groundwater evaluations are difficult to prepare without knowledge of the mine concerned. For a given mine, the estimates at each stage of development will depend on the following:

a. available data
b. complexity of groundwater regime
c. significance of groundwater to slope stability
d. availability of drillholes intended primarily for other purposes such as for exploration, structural assessments, etc
e. size of property.

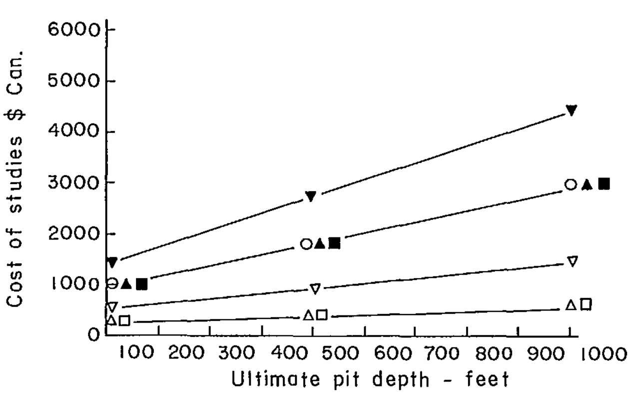
323. Approximate estimates of cost are presented in Fig 54, which shows the anticipated costs of various components of a groundwater evaluation program related to ultimate pit depth for the feasibility, mine design and operating stages of a typical open pit development. In



Feasibility stage - cost estimates



Main design stage - cost estimates



Operating review stage - cost estimates

Fig 54 - Cost estimates for groundwater investigations.

preparing these estimates, an approximately conical pit has been assumed, although experience with actual properties has been taken into account. Drilling costs, which are a major item, are not included. It was also assumed that little data for the area was available prior to the investigation, that the geological conditions were relatively simple and that the majority of the investigations could be carried out by personnel at the mine. The following aspects of groundwater studies were considered:

- △ Initial review of existing data
- □ Collation of hydro statistics (including provision of instrumentation)
- ▽ Permeability testing program within existing drill holes
- ○ Piezometer installation (including instrumentation)
- ▲ Analysis of data
- ■ Determination of the influence of groundwater on slope stability
- ▼ Evaluation of drainage requirements

324. The costs shown are cumulative. Thus, the total costs estimated for a full investigation of a 1000 ft (300 m) deep pit excluding drilling are as follows:

Feasibility stage	$24,000 (1975)
Design stage	$30,000 (1975)
Operating stage	$16,000 (1975)

325. The time required to conduct groundwater investigations depends very much on local factors such as drill rig availability as well as on the size of operation, etc. The following indicates the order of time required for carrying out groundwater assessments for the various development stages:

Phase	Ultimate Pit Depth ft (m)		
	up to 200 (up to 60)	200-500 (60-150)	500-1000 (m) (150-300)
Feasibility	4 months	5 months	6 months
Design	6 months	8 months	10 months
Operating (each review)	3 months	4 months	5 months

PART IV: ILLUSTRATIVE CASE STUDIES

A HYPOTHETICAL CASE STUDY

326. To provide a comprehensive example of the procedures discussed in Part III, the information which follows is presented as a series of reports to mine management from the geotechnical department on groundwater aspects of an imaginary porphyry copper deposit, designated the "Nanbran Mine". It is proposed that, if the investigation is satisfactory the deposit will be mined by open pit. Groundwater aspects of the feasibility, mine design and operating stages as defined in Fig 1 are presented as reports at the completion of each stage. Information has in part been abbreviated to avoid unnecessary repetition.

327. Such reports usually amplify the terms used and describe tests, etc. In the following, such detail is omitted and replaced by (#) indicating that descriptions have been given elsewhere in the chapter. Occasionally, explanatory comments are given in the text. These would not appear in a report proper and are distinguished by italic type.

328. This case study assumes that adequate time for both planning and executing the investigations, as well as money and staff resources, are available. In operating mines, however, groundwater studies may need to be undertaken and remedial measures implemented on short notice.

329. Groundwater monitoring and drainage design would apply to all slopes in a pit, but for purposes of illustration, most attention has been given to representative sections of the two main walls only.

GROUNDWATER REPORT: FEASIBILITY STAGE

INITIAL EXPLORATORY INVESTIGATION

330. The initial geological exploration program consisted of approximately 24,000 ft (7500 m) of NX core drilling and about six square miles (15 sq km) of surface mapping together with restricted areas of shell and auger drilling in the river valley area. The investigation defined a medium-sized elongated porphyry copper deposit with its long axis extending about one mile (1.5 km) north-south in a broad glaciated valley as shown in Fig 55. The deposit is cut off on the east by an extensive thrust fault, having a throw or vertical displacement in excess of 2000 ft (600 m). A river between 100 and 150 ft (60-90 m) wide flows down the western side of the valley, to the west of the orebody. Elevation of the area is between 3000 and 4000 ft (900-1200 m) above sea level.

331. A typical east-west cross-section is shown in Fig 56. The main geological components are:

a. a series of inconsistently bedded fluvio-glacial sands and gravels, with lenses of silt and silty-sands, varying from 25 to 110 ft (8-33 m) thick and averaging 60 ft (18 m). It is characterized by significant variation in permeability which is related to particle size and grading. These are designated "surficial deposits".

b. an extensive porphyritic granite, partially mineralized by copper, with accessory arsenic and antimony. The mineralized zone is elongated, extending approximately one mile (1.5 km) north to south and from 500 to 1150 ft (150-350 m) east to west, and is cut off by a fault. Some 40 joint orientation measurements from the initial boreholes have defined the following sets:

Set No.	Dip		Dip Direction	
	Range	Mean	Range	Mean
1	56° - 63°	60°	058° - 104°	085°
2	86° - 90°	89°	358° - 008°	002°
3	85° - 90°	88°	164° - 192°	180°
4	0° - 7°	3°	varies	

Massive intrusive rocks commonly have a distinctly preferential development of the near-vertical joints compared with those which are near-horizontal, and thus exhibit columnar structures. It is anticipated that the Nanbran porphyry also has this feature. Below the surficial deposits, the upper surface of the porphyry is extensively weathered and oxidized to a depth of between 10 and 60 ft (3-18 m).

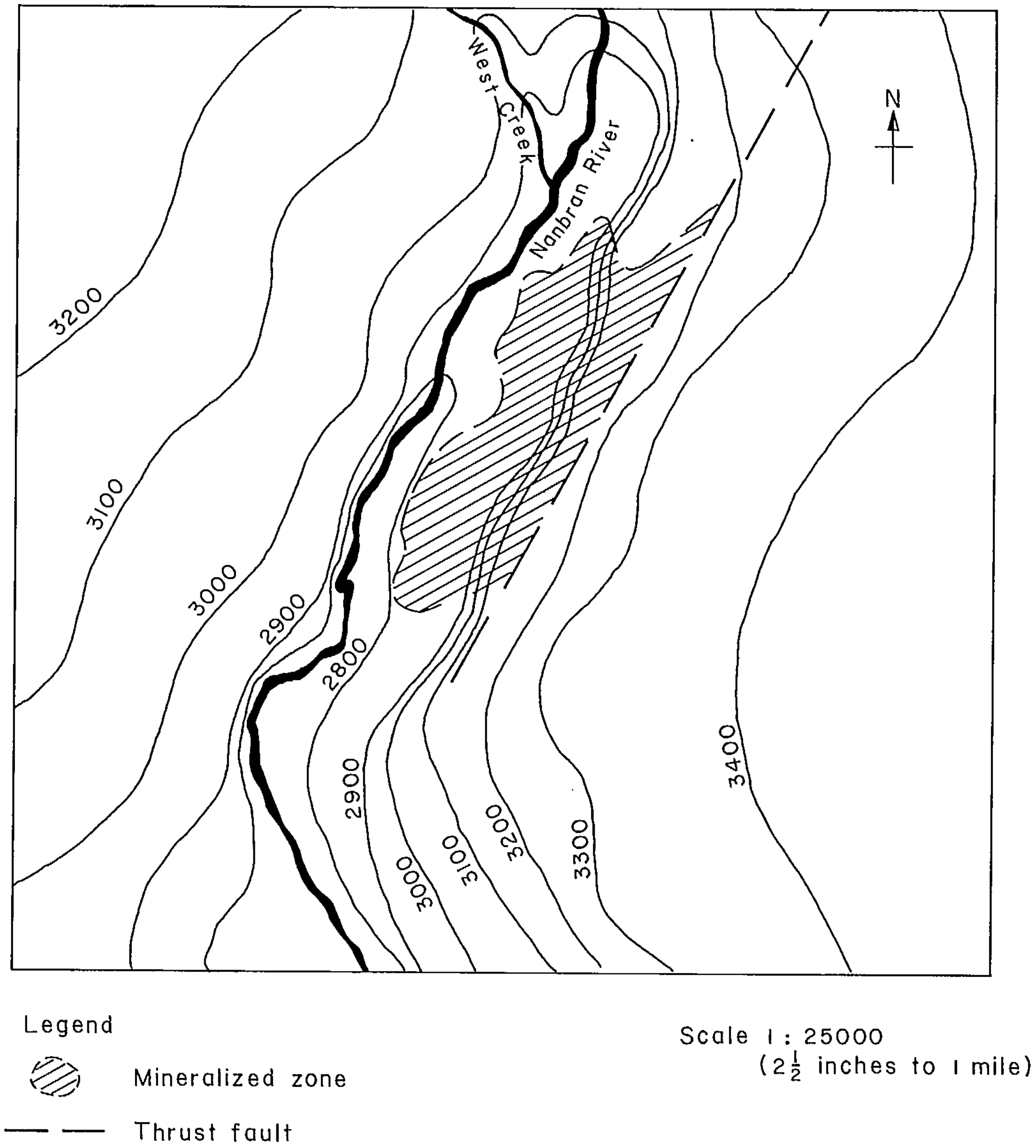


Fig 55 - Location of mineralized zone at Nanbran Mine.

c. A sedimentary series, consisting of interbedded sandstones and shales, dipping approximately 45° at 270°. The sandstones vary in thickness from 20 to 200 ft (6-60 m) and are thickly to very thickly bedded, with orthogonal joint sets normal to the bedding equally well developed and spaced 10 to 30 ft (3-9 m) apart. The shales are predominantly finely laminated.

d. An extensive microdiorite dyke between 20 and 60 ft (6-18 m) thick, dipping from 65° to 85°, averaging 80° at 090°, with a few tight joints normal to the dip.

e. A major thrust fault, trending north-south and dipping 60° to the west, which has thrown the porphyry at least 2000 ft (600 m) upwards relative to the sedimentary series and cut off the orebody. There is an extensive zone of shearing and fracturing extending about 20-30 ft (6-9 m) on both sides. Discontinuous development of clay gouge infilling has occurred.

332. Because the prime function of exploration drilling is to determine the distribution and concentration of mineralization, the degree to which

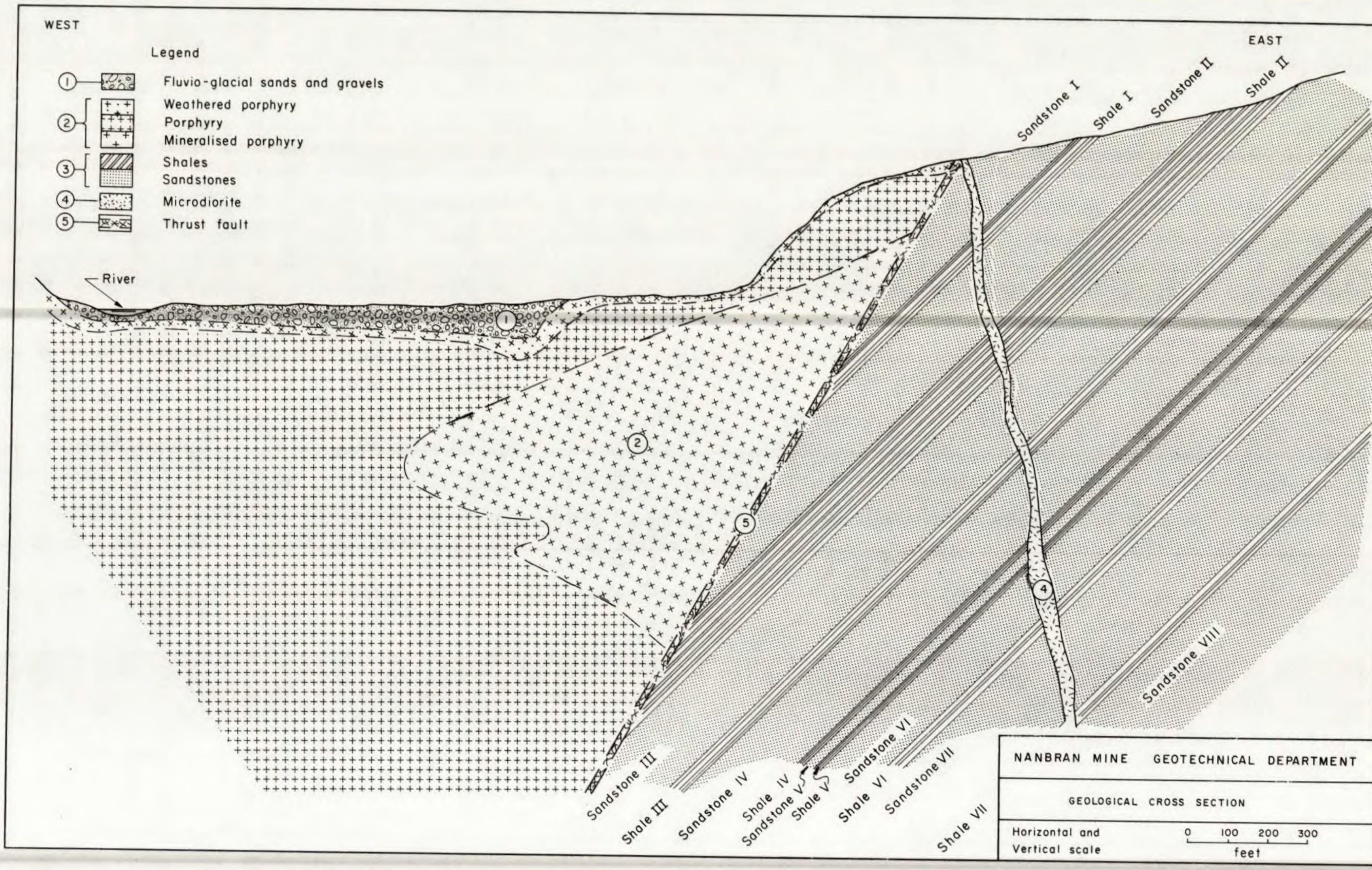


Fig 56 - Geological cross section of Nanbran Mine.

the drilling can provide useful information for geotechnical studies depends on the extent to which those responsible for the geotechnical aspects are involved in the planning and execution of the exploration drilling program.

333. If there is early involvement, much useful geotechnical and hydrogeological information can be obtained at little or no extra cost. The need for specific additional drilling for geotechnical purposes at a later date and at extra cost can, therefore, be minimized. This report assumes there has been good liaison from an early stage.

Hydrology#

334. An examination of precipitation records# available from government sources shows that the annual average precipitation has been in the range of 86 to 110 in. (220-280 cm) over the last 20 years, with a mean of 100 in.(255 cm). About 60% of the precipitation falls as snow. Temperature records# indicate that the annual temperature range is -10°C to 30°C, with generally freezing conditions from November to April. On-site weather records during the exploration period have confirmed these figures.

335. The river drains a considerable catchment area through tributary valleys, and there is extensive vegetation cover. However, the scarp to the east of the orebody is exposed, and vegetation cover on its upper slopes is more sparse resulting in limited recharge#, mainly into the sandstones.

336. Exploration drilling showed that the porphyry itself is surrounded by contact metamorphic rocks which are generally of low permeability: the river is likely to be the principal source of recharge into the porphyry through surficial deposits on the west side.

Hydrogeology#

337. Core evaluation# during exploration drilling was used to select positions for falling head tests# in 12 completed boreholes. Staged falling head tests#, carried out during drilling, were performed in three additional holes.

338. Cores were evaluated according to the following parameters:

a. rock type
b. joint frequency (number of joints per unit length)
c. joint tightness (tight, moderate, loose)
d. degree of staining (heavy, light, moderate)
e. special features, eg, enlargement of joint by solution, chemical deposits, etc.

339. From these data, representative positions for the permeability tests were selected in average rock conditions for each lithological type. Some additional tests on badly fractured and heavily weathered zones were also carried out. Results for a representative test are shown in Fig 57; a summary of falling head permeability tests is given in Table 3.

340. Normally the geotechnical report would include a map and cross sections detailing precise borehole positions. It would also have full field records in the form of borehole logs, permeability observation sheets, and the graphical plot and calculations for every test.

341. From the core evaluation, general geological appraisal and the falling head test results, the hydrogeological properties of the main rock types present were defined as in Table 4.

342. Relative permeabilities within the region may be assessed from permeability measurements using representative values where individual values have shown variation. For example, measured permeabilities in the shale fall into both the low and effectively impermeable classifications, but material considerations suggest the latter to be more realistic. Regional permeability characteristics are shown in Fig 58.

Conclusions

343. The following conclusions were drawn from the initial exploration investigation:

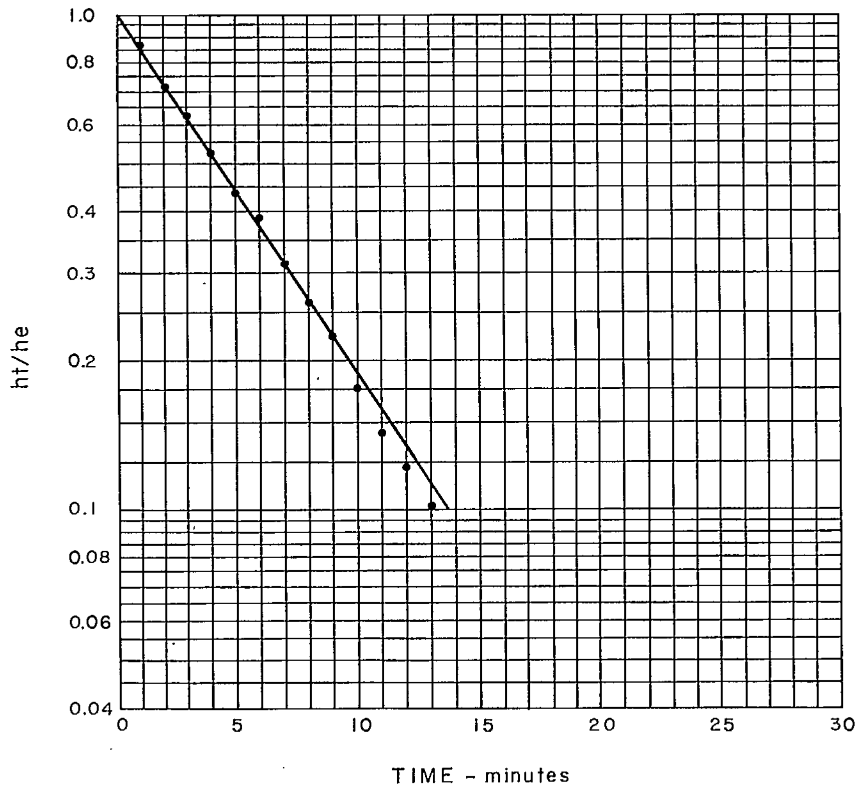
a. The proposed slopes are likely to be recharged from the west by the river through the surficial deposits and from the east by the sandstones dipping into the excavation. Because much of the rock has moderate permeability, the volume of water involved could be significant and require moderate pumping capacity.
b. The succession of sandstone and shales will result in strongly anisotropic water pressure distributions. Considering the adverse dip of

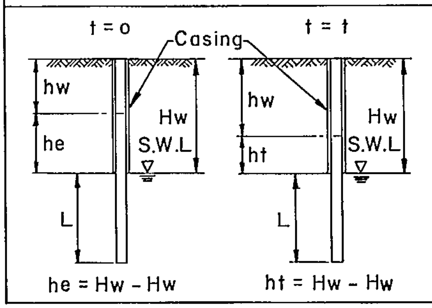
FALLING HEAD TEST	TEST No. 6	DATE 23.7.72	PROJECT NANBRAN
	ENGINEER J.F.H.		BOREHOLE E16

Borehole Co-ordinates 1114.0283 (local)	Collar Elevation 2200 FT AMSL
Depth to top of test section 490 FT.	Length of test section, L 20 FT
Depth of static water level, Hw 55.5 FT.	Radius of borehole, r(NX) 1.5 ins
Excess head, he 55.5 FT.	Radius of standpipe or casing, r_c

Time T-(min.)	1	2	3	4	5	6	7	8	9	10	11	12	13		
Depth to water, hw.(m)	8	15	21	26.5	31	34.5	38	41	43.5	45.5	47.5	49	50		
Excess head ht = Hw-hw(m)	47.5	40.5	34.5	29.0	24.5	21.0	17.5	14.5	12.0	10.0	8.0	6.5	5.5		
$\frac{ht}{he}$	0.86	0.73	0.62	0.52	0.44	0.38	0.31	0.26	0.22	0.18	0.14	0.12	0.10		

HEAD - TIME GRAPH (slope of graph is S)





CALCULATIONS

Permeability, $K = 0.133\ S\left(\frac{r^2}{L}\right)$ metres/sec.

$S = \frac{1}{13.7}$

$K = 0.133 \times \frac{1}{13.7} \times \frac{1.5^2}{20 \times 12} \times 100$ cm/sec.

$= 9.10 \times 10^{-3}$ cm/sec.

Say 9×10^{-3} cm/sec.

Fig 57 - Results of falling head test at Nanbran Mine.

Table 3: Nanbran mine: Results of falling head permeability tests

Borehole no.	Rock type	Test	ft (m)	k. (cm/sec).	Degree of Permeability
73	Porphyry	F.H.	2840(866)	3×10^{-3}	Moderate
91	Porphyry	F.H.	2910(887)	1×10^{-2}	Moderate - high
4	Porphyry	F.H.	2400(732)	2×10^{-4}	Moderate
E16	Porphyry	F.H.	2200(671)	2×10^{-5}	Moderate
28	Porphyry	F.H.	2240(683)	4×10^{-4}	Moderate
48	Porphyry	F.H.	2360(719)	3×10^{-3}	Moderate
12	Porphyry	F.H.	2200(671)	4×10^{-4}	Moderate
84	Porphyry	F.H.	2240(683)	4×10^{-4}	Moderate
9	Metamorphics	F.H.	2260(689)	4×10^{-8}	Effectively impermeable
60	Metamorphics	F.H.	2760(841)	6×10^{-7}	Low
14	Porphyry (shear zone)	F.H.	2400(732)	6×10^{-2}	Moderate - high
61	Microdiorite	F.H.	2550(777)	3×10^{-8}	Effectively impermeable
17	Sandstone	SFH	3250(991)	1×10^{-3}	Moderate
	Shale	SFH	3170(966)	2×10^{-3}	Moderate[1]
	Sandstone	SFH	3050(930)	3×10^{-4}	Moderate
	Shale	SFH	2800(853)	2×10^{-8}	Effectively impermeable
44	Sandstone	SFH	3250(991)	2×10^{-3}	Moderate
	Shale	SFH	3150(960)	2×10^{-4}	Moderate[1]
	Sandstone	SFH	3050(930)	2×10^{-4}	Moderate
22	Porphyry (shear zone)	SFH	3400(1036)	4×10^{-3}	Moderate
	Sandstone	SFH	3310(1009)	4×10^{-3}	Moderate
	Shale	SFH	3270(997)	2×10^{-8}	Effectively impermeable

F.H. = falling head test.
S.F.H. = staged falling head test.

Note [1] These results are considered to be Anomalous in this lithology, and should be disregarded.

Degree of Permeability:

High	10^{-2} to 1 cm/sec.
Moderate	10^{-2} to 10^{-5} cm/sec.
Low	10^{-5} to 10^{-7} cm/sec.
Effectively impermeable	$< 10^{-7}$ cm/sec.

Table 4: Nanbran mine: Hydrogeological properties of main rock types

Rock type	Hydrogeological properties
1. Porphyry	a. Intact rock hard, crystalline and virtually impermeable except in surface weathered zone where the intact permeability is higher. b. Water flow will be entirely restricted to the joints, except for the surface weathering zone. c. Porphyries usually exhibit anisotropic# bulk permeability#, since vertical joints better defined than horizontal. Therefore estimated $k_{vertical} >> k_{horizontal}$. d. Joints tighter at depth, therefore permeability decreasing.
2. Porphyry shear zone	a. Moderate bulk permeability due to fracturing. b. Some regions of low permeability likely where gouge material is present. c. Estimated $k_{parallel\ to\ fault} > k_{normal\ to\ fault}$.
3. Sandstones	a. Intact and bulk permeabilities moderate. b. Isotropic bulk permeability, but seepage will be strongly directed down-dip by the shale aquitards* above and below each horizon.
4. Shales	a. Strongly anisotropic permeability. b. Estimated $k_{parallel\ to\ bedding} >> k_{normal\ to\ bedding}$. c. Possibility of time-related effects due to softening and swelling. d. Weakest rock type, therefore would be likely focus of slope failure if high seepage pressures were allowed to develop.
5. Metamorphic rocks	a. Effectively impermeable.
6. Microdiorite dyke	a. Effectively impermeable. b. Although of an intrinsically strong rock, the dyke is too narrow to ensure stability when close to the excavated slope if high seepage pressures develop in the sandstones: therefore also a likely focus of failure.

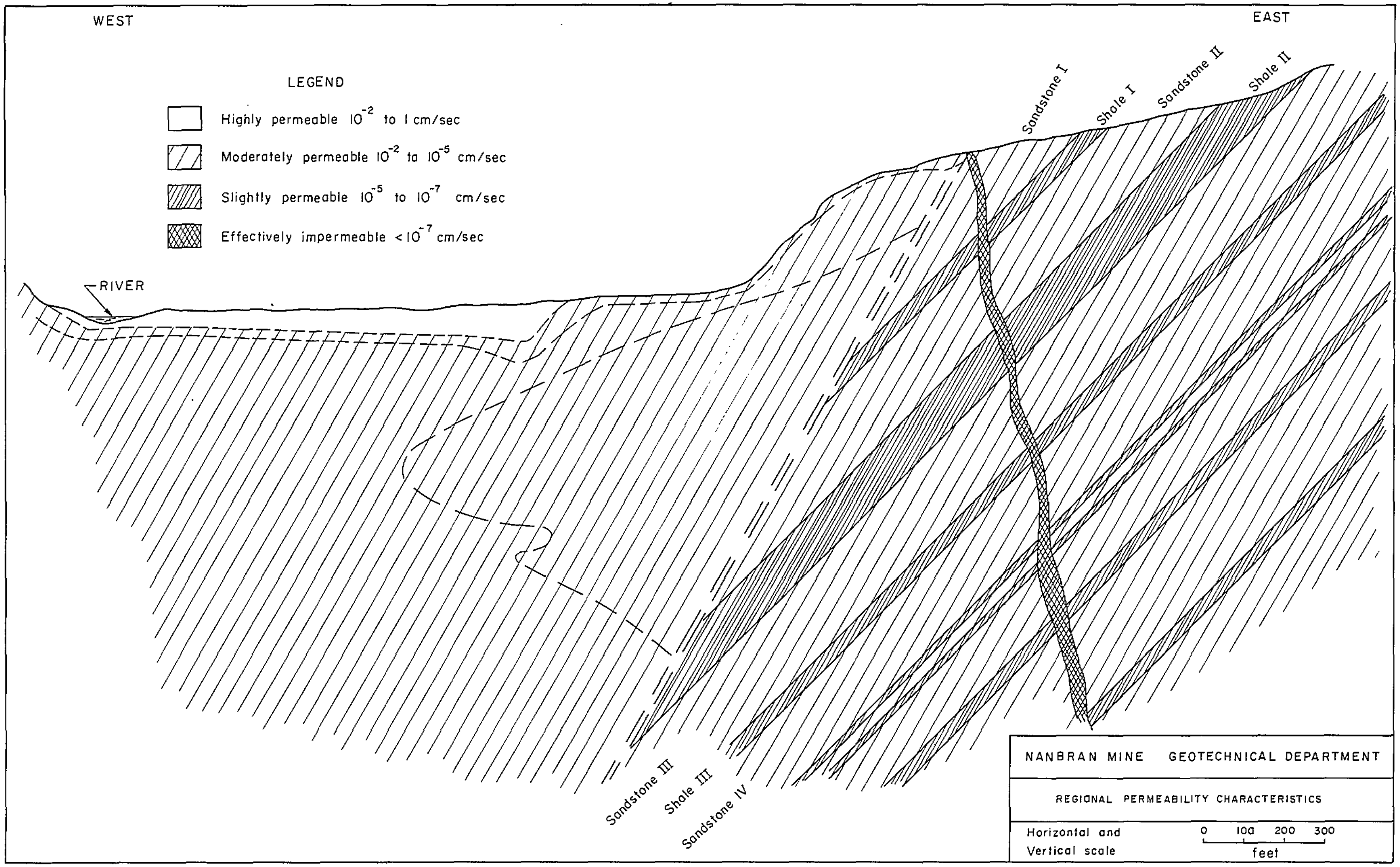


Fig 58 - Regional permeability characteristics at Nanbran Mine.

about 45° into the excavation of these formations, water pressure effects will almost certainly lead to slope instability in the less competent shales unless counter measures are taken during excavation.

Cross-reference might be made here to a Report on Preliminary Design, which, if available, would quantify the expected groundwater effect on slope stability.

c. The presence of the near-vertical impermeable barrier presented by the microdiorite dyke will also adversely affect stability by interrupting free drainage.

d. It is considered that groundwater will be a major influence on the design and operation of the pit. It is felt, however, that the problem can be solved, and the preliminary conclusion is that the project remains feasible.

e. More detailed knowledge of the groundwater parameters are required before design work can begin.

f. Continuous monitoring of slopes throughout the life of the pit will be necessary.

344. A full report on the initial exploration investigation was submitted to management with a proposal for a detailed evaluation of groundwater problems. This proposal was accepted, and results of the further investigation carried out in the following eight months are given below.

DETAILED EVALUATION

345. Figure 59 shows typical measures for the detailed evaluation, including:

a. The drilling of eight additional NX boreholes, equally spaced in pairs along the north-south axis of the orebody, each hole up to 1000 ft (300 m) in length and at an inclination of 45°. These boreholes were to facilitate:
   - core evaluation, RQD etc
   - recording details of joint frequency, size and orientations
   - constant head permeability tests# with packers# to determine permeabilities more accurately than with the falling head tests.

b. The installation of piezometers in a number of existing exploration boreholes. These were of two types:
   - pneumatic piezometer tips at various levels to measure groundwater pressures in the orebody,
   - shallow standpipe piezometers to ensure adequate definition of the undisturbed groundwater table and to measure its seasonal variations. Of greatest concern in this respect were the surficial deposits on the west side and the area on the east scarp within influence of the proposed adit.

c. In the exploration adit in the east scarp, the detailed mapping of joints, an assessment of the yield of the main rock types and probable efficiency of drainage.

d. Drilling of additional boreholes and installing piezometers to measure groundwater pressure distribution throughout the sedimentary units on the east side, with particular attention to the zone of influence of the proposed adit.

e. A well test# in the surficial deposits to determine their importance in recharging the porphyry and whether measures were required to reduce seepage from the river.

346. In practice, a geotechnical report would include a plan showing positions of the study cross sections and details of the positions of instruments on each cross section. Detailed logs would be provided for each borehole used, together with field records and calculations for every packer test and observation logs for each piezometer tip. For the sake of brevity, only a single representative report sheet for each of the above items of data is introduced at an appropriate point in the text which follows.

Information from Inclined Boreholes

347. Positions of inclined boreholes were chosen to provide the maximum information in the main area of geotechnical interest.

348. Previous experience has shown that the zone of interest for groundwater studies relating to open pit design, ie, the zone within which variations of groundwater distribution and pressure will influence the stability of the excavated slope, is approximately defined by a

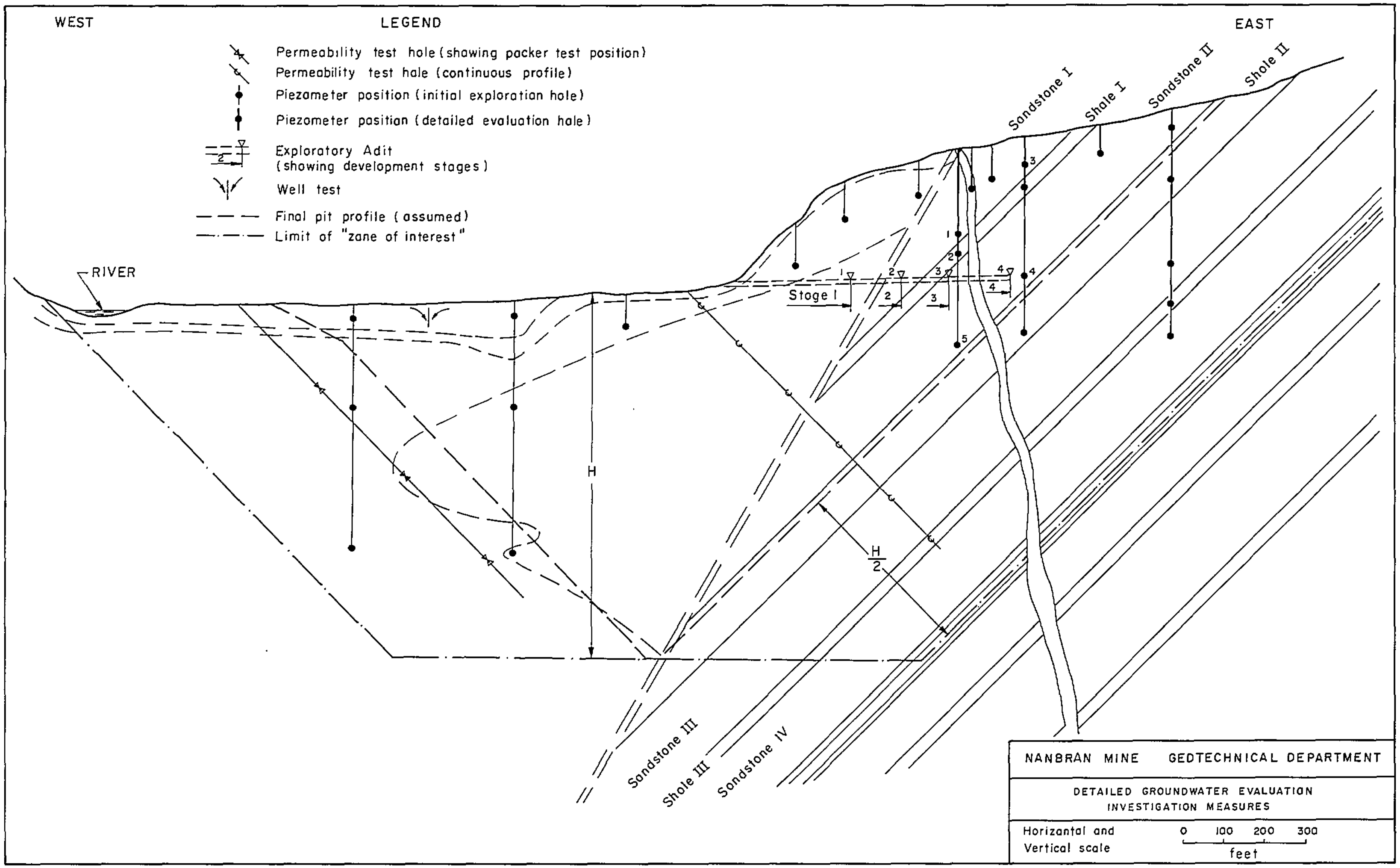


Fig 59 - Nanbran Mine detailed groundwater evaluation: investigation measures.

thickness of H/2, measured normal to the slope where H is the projected overall pit depth. This zone is defined in Fig 59. In the case of this deposit, H/2 is approximately 450 ft (135 m).

349. The inclined holes on the west side of the deposit were drilled parallel to the boundaries of the zone of interest, and about one quarter of the way into it from the planned final slopes.

350. On the east side, the inclined holes were drilled to intersect the shear zone and the sedimentary units at right angles. The inclination allowed measurements to be made of permeability parallel to the bedding. This was particularly important in the shales, where the preliminary geological assessment had suggested the likelihood of strongly anisotropic permeabilities.

351. Following core evaluation, positions of the individual packer tests were selected to provide representative information. For one central cross section, continuous packer tests - ie, over the entire hole length - were carried out to obtain complete permeability profiles through the rock units. Particular attention was paid to the shear zone associated with the fault. In the sedimentary units, positions were selected to test the permeability of each horizon in the series. A portion of a typical borehole record for an inclined hole is shown in Fig 60.

352. Thirty constant head packer tests# were carried out using standard procedures. Both mechanical compression packer and pneumatic packer installations were used. Mechanical compression packers were used for tests carried out while drilling was temporarily suspended. Pneumatic packers were used for selective tests in completed boreholes.

353. Some difficulty was experienced with the packers sticking in the boreholes at depth. The packer units were equipped with pressure transducers# to measure pressure in the test sections before, during and after the tests.

354. The mechanical seals in general worked well but in some cases were not effective because of their relatively short length; some proved impossible to remove without damage.

355. The results of a typical packer test using the pneumatic unit are shown in Fig 61. Results of these tests confirmed the previous results given in Table 3 and the permeability distribution shown in Fig 58. Core evaluation of the new holes also confirmed the relative permeabilities of the different zones.

356. Tests in the shales showed a marked non-linearity between flow rate and excess pressure over static at relatively high pressures, indicating that the partings were opening. Tests at low applied pressures indicated permeabilities parallel to the bedding of from 4×10^{-5} to 2×10^{-7} cm/sec, with an average of 3×10^{-6} cm/sec. The direction of the maximum permeability could not be determined from the tests, but was inferred from material considerations, ie, from previous experience of the directional permeabilities of shales and similar rock types.

357. Measurements in the inclined holes in the sandstones indicated a few very tight and poorly developed bedding planes. Joints normal to the bedding tended to be open and continuous. Therefore, orthogonal joints normal to the bedding were evidenced to be the important discontinuities with respect to groundwater flow.

358. In the porphyry, the joint observations from the inclined holes confirmed the existence of strongly preferential near-vertical jointing in two equally developed sets, with poorly defined, tight and inconsistently developed near-horizontal joints. Packer tests in the porphyry on sections of hole with and without horizontal joint intersections showed practically no difference in results, indicating that most of the flow occurred in the major vertical joint sets where k values between 5×10^{-4} cm/sec and 1×10^{-5} cm/sec were obtained. The average value of k in the horizontal joints only was 8×10^{-6} cm/sec. The spread of results for permeability was more limited than that obtained with the falling head tests; the variation of the falling head test data was caused by the inconsistency with which individual vertical holes intercepted near-vertical joints, and the mean figure of 8×10^{-4} cm/sec obtained for the bulk of the porphyry is

considered representative.

359. Exceptions were at the highest elevations in the porphyry where more open jointing and some oxidation and weathering resulted in marginally higher permeabilities, and the shear zone, where a mean permeability of 6×10^{-4} cm/sec was measured. Contrary to expectation, excluding the shear zone, there was no measurable decrease of permeability with depth. Comment Id in Table 4 is therefore not valid.

Information from Piezometers

360. Piezometers of the pneumatic type (Class Cl#) were installed at varying elevations in twenty of the existing exploration boreholes. Their locations were selected from an examination of the cores to provide representative information about the main rock types. Positions of individual piezometer tips in the sedimentary units were selected to monitor conditions in successive horizons to:

a. provide information on the behaviour of shales and sandstones;
b. test for possible artesian conditions#;
c. test for the development of perched# water tables;
d. establish the variation of groundwater pressure with depth throughout the sequence;
e. determine the initial groundwater table and its seasonal variation;
f. monitor the effect of driving the exploration adit.

The significance of the exploration adit piezometric data is discussed in para 365 et seq.

361. Additional monitoring facilities were provided by 24 standpipe piezometers up to 120 ft (35 m) long. Steel pipes 1.5 in. (3.8 cm) in diameter were used fully grouted above the

<table>
<tr><td colspan="2">Project
NANBRAN MINE FEASIBILITY STUDY.</td><td>Borehole No. 51
Sheet 1 of 3</td></tr>
<tr><td colspan="2">Purpose
ADDITIONAL GEOLOGICAL INVESTIGATION AND PERMEABILITY TESTING.</td><td>Type
Geological.</td></tr>
<tr><td>Datum and Level Collar 3015 ft. AMSL
Ground Level 3013 ft. AMSL</td><td>Inclination, Direction and Depth
45° East
870 ft. Depth</td><td>Co-ordinates
+3181009.99
-8197118</td></tr>
<tr><td>Machine
Seco 12
Hydraulic feed.</td><td>Core Barrel, Bit Design and Drilling Fluid
Triple Tube, with split inner barrel. Rotary diamond bit. Water flush.</td><td>Hole and Casing Diameter(s)
NX/N to 50m
NX open to 145.0m</td></tr>
<tr><td colspan="2">Key
R.Q.D. Rock Quality Designation
▼ Morning Water Level
═══ Casing Depth
——— Borehole Depth
* Scale Change
● Major Structure</td><td>Date Started
3.2.75
Date Completed
23.2.75
Depth Scale
1:100
Logged by
L.W.T.</td></tr>
<tr><td colspan="3">Remarks</td></tr>
<tr><td>Drilling Contractor
D.W.A.</td><td colspan="2">Client
NANBRAN COPPER MINES LTD.</td></tr>
</table>

Fig 60 - Nanbran Mine feasibility study: test drilling logs (continued overleaf).

Geological Log

Sheet 2 of 3 | Borehole No. 51

Downhole Depth	Instrumentation	Drilling & Casing Progress	Core Runs, Diameter & Recovery %	Discontinuities: Type	Dip	Direction	Description	Reduced Level (Thickness)	Downhole Depth
	Piezometer installed Tip at 480ft.		NX 76 75%				Medium bedded light grey medium grained weak SANDSTONE (1)	2679	480
			NX 76 80%						
		3·2·75	NX 76 100%						520
	Backfilled with bentonite cement to ground level	4·2·75	NX 76 100%						
							Slightly weathered thinly bedded dark grey moderately weak. SHALE (8)	2630	
		3·2·75	NX 76 100%						560
			NX 76 100%						
			NX 76 75%				Moderately weathered laminated black weak MUDSTONE (10)	2595	600
			NX 76 100%				Slightly weathered thinly bedded dark grey moderately weak SHALE (8)	2592	
			NX 76 100%						640
		4·2·75	NX 76 100%					2539	680

Fig 60 (continued) - Nanbran Mine feasibility study: test drilling logs.

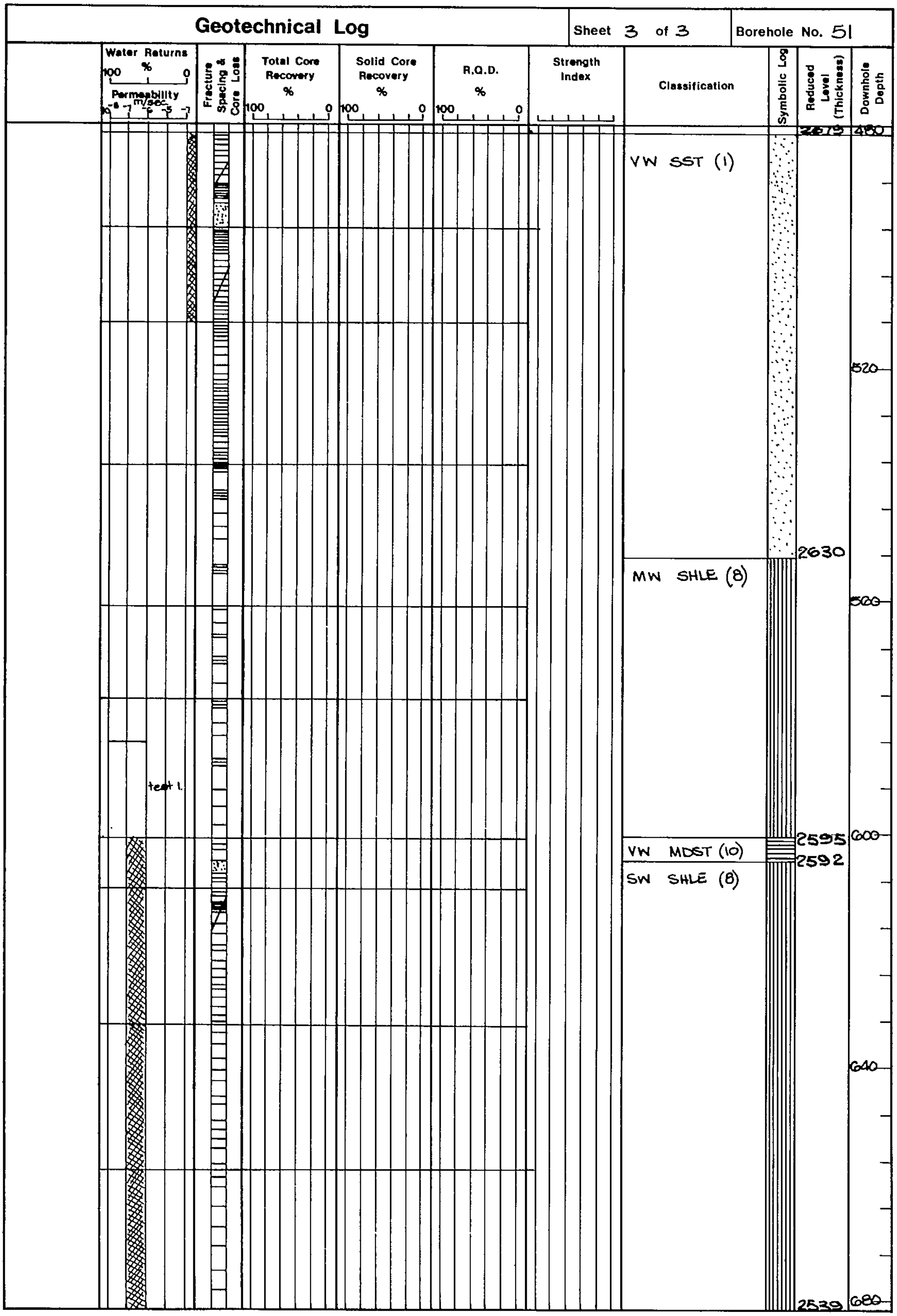


Fig 60 (continued) - Nanbran Mine feasibility study: test drilling logs.

SINGLE PACKER TEST	TEST No. 14	DATE 3.6.75	PROJECT NANBRAN
	ENGINEER J.F.H.		BOREHOLE 22

Borehole Co-ordinates		Collar Elevation	
Depth of borehole	363 FT	Length of test section, L	25 FT
Length of casing	338 FT	Radius of borehole, r	1.5 ins (NQ)
Depth of static water level, Hw	100 FT	Gauge height above S.W.L, hg	105.6 FT

Test * Pressure	Average Flow		INCREASING PRESSURE							DECREASING PRESSURE						
			TIME - minutes							TIME - minutes						
			0	1	2	3	4	5		0	1	2	3	4	5	
		Meter Reading	818.5	819.2	819.9	820.5	821.2	821.9		839.3	840.0	840.5	841.1	841.8	842.2	Av
		Take		0.7	0.7	0.6	0.7	0.7			0.7	0.5	0.6	0.7	0.4	Flow
		Flow		0.7	1.4	2.0	2.7	3.4			0.7	1.2	1.8	2.5	2.9	=0.58
		Meter Reading	823.4	824.5	825.6	826.6	827.6	828.8								
		Take		1.1	1.1	1.0	1.0	1.2								
		Flow		1.1	2.2	3.2	4.2	5.4								
		Meter Reading	830.6	832.0	833.4	835.0	836.4	837.8								
		Take		1.4	1.4	1.6	1.4	1.4								
		Flow		1.4	2.8	4.4	5.8	7.2								
		Meter Reading														
		Take														
		Flow														
		Meter Reading														
		Take														
		Flow														

1. Gauge pressure hp	16.1	133.5	251.0	16.1								
2. Gauge height hg (m)	105.6	105.6	105.6	105.6								
3. Fricton loss	6.3	8.4	10.5	6.3								
Flow (litres/min.)	0.68	1.07	1.46	0.60								
Excess head 1+2+3	115.4	230.7	346.1	115.4								

REQUIRED EXCESS HEAD = TEST PRESSURE x 2.307

CALCULATIONS

Permeability

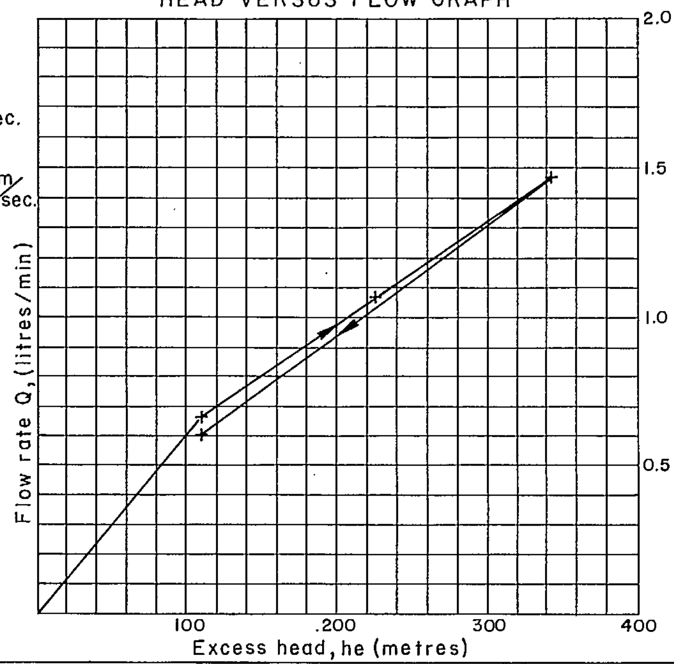
$$K = \frac{5.833}{\pi L}\left(\frac{Q}{he}\right)_o \times 10^{-5} \text{ metres/sec.}$$

$$= \frac{5.833 \cdot Q}{\pi(25 \times 0.3048)\,1154} \quad 0.68 \times 10^{-3} \text{ cm/sec.}$$

$$= 1.44 \times 10^{-6}$$

Say 1.5×10^{-6} cm/sec.

HEAD VERSUS FLOW GRAPH



Note
* All pressures to be expressed as heads ($m.H_2O$)

Fig 61 - Nanbran Mine: results of single packer tests.

collector zone.

362. The piezometer layout was selected with regard not only to its relevance to the current investigation, but also to the long-term monitoring of groundwater behaviour during the life of the pit. All the piezometers outside the eventual pit limits are expected to last the life of the mine. Particular care was taken during their installation to ensure long-term effectiveness, and all the surface positions were protected by a concrete collar, clearly marked and fenced. The standpipes outside the eventual pit limits were similarly treated.

363. Pneumatic Class C1 type piezometers were selected for the deep holes for the following reasons.

a. Fairly rapid changes in piezometric level# in response to seasonal variations and mine development were anticipated. The majority of the rocks to be tested have medium permeability, so that high volume demand# instruments would be unsatisfactory. This ruled out Class A instruments, whereas C1 instruments offered adequate response times for permeabilities down to 10^{-8} cm/sec.

b. The topography ruled out the use of closed tube (Class B#) types.

c. Long term monitoring was envisaged, and C1 instruments offer long term stability.

d. Multi-stage installations required a small unit.

e. In view of the winter freeze period, an air operated device# was considered preferable to an hydraulic type#.

364. Information obtained from the near-surface piezometer tips indicated vertical seasonal variations in the groundwater table. In the surficial deposits this was 12 to 15 ft (4-5 m) between mean midsummer and spring levels, the latter being measured immediately after the spring thaw. During the freeze period it was frequently impossible to obtain any readings from the standpipe piezometers. During about 15 days immediately following the thaw, groundwater levels generally rose to surface, particularly in the surficial deposits, thereby indicating complete saturation to ground level. This observation indicates the likely development each year of conditions that will be highly detrimental to stability, particularly in the overburden, and the long term monitoring programme must be related to this factor. A typical near-surface piezometer tip response over 12 months is shown in Fig 62.

Information from Exploratory Adit

365. A nominal 7 ft (2.2 m) high by 5.5 ft

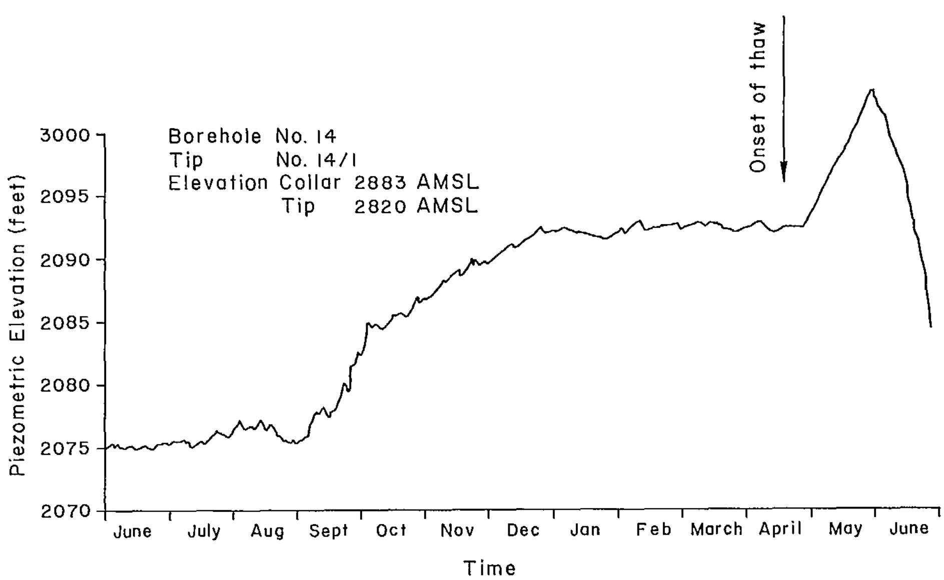


Fig 62 - Nanbran Mine: typical response of a near-surface piezometer tip.

(1.8 m) wide adit was driven into the east scarp. The prime purpose of the adit was to obtain ore samples. It was agreed to extend it into the sedimentary units to obtain:

a. structural information and samples for preliminary slope design,
b. hydrogeological information.

366. To suit the groundwater investigation, the adit was developed in four stages, each being selected to provide groundwater and structural geological information in each rock type encountered, and to indicate the regional response in the area around the adit to the preferential drainage path provided by it. The latter information was necessary to determine to what extent stability of the east wall of the pit could be improved by installing drainage adits during mining. The cost effectiveness of drainage as an alternative to more conservative slope geometry could thus be estimated.

367. Monitoring of piezometric levels was carried out daily, both to build up information on seasonal variations (see above) and to monitor behaviour during adit development. At the end of each stage of adit development, mining ceased until the piezometer readings became constant or nearly constant, indicating that steady-state conditions had been reached. This took from one to seven weeks, depending on the rock type.

368. The adit was developed at a slight incline to facilitate drainage by gravity. Drainage was led via a settlement tank and vee notch weir to an existing stream. Daily vee notch readings were taken and are shown in Fig 63. The results showed

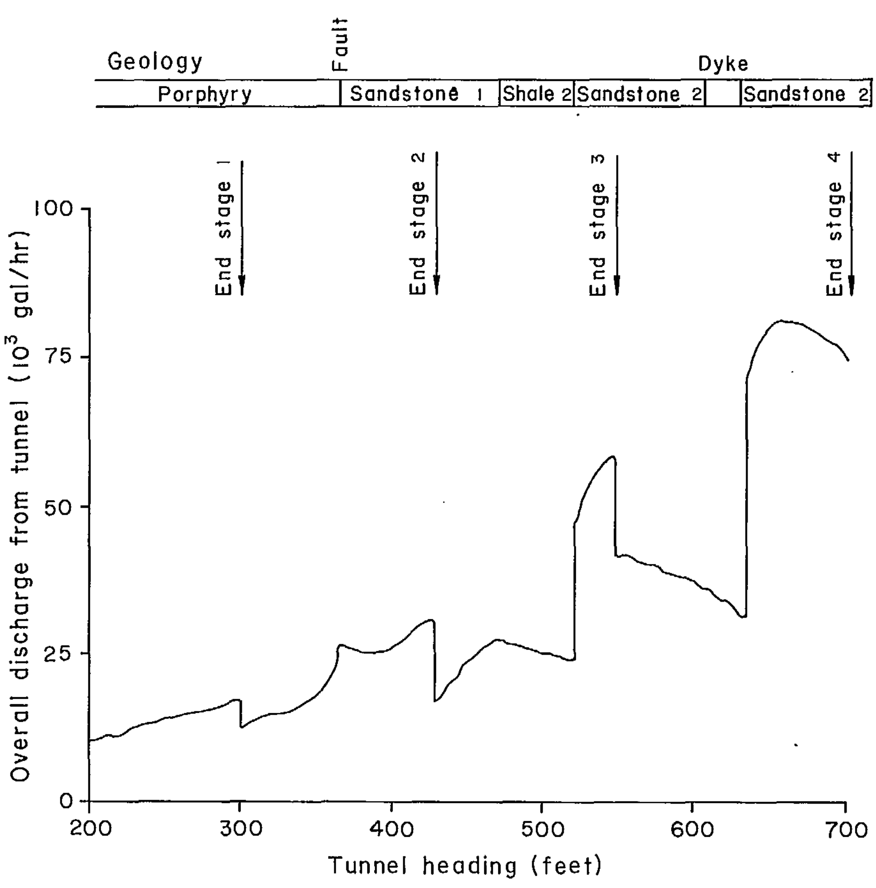


Fig 63 - Nanbran Mine: tunnel discharge plotted against heading distance.

a more or less linear increase in discharge rate with tunnel length through the porphyry, accelerating as the fault was reached. Initially, the drainage rate from Sandstone I increased at the same rate but slowed as Shale I was approached, indicating that the dyke was preventing effective drainage of the portion of Sandstone I east of the dyke. Discharge rate progressively decreased while the heading was in shale.

369. Sandstone II exhibited behaviour similar to Sandstone I, but to a more marked degree. After an initial high inflow, drainage began to decrease markedly until the adit passed through the dyke. On penetrating the dyke, high groundwater pressures and significant initial flows were encountered. The flows gradually decreased as Sandstone II east of the dyke began to drain. The data shows clearly the influence of the low permeability shales and the virtually impermeable dyke on the groundwater behaviour. The dyke divides the groundwater flow into two distinct regions: the downstream region towards the portal and a second region upstream of the dyke.

370. The mean phreatic surfaces measured at each stage of adit development can most conveniently be expressed diagrammatically. Those in the porphyry and the sandstones are shown in Fig 64.

371. As the adit was driven through the region downstream of the dyke, a progressive rise in the rate of discharge from the heading was observed, and progressive drawdown in the area west of the dyke was indicated by piezometer 1 ($GWT_0 \rightarrow GWT_1 \rightarrow GWT_2$). At the same time, little change was observed in the piezometric levels upstream of the dyke. For example, at the end of Stage 2 of adit development, the phreatic surface vertically above the tunnel end had dropped about 120 ft (35 m) in Sandstone I but eastwards from the dyke the phreatic surface had not responded at all.

372. Some typical piezometer response curves together with the basic geology of the adit and the positions of each stage completion are shown in Fig 65. Positions of the individual tips 1 to 5 are shown in Fig 64.

373. The following information may be obtained from the curves in Fig 65:

a. Comparing the rate of adjustment of the groundwater levels between piezometer tip 2 in Shale I and tips 1 and 5 in the sandstones, indicated by steepness of the falling portions of the curve, confirms a much greater rate of dissipation of water pressure in the sandstones than in the shales, as would be expected from the greater permeability of the former.
b. Tip 3 in the portion of Sandstone I east of the dyke, which should show a similar rate of response to tips 1 and 5, exhibits a much slower response; this is due to the effect of the dyke and the underlying shale unit impeding to a considerable degree free drainage of that portion of the sandstone.
c. Under free draining conditions, piezometer tips 1 and 5 would simultaneously have indicated the commencement of drawdown. However, it is apparent from Fig 65 that tip 5 did not respond until the adit had passed through Shale I. This indicates that the shales, because of their low permeability, are effectively dividing the groundwater levels in the sandstones into separate regions. The significance of this fact in planning the pit design is that any method to improve stability by drainage must ensure that the boreholes, adits etc, pass through the shale into the sandstone behind to ensure effective drainage.

Information from Well Test

374. To determine the hydraulic properties of the fluvio-glacial cover, a well was drilled by a contractor specializing in well development. The well terminated approximately 10 ft (3 m) below the top of the unweathered porphyry. Eight fully grouted standpipe piezometers were installed around the well in four sets of two, radiating from the well head in each quadrant. Each pair was 100 ft (30 m) and 250 ft (75 m) respectively from the well head.

375. A series of tests at different steady rates of pumping from the well was carried out, the cone of depression# - ie, radial drawdown - being determined from the surrounding piezometers when steady-state conditions were reached. From

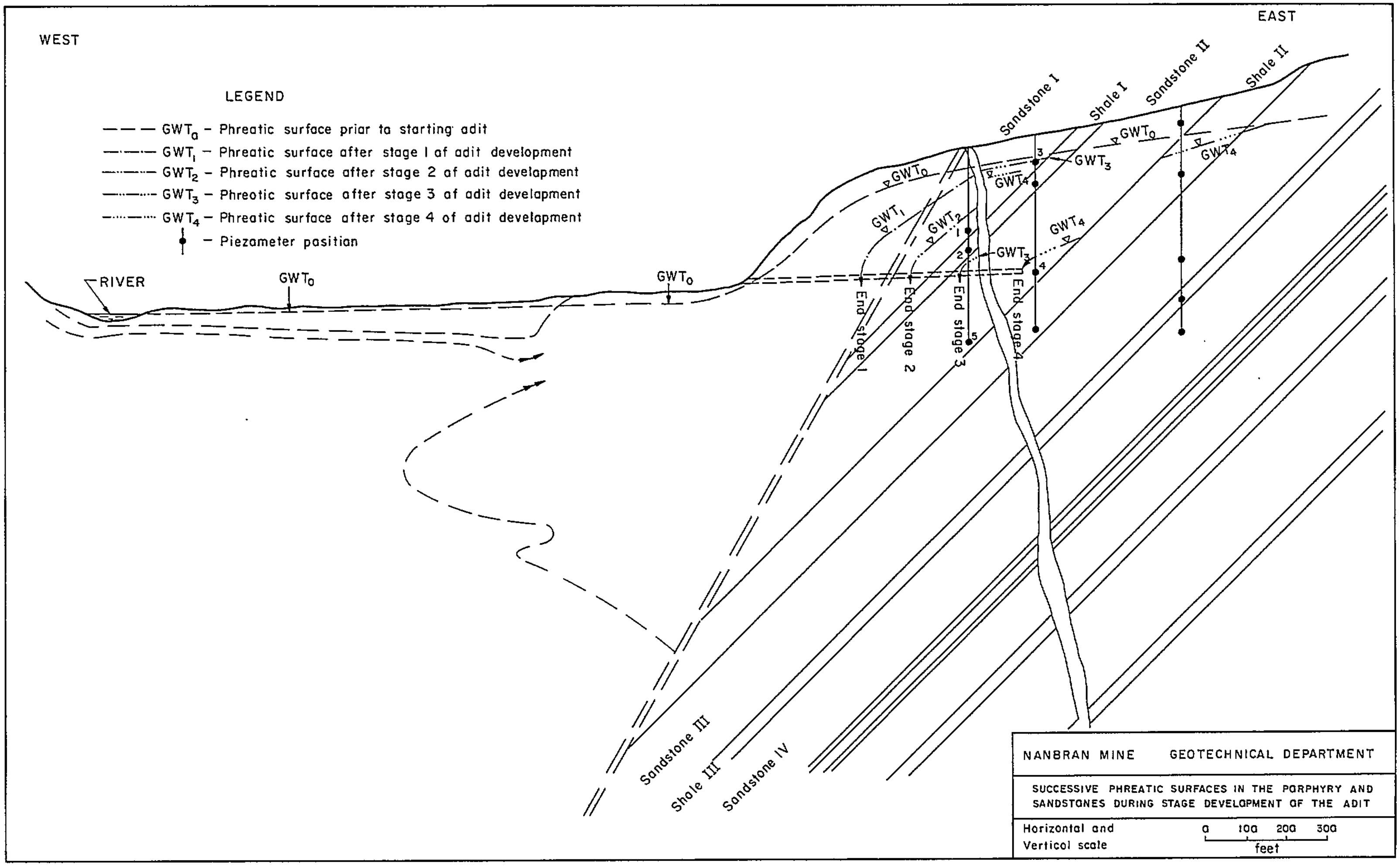


Fig 64 - Nanbran Mine: successive phreatic surfaces in the porphyry and sandstones during stage development of the adit.

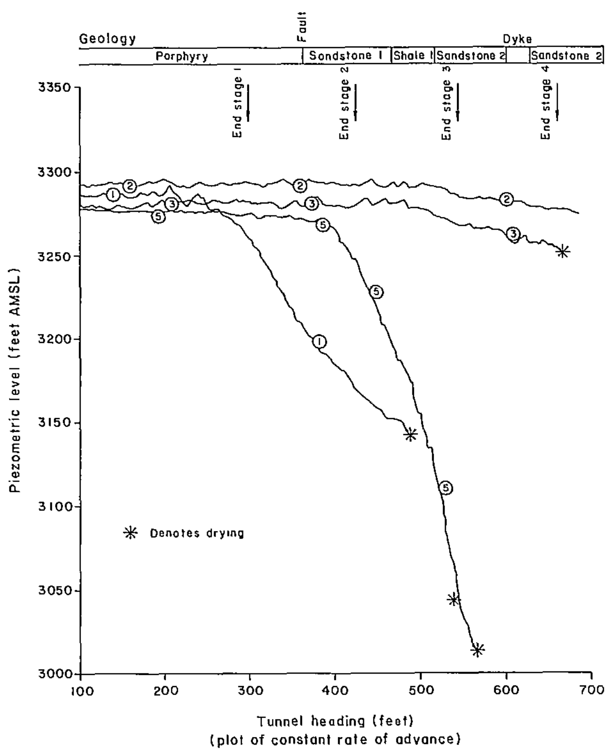


Fig 65 - Nanbran Mine: typical piezometer response curves.

these observations a mean bulk permeability of 1 x 10^{-2} cm/sec was calculated, the range being 8 x 10^{-2} cm/sec to 6 x 10^{-3} cm/sec.

376. Construction of flow nets in radial coordinates# was used as a basis for calculating the net potential recharge through the surficial deposits in the proposed slopes along the western side of the orebody. The mean quantities so obtained were 3300 gpm (15,000 lpm) under average conditions and 4000 gpm (18,000 lpm) under the most adverse conditions, using the mean permeability value obtained from the well tests.

377. Based on the above results, some form of groundwater control will probably be required, both to maintain stability of the slopes and to minimize long term pumping requirements.

Conclusions

378. The following conclusions were drawn from the detailed evaluation:

a. Further testing confirmed the validity of the ranges of hydrogeological properties determined by the initial exploration investigation, summarized in Tables 3 and 4, except that the permeability of the porphyry is no longer considered to decrease with increasing depth.
b. The additional permeability tests enabled relative permeabilities to be inferred as follows:

Shales:

$k_{\text{parallel to bedding}} > k_{\text{normal to bedding}}$

Sandstones:

$k_{\text{parallel to bedding}} < k_{\text{normal to bedding}}$

Porphyry:

$k_{\text{vertical}} > k_{\text{horizontal}}$

Dyke:

Virtually impermeable.

c. Free movement of groundwater down-dip in the sedimentary units is substantially restricted by the microdiorite dyke.
d. The shale beds act as aquitards# and inhibit the vertical movement of groundwater in the sandstones; therefore, despite the greater

permeability normal to the bedding of the sandstones, the predominent flow direction will be controlled by the shales.

e. The fact that groundwater pressures react very slowly in the shales and in areas confined by them or occluded by the dyke, could have important repercussions during mining. A high rate of bench advance in such an area would lead to temporary high water pressures close to the surface of the slope and instability could result. Local slope drainage may therefore have to be considered.

f. In considering slope drainage, two factors must be borne in mind. First, because groundwater conditions in the sandstones react very slowly through the shales, a sandstone can only drain if it is intersected directly by the drainage system. Second, significant improvements in stability due to slope drainage will only occur if the drainage measures extend through the dyke where it lies close to the excavated slope.

g. Because of the significant rates of potential flow through the surficial deposits, a method of preventing, or substantially decreasing, the recharge from the river through the surficial deposits should be considered.

h. An effective catchment drain should be installed around the perimeter of the proposed open pit to intercept both surface water and seepage through the surficial deposits, thereby minimizing pit pumping and recharge into the slope.

GROUNDWATER REPORT: MINE DESIGN STAGE

379. In practice, the groundwater aspects of mine design can in no way be divorced from the overall design, because groundwater information is merely one item of input data required for a rational stability evaluation. Methods for overall stability analyses and slope designs are covered in depth in the Design chapter, and are not reiterated here. This section attempts to summarize briefly the groundwater factors which must be incorporated in the overall design, without itself purporting to be the complete design; many assumptions, therefore, have to be made regarding other aspects of the overall design. The two primary functions of the design stage are:

a. to provide estimates of groundwater conditions at various phases of the mine development

b. to determine the most cost-effective methods of groundwater control should the stability studies indicate they are necessary.

FIELD GROUNDWATER STUDY

380. Whether or not a field groundwater study (Fig 1) is necessary as a separate aspect of the Design stage depends firstly on the thoroughness of instrumentation installed during the feasibility study and secondly on the degree to which the information from the feasibility study is considered to be representative of the whole mining area.

381. In other words, the evaluation of groundwater conditions is to a certain extent an iterative process. Preliminary mine design work and a considered evaluation of the degree of uniformity of the geology and hydrogeology may indicate areas of uncertainty where additional studies are required to "firm up" the input information, following which the improved information is used for more exact design studies, and so on. In this instance, it is considered that the nature of the preliminary investigations was sufficiently comprehensive to make further evaluation unnecessary.

CURRENT AND FUTURE GROUNDWATER CONDITIONS

382. Current conditions have been adequately determined in the feasibility study with regard to:

a. variations of bulk permeability of component rocks and surficial deposits

b. predominant geological structural features, which in turn control hydraulic behaviour in the rock mass

c. varying degrees of anisotropy in the component rocks, and the effect of anisotropy on groundwater pressure distribution and drainage responses

d. main sources of recharge to the proposed pit slopes and, in the case of the west side of the deposit, some guide to the quantity of water that would be involved

e. seasonal variations in precipitation and temperature and identification of the freeze-thaw period as having a crucial effect on groundwater flow and slope stability.

383. The prediction of future groundwater conditions was carried out by means of an analytical model using the data obtained during the feasibility stage. The proposed excavation phases, as shown in Fig 66, were simulated and the groundwater conditions for each phase were predicted from the model. The model consisted of two 2-dimensional finite element analyses, one for each of the east and west slopes.

Finite Element Analysis: East Slope

384. The two-dimensional seepage program# was used for this study. The region modelled is shown in Fig 67(b). The permeability characteristics were derived from the field test data as shown in Fig 58. Boundary conditions at the eastern 'edge' of the model were determined from field piezometric data. It was conservatively assumed that open pit operations would have no influence on the groundwater conditions at this distance.

385. Steady state analyses were carried out for each excavation phase, assuming natural drawdown only. The results of one such analysis are shown in Fig 68. The results clearly demonstrate the significant influence of the effectively impermeable dyke and shale units. Similar conditions were observed during driving of the exploration adit.

386. Normally results for all phases would be given. For the sake of brevity, only results at completion of phase 2 are shown.

Finite Element Analysis: West Slope

387. For the west slope, plan modelling was considered appropriate. The equipotential distribution was determined using measured groundwater table levels and permeabilities. The position and elevation of the river determined the western boundary condition. A series of steady state analyses was carried out to study the effect of predicted seasonal variations, including the transient condition during the spring melt due to flow through the unsaturated upper zone of the surficial deposits.

388. A series of analyses was carried out for each excavation phase assuming natural drawdown only.

389. To determine the effect on slope stability, the resulting flow nets in radial coordinates were transformed onto vertical cross sections at representative locations. From these numerical analyses, instabilities due to adverse groundwater conditions were predicted for all phases of pit development. Some method of groundwater control will therefore be required before overburden stripping and mining can commence.

390. The plan models also defined some preferential flow paths through the surficial deposits, indicated by concentrations of flowlines, thereby producing some useful data for the preliminary design of the seepage control measures (para 414 et seq).

Results of Stability Studies

391. The groundwater pressure distributions obtained from the seepage analyses were used as input to the overall slope stability study. The following results, pertinent to groundwater conditions, were obtained.

392. The seepage analyses indicated that, under severe recharge conditions, such as during a thaw, the west slopes could become fully saturated. Under these conditions, although large scale instabilities at the anticipated mining angles were unlikely, the probability of large blocks or wedges sliding on individual benches was very high. This could result from hydraulic driving forces# acting on wedges formed by adverse combinations of the vertical and horizontal porphyry joints and the bench face.

393. Probable rotational shear instability was indicated in silty portions of the superficial deposits where the overburden cut was deep, for example at slope angles greater than 25° for a face height of 100 ft (30 m). These conditions

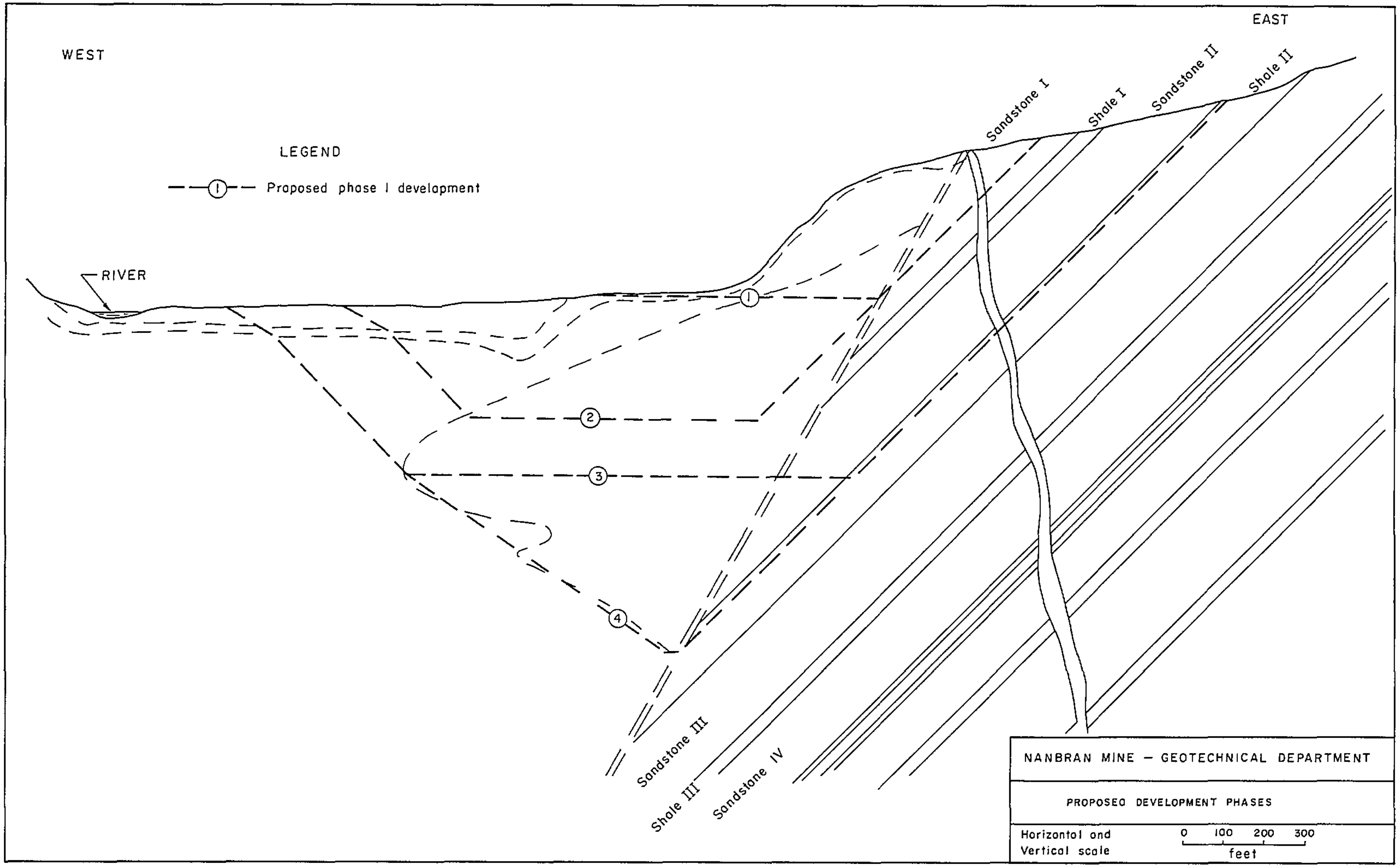


Fig 66 - Nanbran Mine: proposed development phases.

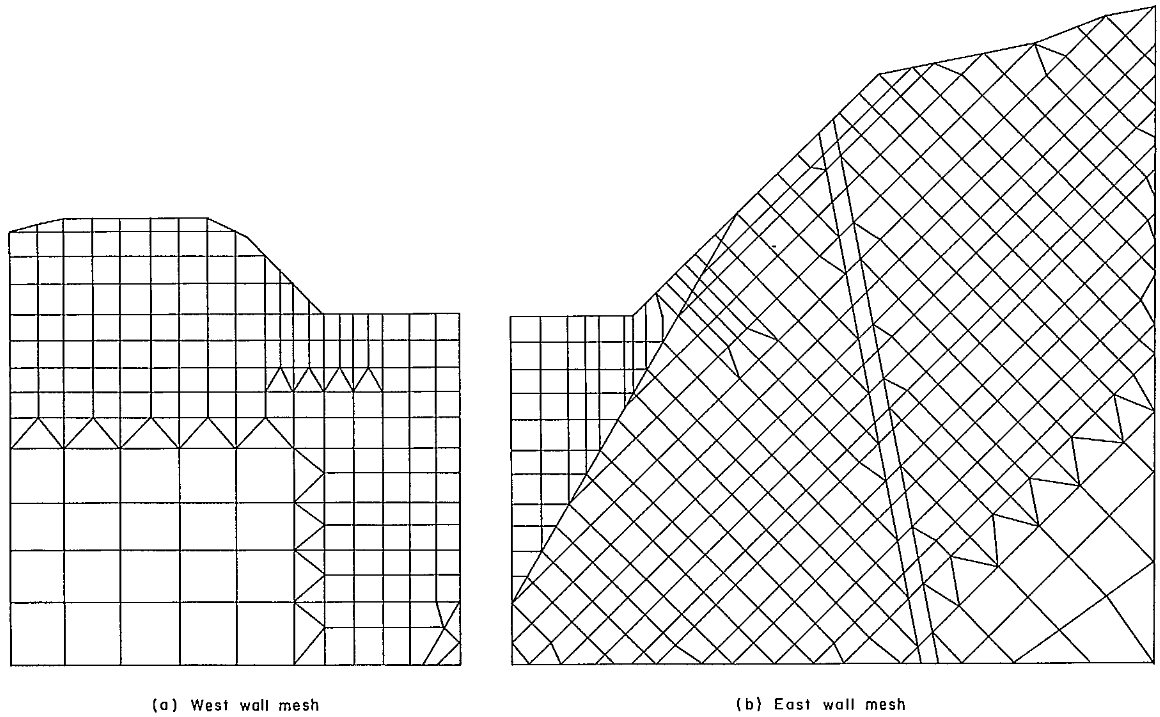


Fig 67 - Nanbran Mine: finite element meshes for seepage analysis.

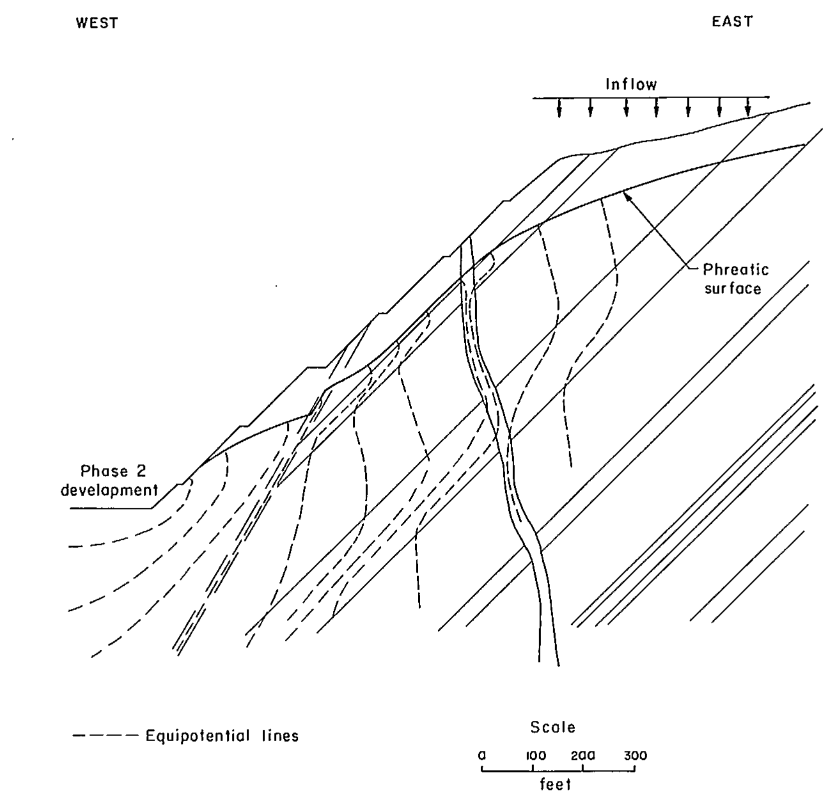


Fig 68 - Nanbran Mine: seepage analysis: phase 2 east slope equipotential distribution.

are indicated in Fig 69. Further analyses indicated that, to provide an adequate degree of stability at all times, measures would be necessary to ensure a groundwater pressure distribution represented by curve 2. Two possible methods were evaluated:

a. drainage

b. reduction in seepage from the river.

Clearly, operational considerations would favour the latter method, because the drainage measures required would be extensive. It was therefore decided to examine seepage control first.

394. Regarding the east slopes, stability analyses showed that, because of structural constraints, 45° slopes are the steepest that can be developed. Steeper angles would significantly increase the risk of large-scale planar instabilities within the shales or on shale/ sandstone contacts because the 45° bedding of the sedimentary rocks then "daylights" into the face.

395. At bench angles of 45°, the groundwater pressure distribution would result in instability at several critical stages during the anticipated pit development. Two methods of achieving an acceptable degree of stability were evaluated:

a. flattening the overall slope angle

b. providing slope drainage.

396. A series of stability re-evaluations using the same groundwater pressure distributions but modified slope geometry indicated that the slope angle would need to be reduced by between 5° and 8°, depending on the mode of instability. This would require the removal of more than 7 million cubic yards (5.5 million cu m) of additional waste from the east side of the pit, at a cost, in net present value terms, of over $2.5 million.

397. The potential instability mechanism was determined to be a slab slide either through toe rupture of the rock, or along a low angle joint in the sandstone or shale units. Two drawdown curves

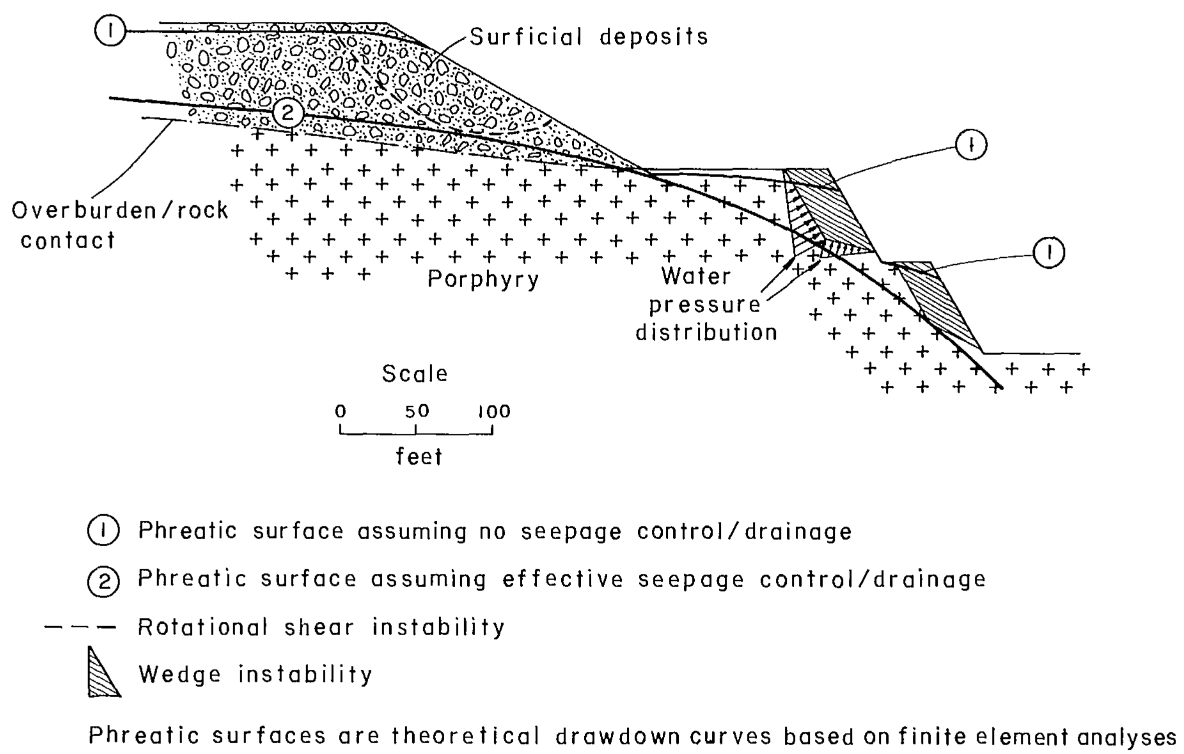


Fig 69 - Nanbran Mine: possible instability modes for west slope.

were derived for this phase of development as shown in Fig 70. Drawdown curve 1 is based on measured permeability values in Shale I and the microdiorite dyke and is the predicted drainage pattern resulting from the excavation only, ie, with no artificial drainage. Curve 2 assumes that sufficient horizontal drainage has been provided through those units to develop normal drawdown in both regions of the sandstone, ie, the effective permeability of Shale I has been made equal to that of Sandstone I. These drawdown curves were obtained by means of finite element analyses. With no artificial drainage (curve 1), critical stability conditions were shown to be possible at the contacts between Sandstone I/Shale I and Shale I/Sandstone II, termed slide mechanisms A and B respectively in Fig 70. It was shown that, provided the average groundwater conditions in the slope could be maintained below those represented by curve 2, the required degree of stability could be maintained under all conditions. This criterion was adopted in designing the drainage measures, as described below. The groundwater pressure distributions on the potential slide surfaces under both conditions are shown in Fig 71.

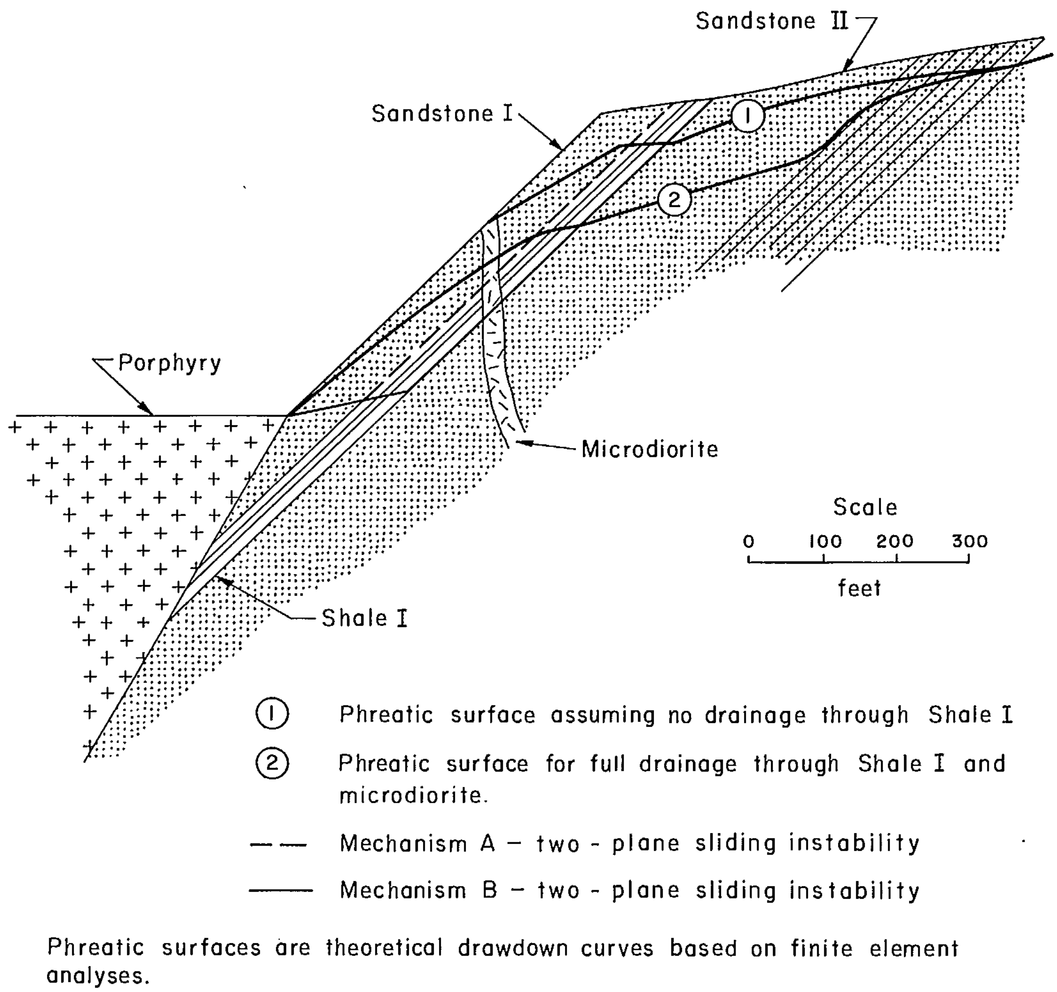


Fig 70 - Nanbran Mine: possible instability modes for east slope, phase 1 development.

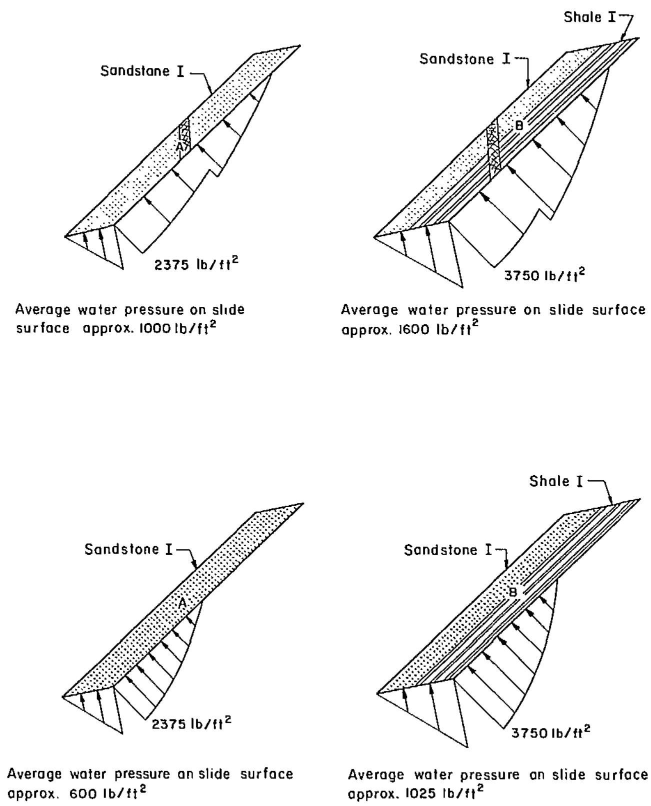


Fig 71 - Nanbran Mine: theoretical water pressure distributions on base of potential instabilities.

GROUNDWATER CONTROL AND DRAINAGE

398. Following an overall stability study by the mine geotechnical department, the following conclusions were reached with respect to possible groundwater control:

a. Because of permeability variations, the eastern slopes in the sedimentary units will require an effective drainage system to allow economic slopes to be mined. The drainage system must be monitored to ensure proper operation.
b. The high seepage rates expected through the surficial deposits adjacent to the river will adversely influence stability of the west side overburden and porphyry slopes, as well as incurring significant mine dewatering costs. A method of substantially reducing the potential seepage should be determined.
c. The western porphyry slopes would be temporary as the mine developed and final slopes could thus be determined by means of monitored trials. Control of groundwater in the overlying overburden materials will be essential to:
   i prevent excessive recharge into the porphyry slopes below
   ii permit natural drawdown to occur.

   The need for additional drainage measures should be determined following the initial excavation stages because such measures are not critical in terms of the current design. Slope steepness could be reduced slightly below target if necessary to ensure stability without adversely affecting operating costs.
d. North and south final slopes would be in the margins of the mineralized zone within the porphyry, and would have similar permeabilities. Because moderate adjustments to ensure stability may be made to overall slope angles with only small effects on the waste/ore ratio, the expense of slope drainage measures to result in

steeper stable angles is not considered warranted.

e. Surface run-off should be controlled by perimeter drains advanced ahead of pit development.

The practical aspects of conclusions b, c and e are considered below.

PRELIMINARY DRAINAGE DESIGN - EAST SLOPE

399. The exploration adit clearly demonstrated the effect of the shales and the microdiorite dyke in preventing free drainage from the sandstones. For effective drainage it is essential that the system should be varied according to circumstances, and geological input must be used to determine the layout for each part of the pit. A general parameter, determined on the basis of the initial seepage analyses, is that drainage of the sandstone must be obtained as far as 100 ft (30 m) into the slope for the proposed Phase 1 development.

400. Three alternative drainage designs were considered, namely drainage adits, pumped wells, and horizontal drainholes. Owing to the geometry of the effectively impermeable shale horizons and the microdiorite dyke, considerable flexibility in layout was required.

401. Drainage adits were not considered feasible because of the numbers of levels at which they would be required and their lack of flexibility.

402. Pumped vertical wells were investigated in detail. Because a large number of installations would be required, estimates indicated this method would be more expensive than horizontal drain holes. Failure of the pumping system, if the electrical supply failed, was also considered a disadvantage.

403. Horizontal drainholes, intercepting the major sandstone horizons, proved to be the best system for conditions on the east wall. Such a system could readily be adapted to the complex geological conditions and could be refined progressively in detail during the operating stage.

404. Re-evaluation of drainage measures to promote stability was carried out, the distribution and length of drainage holes, inclined slightly upwards, being altered to provide the required degree of stability.

405. It was concluded that this could be obtained using arrays of holes typically spaced horizontally at 60 ft (18 m) and at vertical intervals of approximately 100 ft (30 m). Because the drainage improvement is not greatly influenced by borehole diameter, NX holes were selected pending a detailed review of drilling equipment suitable for such installations.

406. The mining plan indicated that even for the early stages of pit development, relatively steep slopes were required on the east wall to minimize stripping ratios. Thus, drainage of the intermediate east wall slopes was necessary. The experience gained from this intermediate stage would form the basis for the drainage design of the final wall.

407. The general arrangement of drainage holes for Phase 2 of the pit development is shown in Fig 72. The layout indicates the way in which the hole depth must be varied according to geology. Lines A and B extend 100 ft (30 m) into Sandstone II from the shale horizon and the dyke respectively. If line C were drilled only 100 ft (30 m) east of Shale I, little drainage of the portion of the sandstone east of the dyke would be achieved. Therefore, it should be extended through the dyke into sandstone; furthermore it is considered advisable to extend it into Shale II to relieve groundwater pressures in that horizon ahead of future excavation (Phase 3).

408. The general hole depth required for the initial stages is 240 ft (75 m), with the exception of lines B and C, where 280 ft (85 m) and 400 ft (120 m) respectively are required. It should be emphasized that:

a. Hole depth should be varied according to the actual geology encountered.

b. Layout will vary along the length of the slopes.

c. The design is preliminary only and may require modification in the light of experience gained from monitoring carried out during pit development.

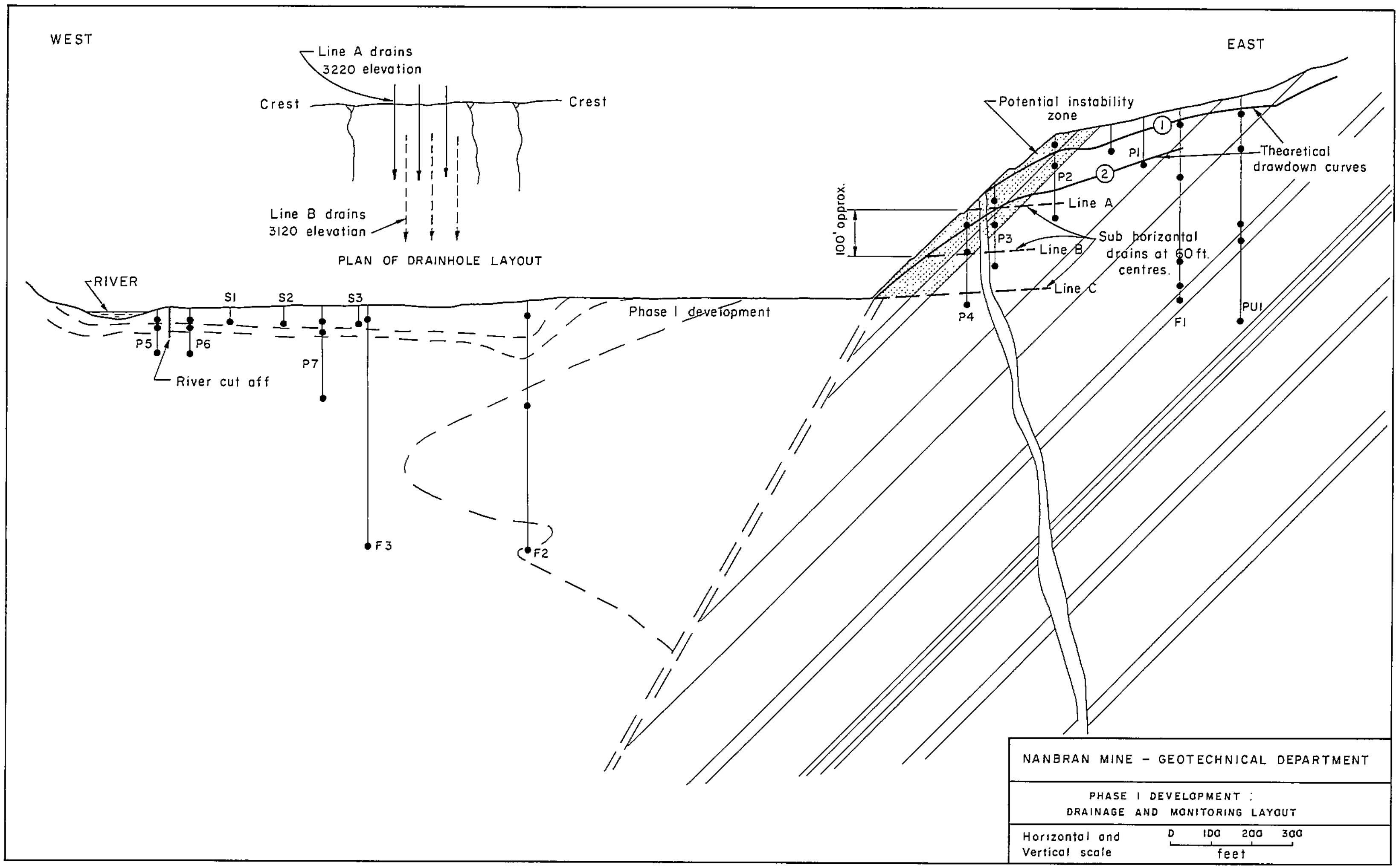


Fig 72 - Nanbran Mine phase 1 development: drainage and monitoring layout.

409. A minimum of 15 ft (5 m) at the surface of each drainage hole should be lined with pipe, extending from the mouth - at which the joint should be water-tight - to the bench drain.

410. To prevent the ends of the drainholes from freezing and preventing adequate drainage, it is recommended that the exposed end of each drainage pipe be insulated with at least 3 ft (1 m) minimum cover of fine rock extending to the edge of the bench drain.

411. The vertical interval for the drainage holes is based on a preliminary bench design. Modification to this design resulting from haul road locations etc may lead to changes in layout of the drainage system.

412. Bench design should incorporate surface drains along the full length of each bench below each row of holes and benches should be excavated so as to ensure a slight and constant gradient to direct the water southwards. Preliminary design studies have indicated that the south end of the pit is the best in which to effect drainage. These have also indicated that drainage can be discharged via surface drains from lines A and B, and possibly line C, whereas below line C collector sumps and pumping installations would be required.

413. To be effective, the drains must be installed from the working level concurrently with the excavation of each bench. The time response of the drain system will be evaluated following preliminary monitoring and will then be checked against the proposed rate of excavation.

SEEPAGE CONTROL - WEST SLOPE

414. Three methods of reducing potential seepage through the surficial deposits in the west side of the pit were evaluated:

a. River Diversion: The prospect of relocating the river substantially westwards was investigated. Topography is generally unfavourable and would require deep cuttings. Lining of some cuts would also be required to prevent erosion. The estimated cost was $4.6 million.

b. Lining the River Channel: An impermeable lining installed in the river bed would be effective in reducing seepage into the superficial cover. However, technical problems associated with the width of the river - 150 ft (45 m) or more in places - would require further development of existing technology. A temporary diversion channel would also be required. The preliminary cost estimate was $4.3 million.

c. Subsurface Cut-off: The provision of a physical barrier to seepage installed east of the river and parallel to it along the length of the orebody offered the opportunity to reduce seepage substantially without affecting the river itself. Two possibilities were envisaged:

   i a well curtain consisting of a line of wells parallel to the river course containing submersible pumps working continuously to lower the water level and to cut off lateral movement of water;

   ii a grout curtain consisting of a line of closely spaced small diameter boreholes parallel to the river, injected with a suitable grout under pressure.

The recommended control method was the grout curtain, rather than the well curtain which was rejected because of safety considerations. In the event of a power failure, the well curtain would cease to function, possibly resulting in flooding or slope instability whereas a grout curtain subject to monitoring and, if necessary, to maintenance by periodic installation of additional grout holes, would be a fail safe system.

415. Under homogeneous conditions it is possible to design a grout curtain using flow net construction or other analogue to predict seepage conditions based on measured piezometric elevations and permeabilities, together with grout-take tests. However, in this case, exploration and numerical analysis had indicated significant permeability differences because of the varying nature of the surficial deposits. It was therefore apparent that the extent of sealing needed, ie, the number and spacing of grout holes, would vary along the length of the curtain.

416. In view of the anticipated variation in the extent of sealing necessary, it would obviously be unsound economically to design to a

standard adequate for the worst case and apply that design along the whole length. Therefore, a "design as you build" approach was recommended, using the principle of stage grouting, and embodying the following design parameters.

a. Grouting must be done in summer when groundwater levels are lowest.
b. The grout holes should extend through the surficial deposits and the weathered porphyry, both of which have relatively high permeability. Rather than specifying a standard depth of hole, therefore, the criterion should be that each hole must extend at least 5 ft (1.5 m) into sound or unweathered porphyry. A geologist should be made available to monitor the drilling and to supervise the contractor in this respect.
c. An initial spacing of 12 ft (4 m) is recommended. Alternate hole positions should be completed first over a partial length of the curtain followed by completion of intermediate holes after the grout in the first holes has set. The grout take in the intermediate holes will provide a guide to further requirements in that particular section. If the take is very low, second stage holes are probably unnecessary. If the take is high, further holes spaced more closely should be drilled, until grout take is reduced to a low value. In this way, the variations necessary in the spacing of grout holes can be achieved at lowest possible expense, ie, during the time that all the necessary plant is on site. The process is repeated if necessary with third or fourth stage holes.

   For guidance in measuring the effectiveness of grouting, a limiting figure of 10% is suggested, calculated as follows:

   Grout take in first holes = A lb/ft (Kg/m) of hole.

   Grout take in second holes = B lb/ft (Kg/m) of hole.

   If (A-B) is greater than 0.1A, proceed with further holes.
d. Long-term monitoring by piezometers and seepage observations/measurements must be carried out as discussed below.

The parameters to be used for grout curtain design are summarized in Table 5 and the suggested grout hole layout is illustrated in Fig 73.

417. The budget cost calculated for the grouting installation is $860,000, assuming secondary grouting over 80% and tertiary grouting over 40% of the length of the orebody.

418. The crucial stage in the grout curtain performance will be between Phase I and Phase II pit development (Fig 66) at the point when the pit floor coincides with the base of the surficial deposits. Provided that the design requirement of a three-quarters drop in piezometric level from west to east across the curtain is maintained during this stage, the curtain will be adequate as the pit is deepened.

419. Because of the varied nature of the surficial deposits, however, high seasonal seepage rates may result in fine material washing out from the overburden in places, and a graded filter should be installed at such locations to prevent internal erosion leading to instability.

Control of Surface Water

420. The entire perimeter of the pit should be surrounded by a surface water interceptor drain to prevent rain and melting snow from entering the excavation. Two essential requirements should be met:

a. The dimensions of the drain must be adequate to cope with the run-off volume generated under conditions of peak rainfall or thaw. This is to avoid the possibility of the drain overflowing and allowing water to enter the pit and cause surface recharge of the pit slopes at a time when groundwater pressures would already be high.
b. The drain should be lined to prevent seepage into the slopes below and to prevent erosion and scour. A cost/benefit study by the mine engineering department on different lining materials - concrete, asphalt, and butyl rubber - is required.

Table 5: Nanbran mine: Grout curtain design and monitoring

Feature	Primary design	Monitoring method	Subsequent design
Hole depth	To 5 ft (1.5 m) below top of sound porphyry.	Supervision by geologist.	-
Hole spacing	First stage holes: single row at 12 ft (4 m) centres, each section grouted in two consecutive stages of alternate holes.	Measure grout take in first stage holes (=A), and grout take in second stage hole (=B). If A-B $\not<$ 0.1A-------------	Second stage holes at half spacing of primaries, in line.
		If (A-B) < 0.1A, no further holes necessary	
		monitor as above, and if (A-B) $\not<$ 0.1A-----------	Third stage holes at half spacing (6 ft (2 m) centres) inset 1 ft (30 cm).
Curtain performance (minimum overall standard).	Curtain should effect 75% reduction in piezometric pressure across its width.	Piezometer installations at 1000 ft (300 m) intervals along length of curtain, consisting of two tips on each side of curtain, 10 ft (3 m) apart, 10 ft (3 m) from curtain and 10 ft (3 m) above base of curtain. If piezometer elevation, above base of curtain, is more than 25% of elevation on west side----	Additional holes at half previous spacing, as above
Curtain performance (localized behaviour).	High volumes of localized seepage may result from local geological conditions or curtain damage.	Selective seepage volume measurements at pit edge by V notch Possible use of tracers to identify flow paths.	Intensive additional grouting in affected area of curtain. Possibly augmented by wells.

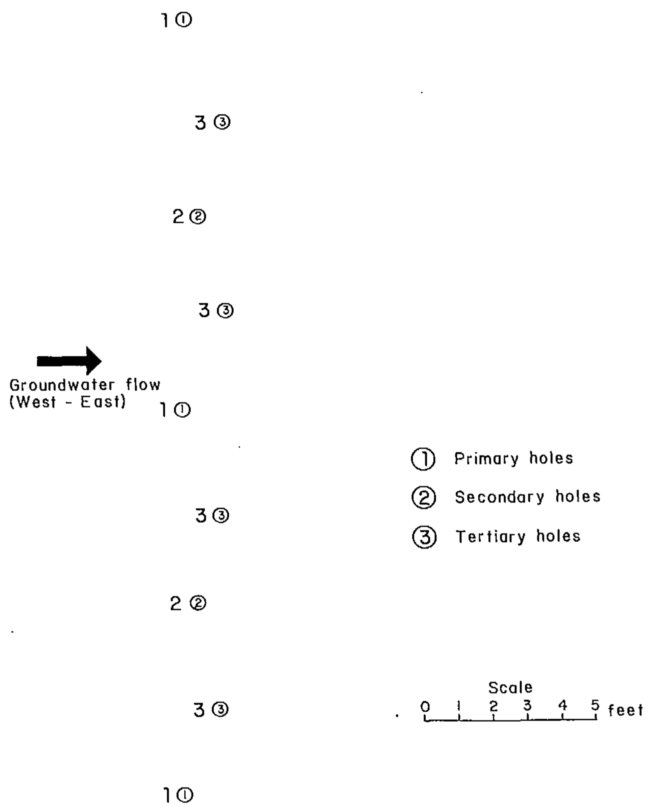


Fig 73 - Nanbran Mine: grout curtain hole layout.

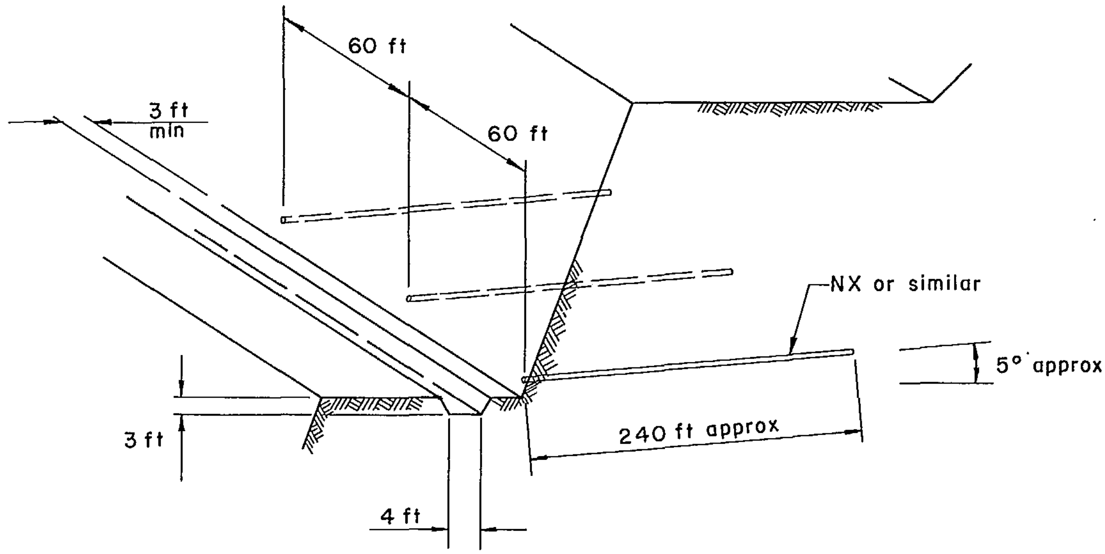
421. Because the main catchment area lies in the high ground to the east, there is little tendency for water to enter the site from the north or west. Having defined the catchment as 3.89 sq miles (9.95 sq km), the peak run-off was calculated as 440 cusecs# (12.5 cu m/sec). It is therefore recommended that the perimeter drain on the east side should be 12 ft (3.5 m) wide and 6 ft (1.75 m) deep. Because less severe inflow conditions exist to the north and west, dimensions of 8 ft (2.5 m) by 4 ft (1.25 m) deep are proposed.

422. No surface run-off interceptor drain is required at the south end of the property, and the other drains should run southwards and discharge to the river system. Schematic lining specification for the permanent bench drains is illustrated in Fig 74. Final design will depend on the outcome of the evaluation proposed in para 420(b).

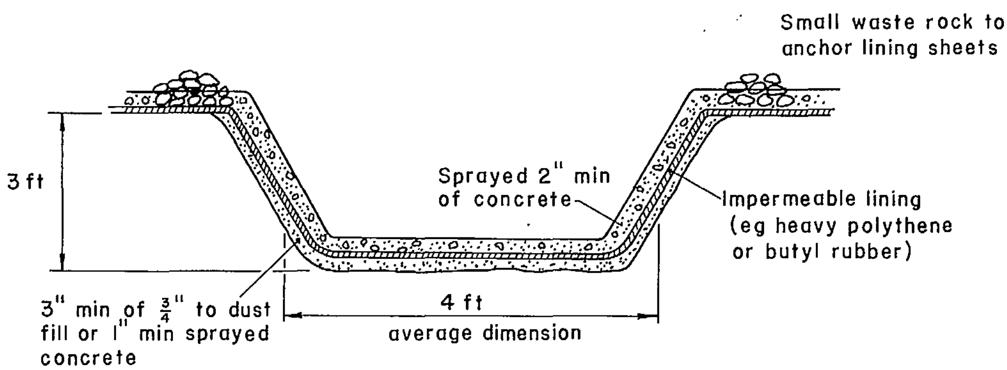
423. The drains proposed for the top of individual benches serve two purposes:

a. to concentrate surface water from the top and face of the bench above;
b. to collect and dispose of the water from the drainage boreholes.

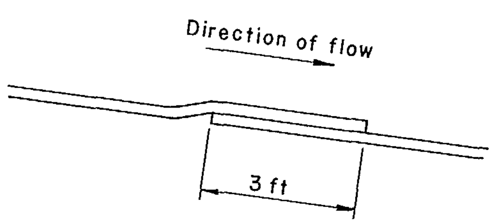
424. Initial flows from the boreholes, ie immediately following drilling of the holes and before much displacement of the phreatic surface, were estimated using flow-net sketching, and a value of about 3 gpm per ft (0.75 l/sec/m) of slope face was derived, indicating approximately 40 cusecs (1.1 cu m/sec) for the full length of the pit. To this, a further 36 cusecs (1 cu m/sec) of possible surface water under peak conditions should be added. Nominal drain dimensions of 4 ft by 3 ft deep (1.25 x 1 m) are therefore recommended.



(a) Spacing and dimensions of drain holes and bench drain (Not to scale)



(b) Details of bench drain construction



(c) Direction and minimum amount of overlap

Fig 74 - Nanbran Mine: detail of bench drain construction.

GROUNDWATER REPORT: OPERATING STAGE

425. Four groundwater reports covering the four development phases of the mine illustrated in Fig 66 are given. Groundwater aspects are summarized at the completion of each phase.

426. In the early phase, the design recommendations are implemented and are subsequently refined in the light of practical experience.

427. Considerable emphasis is placed on monitoring because it is essential in checking validity of the design and in re-evaluating the initial development phases with respect to the final pit slopes. The value of the initial development as a pilot model for later pit phases is emphasized. Generally, groundwater evaluation would be a more or less continuous process during the mine development and results of testing or monitoring may well be used to make intermediate changes to the slope design or the proposed drainage measures.

428. The implementation and effect of the various drainage measures are described and reviewed in relation to the original design objectives.

GROUNDWATER REPORT - COMPLETION OF PHASE I

429. During Phase 1, significant slopes were developed on the east wall of the mine. The intermediate slopes were carefully monitored as a trial for later phases.

430. The following objectives were achieved during this phase:

a. monitoring of groundwater conditions as part of a detailed stability evaluation of east wall slopes
b. installation of trial drainage measures on east wall slopes
c. monitoring of groundwater conditions to the west of the open pit to check effectiveness of the grout curtain
d. long term groundwater monitoring around open pit margins.

Monitoring

431. A typical section, indicating the layout of monitoring piezometers, is shown in Fig 72. Various instruments were used; those selected, their function in the monitoring system and the reasons for their choice are summarized in Table 6.

East Wall Stability

432. Throughout the development towards Phase 1 pit geometry, inclined drainage holes were drilled and maintained in accordance with the preliminary

Table 6: Nanbran mine: Summary of monitoring piezometers

Position no.	Function	Type	Read-out	Reason for choice of type
(Figure 72)				
P1-P4	Point measurement of groundwater pressures in sandstones I and II, shale I, ie, to define conditions in the active zone of Phase I pit development.	Pneumatic (Class C2)	1	Moderate sensitivity and economy.
PU1	Long-term monitoring of boundary conditions (located outside ultimate excavation area).	Pneumatic (Class C1)	1	Long-term stability, low volume demand, therefore accurate, fast response to small changes.
S1-S4	To monitor seasonal variation in groundwater level and effectiveness of grout curtain.	Standpipe (drive-in)	2	No boring required. Inexpensive.
P5, P6	To determine effectiveness of grout curtain	Pneumatic (Class C1)	1	Rapid response. No freezing problems. Remote read-out available.
P7	Initial and medium-term monitoring of drawdown conditions in the porphyry.	Pneumatic (Class C2)	1	Moderate sensitivity and economy.

Read-out Types: 1. Multi-transducer pneumatic read-out unit.
2. Electric dip meter and audiosignal.

design. The drainage design was originally specified before the bench layout was completed. The bench layout eventually adopted was a 110 ft (33 m) vertical bench interval, mined out as two benches of 55 ft (16.5 m) with a 12 ft (3.5 m) wide catch berm between. A 30 ft (9 m) wide berm would be left between the double benches. The bench drains were installed on the 12 ft (3.5 m) berm, before excavation of the lower levels.

433. Measured groundwater pressures at peak recharge were never more than 85% of critical values, and at other times were generally less than 75%.

434. Observations were made of flow rates along bench drains, and these were correlated with precipitation data and piezometric observations.

An average response time of approximately 16 hours to increased recharge was indicated in the east wall.

West Wall Stability

435. Excavating the west wall did not start during this phase, although installation of the sub-surface cut-off was completed, embodying the following aspects:

a. Grouting was carried out during the summer months.
b. Supervision was exercised to ensure that each hole entered unweathered porphyry to a depth of at least 5 ft (1.5 m).
c. A quick-setting grout was introduced through the drilling rods, the rods being withdrawn as grouting proceeded.
d. Because of the fairly low strength of the deposits, care was taken not to cause uplift and cavitation: maximum grouting pressures were kept to 1 psi per ft (25 kPa/m) of depth. The grouting pressure was reduced progressively as the drill stem was withdrawn.
e. The principle of stage grouting was applied, the number of stages during construction conforming to the critera set out in Table 5 (hole spacing). Further control was derived from piezometric observations, as set out in Table 5 (curtain performance).

436. Efficiency of the cut-off was determined from the following expression:

$$E = 100 \frac{H_1 - H_2}{H_1} \%$$

where E is the efficiency of hydraulic isolation
H_1 is phreatic elevation above base of cut-off, on the river side of the cut-off
H_2 is phreatic elevation above base of cut-off on the pit side of the cut-off.

The design requirement (discussed above) is that E must be greater than 75% for adequate performance.

437. A typical series of results, obtained from observations on piezometer installations P5 and P6 (Fig 72) is reproduced in Table 7. The figures represent a fairly extreme case. Fourth stage grouting was necessary in a few isolated areas; in general, adequate hydraulic isolation was achieved with two- or three-stage grouting. All together 1090 grout holes were required, the stage requirements being as follows:

Second stage holes: 60% of length of west wall
Third stage holes: 34% of length of west wall
Fourth stage holes: 11% of length of west wall.

The total cost was $810,000.

Table 7: Nanbran mine: Grout curtain development

Grouting stage	Elevation of phreatic surface ft (m) (P5)	(P6)	Average elevation of base of grout curtain ft (m)	E(%)	Remarks
First	2990.0 (911.3)	2960.5 (902.4)	2924.4 (891.4)	45	Second stage grouting required.
Second	2990.0 (911.3)	2942.6 (896.9)	2927.6 (892.3)	73	Third stage grouting required.
Third	2990.3 (911.4)	2941.9 (896.7)	2927.0 (892.1)	76	Adequate hydraulic isolation.

Conclusions

438. The following groundwater information was obtained during Phase 1:

a. The initial drawdown was as predicted by the design analyses.
b. The preliminary drainage design for the east wall is adequate to maintain the required degree of stability at all times.
c. The grout curtain as installed appears to be an effective method of groundwater control and should lead to favourable long term stability conditions on the west wall.

GROUNDWATER REPORT: COMPLETION OF PHASE 2

Monitoring

439. Most of the piezometers provided for the Phase 1 development were used during Phase 2; the overall monitoring layout is shown in Fig 75. Additional instruments were installed extending downwards from the Phase 1 benches to monitor conditions in the Phase 2 active zone of the east wall (eg, P8 in Fig 75) and to provide additional information about groundwater behaviour in the west slope (eg, P9 in Fig 75). The instruments were similar to those previously installed in these two locations.

East Wall Stability

440. Groundwater pressures were generally acceptable throughout Phase 2 development, and no potential instability was predicted from routine analyses. Although it had been decided previously not to install drainage measures generally in the porphyry, drains on the same pattern as in Phase 1 were drilled through the porphyry to the sedimentary rock to provide a safeguard against deep-seated instability developing in the east wall. The drainage configuration was identical to that of Phase 1 and is shown in Fig 72.

West Wall Overburden Slopes

441. Cut slopes in the surficial deposits were excavated at angles of between 30° and 38°. No evidence of piping or instability was apparent; nevertheless, because of the shortened drainage path between the cut and the river which would result from Phase 3 development, analyses of the sensitivity of stability to groundwater pressures were carried out. As a result, it was decided that wherever slopes in surficial material exceed 60 ft (18 m) in height, the maximum slope angle should be 30°. Some reprofiling was carried out to achieve this.

Porphyry Walls Stability

442. Excavated slopes in the porphyry were developed to overall angles of between 45° and 62° with an average of 55°. Piezometers of the same type as used previously were used to measure groundwater pressures for stability analyses. These indicated probable sliding under extreme conditions. It is recommended that future development in porphyry slopes should be to an overall angle of 45°, if this can be done without increasing the ore/waste ratio: if an increase would result, an investigation should be made to determine the optimum economic balance between increased waste stripping and drainage to maintain adequate stability.

443. In practice, the monitoring reports would probably include sections dealing with the groundwater conditions and stability of the north and south walls. For brevity, these have been omitted.

444. The performance of the grout curtain improved as the pit was deepened. A typical result was as follows:

P5 Elevation ft (m)	P6 Elevation ft (m)	Elevation at base of curtain, ft (m)	E%
2991.3 (911.7)	2939.2 (895.9)	2927.0 (892.1)	81

Because the vertical permeability of the porphyry is likely to affect drawdown to an increasing degree in future development, it is proposed that the frequency of reading piezometers P5, P6, S1, and S2 be reduced from twice weekly to monthly.

Conclusions

445. The following information was obtained

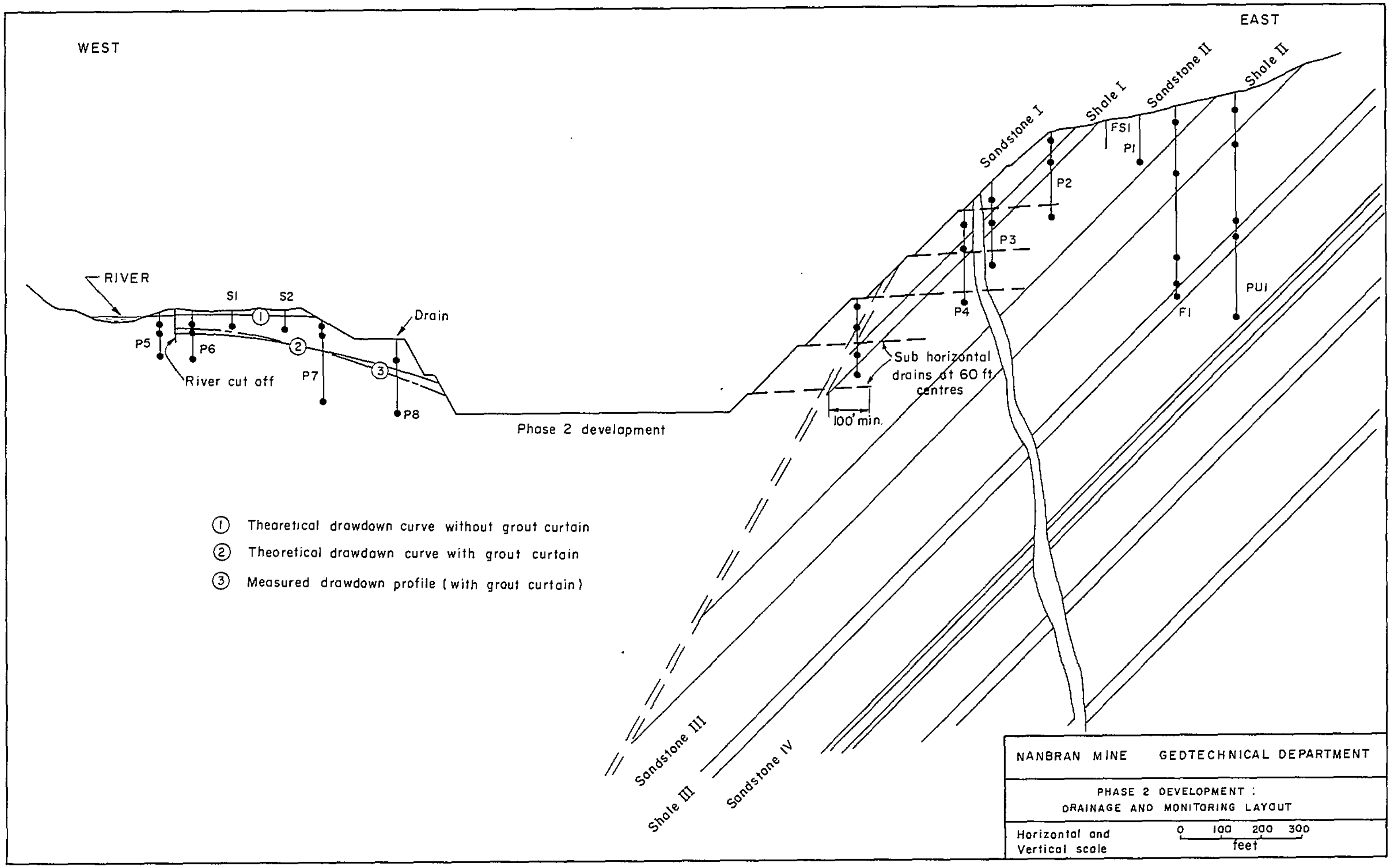


Fig 75 - Nanbran Mine phase 2 development: drainage and monitoring layout.

during Phase 2:

a. The drainage design for the east wall remains adequate at the increased depth to maintain the required degree of stability.
b. Future slopes in porphyry should be excavated to overall slope angles of not greater than 45°, assuming that current drawdown estimates are maintained; if necessary, the cost-effectiveness of drainage should be determined.
c. Adequate hydraulic isolation, indicated by an efficiency in excess of 75%, is being provided by the grouted cut-off. Frequency of monitoring should be reduced.
d. Cuts in overburden should not exceed an overall angle of 30° if the slope height is greater than 60 ft (18 m).

GROUNDWATER REPORT: COMPLETION OF PHASE 3

Monitoring

446. Completion of the Phase 3 development and the associated monitoring/drainage layouts are shown in Fig 76. Only piezometers numbers PU1, P5 and P6 remained from the original instrumentation; these were augmented by additional instruments (P10, P11 and P12 in Fig 76).

East Wall Stability

447. It was recognized that design decisions for the ultimate east wall profile should be made early in the Phase 3 development, so that the mining plan could be finalised. Information to assist in the design was available from two sources:

a. experience with Phases 1 and 2, which had indicated that the drainage design had proved adequate up to the end of Phase 2.
b. an assessment of the drainage behaviour of the lower regions of Phase 3 slopes, based on a thoroughly instrumented trial slope.

448. It was apparent during the first thaw after excavating the Phase 3 trial slope that adequate drainage was not being achieved using the Phase 2 drainage design. In a section of the east wall selected for tests, the effects of altering the drainage pattern in the following ways were evaluated:

a. reducing vertical spacing of lines of drainage holes
b. increasing depth of drainage holes
c. decreasing spacing of drainage holes.

Groundwater pressures were monitored using piezometers, additional installations being made on the test section.

449. A typical series of results is reproduced in Fig 77, from which it may be seen that both increasing the hole depth and decreasing the spacing improved stability. Decreasing the vertical interval was less effective; the main effect of introducing a line of intermediate holes was to decrease discharge from those in the row below without substantially altering water pressure distribution.

450. Following the tests, it was decided to mine the remainder of Phase 3 to the same overall slope angles and disposition of benches as in Phase 2, but to decrease the lateral spacing of drainage holes to increase drainage capacity. The alternative, which was to mine to more conservative slope angles, would have involved considerably greater expenditure which it was felt was not justified in view of the uncertainty of such adverse conditions developing.

451. The drainage design for Phase 3 slopes, therefore, evolved from three distinct considerations:

a. Using the groundwater pressure distributions previously obtained from monitoring, the stability of the top 300 ft (90 m) of the east wall was reassessed taking into account Phase 3 slope geometry, from which it was apparent that drainage would not be necessary in this region. Therefore no drainage measures were applied to the top three benches.
b. Similar analyses east of the microdiorite dyke, showed that the Phase 2 drainage design would provide adequate stability in those slopes. NX holes at 60 ft (18 m) centres and vertical intervals of 110 ft (33 m) extending a minimum of 100 ft (30 m) into sandstone were specified.
c. From the results of the trial slope evaluation, NX holes at 30 ft (9 m) centres and vertical intervals of 110 ft (33 m) were specified for the three lowest bench levels, extending a

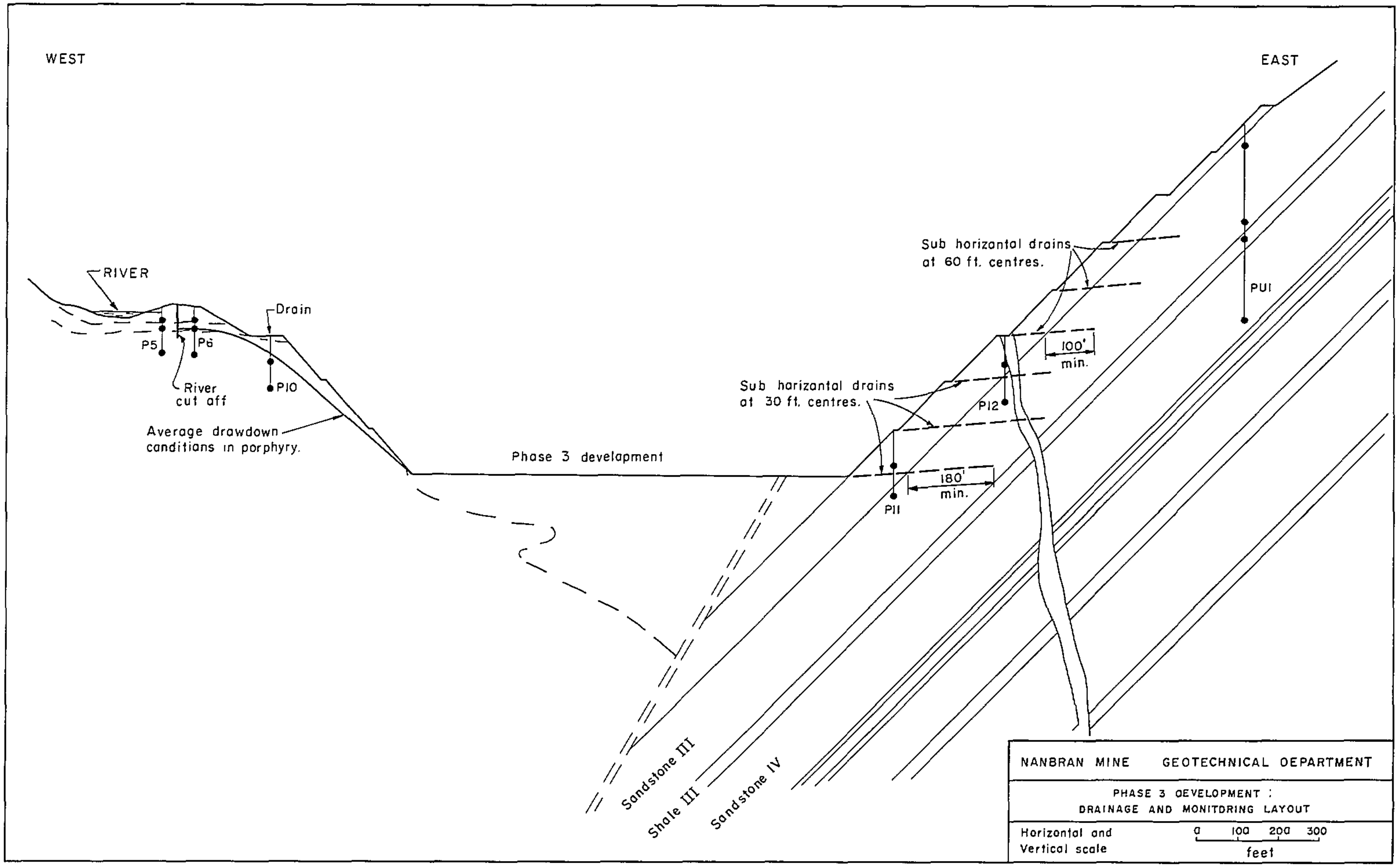
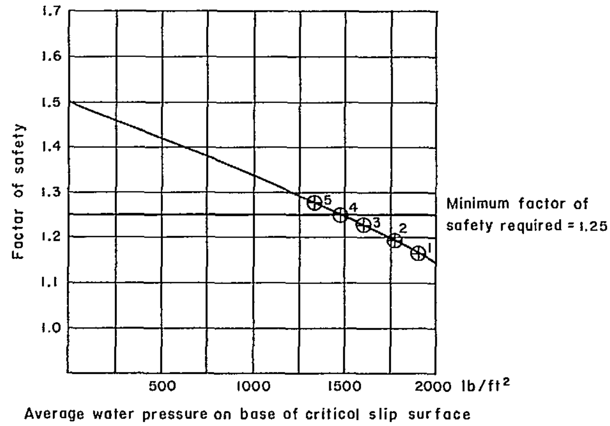


Fig 76 - Nanbran Mine phase 3 development: drainage and monitoring layout.



	Hole depth (feet)	Hole spacing (feet)	Vertical interval (feet)
1	100	60	110
2	100	60	55
3	100	30	110
4	180	60	110
5	180	30	110

Fig 77 - Nanbran Mine: effect on factor of safety and on water pressure on the potential sliding surface of drainage patterns.

minimum of 180 ft (54 m) into sandstone. The details are shown in Fig 76. Results of groundwater monitoring indicated satisfactory stability conditions at all times.

West Wall Overburden Slopes

452. During Phase 3, the west wall slopes were mined to the final profile. Overburden slopes higher than 60 ft (18 m) were cut at 30° as recommended in the previous phase. The grouted cut-off continued to operate well, ensuring favourable groundwater conditions within the overburden.

Porphyry Wall Stability

453. Monitoring of the grouted cut-off was continued throughout Phase 3, although at a reduced frequency. As the pit was deepened, the phreatic surface on the east side of the cut-off continued to drop.

454. It is proposed, therefore, to discontinue routine monitoring of the cut-off curtain. Visual inspection of the perimeter drain, which is part of the mandatory daily inspection of the pit surface, together with monitoring of groundwater pressures at depth in the porphyry slopes, will be relied on to indicate any defects that may occur in the grout curtain. Such a possibility is considered highly unlikely.

455. In accordance with previous recommendations, final slopes in porphyry were developed to overall angles of 45° and weekly groundwater monitoring indicated that conditions were satisfactory at all times. Because the shape of the orebody will result in flatter slopes at increased depth, it is not anticipated that any instability will result from groundwater pressure; however, monitoring will continue at intervals of six weeks except during the spring, when it will be weekly.

Conclusions

456. The following information was obtained during Phase 3:

a. Based on previous experience and a monitored trial slope, the Phase 3 ultimate east wall drainage design was specified as being similar to the Phase 2 design. The calculated risk of underestimating possible deterioration in stability from increased drainage problems was recognized.

b. Drainage design for the east wall was specified as:
   - i top 300 ft (90 m) - no drainage
   - ii slopes below 300 ft (90 m) elevation and in ground east of dyke - as Phase 2
   - iii lower slopes - NX holes at 30 ft (9 m) centres, 110 ft (33 m) vertical interval, extending a minimum of 180 ft (54 m) into sandstone.

c. No stability problems arose in the west wall and none are anticipated in future development.

d. Monitoring of the grouted cut-off can be discontinued and frequency of monitoring porphyry slopes can be reduced, except during the spring thaw.

GROUNDWATER REPORT: COMPLETION OF PHASE 4

Monitoring

457. The piezometers remaining from the Phase 3 monitoring were augmented by additional instruments in the east wall, eg, P13 and P14 in Fig 78.

East Wall Stability

458. During excavation of the east wall through the fault zone, considerable problems of local stability were encountered and several slides on individual benches took place. Continuous lining of drainholes with perforated pipe was necessary. Many of the drainage boreholes had to be re-drilled because of dislocations caused by movement in the holes before perforated piping could be installed. Initial water yields from drains passing through the shear zone were very high, and then fell off. Substantial recharge from areas outside the pit limits during heavy rain and the thaw caused increased flows.

459. During the early parts of Phase 4 development, drainage was installed according to the design for the lower Phase 3 slopes, ie NX holes at 30 ft (9 m) centres and vertical intervals of 110 ft (33 m) extending a minimum of 180 ft (54 m) into sandstone.

460. Groundwater monitoring indicated that groundwater pressures in the east wall Phase 4 working slopes could exceed the levels for critical stability, and particular attention was paid to installing drainage with minimum delay after excavating each bench. The tests carried out during Phase 3 had indicated the need for longer holes at 30 ft centres and this, combined with the need for immediate drain installation and problems associated with bad ground, increased the cost of drainage measures threefold.

461. During development of the lower Phase 4 working slopes, monitoring indicated the frequent occurrence of excessively high groundwater pressures in Shale II, and attempts were made in an instrumented test section to alleviate these by installing further drain holes at half the previous vertical and horizontal spacing, ie at 55 ft (17 m) vertical interval and 15 ft (5 m) centres. However, the effect of this was negligible. To determine reasons for the poor drainage performance, a series of cored horizontal boreholes was drilled into the east wall for a geotechnical evaluation of Sandstones II and III and Shale II. Laboratory tests were carried out to determine the permeability and shear strengths of specimens of shale at normal stress levels representative of field conditions. The results of the testing programme indicated:

a. lower joint frequency in the sandstones than at higher pit elevations;

b. complete absence of weathering on the joints and bedding planes, resulting in tighter and less permeable discontinuities;

c. permeabilities two orders of magnitude lower in both rock types than found in the initial investigation;

d. some evidence of deformation of the shale at the stress levels prevalent in the base of the pit wall.

462. It was clear that slope stability was being affected by new factors related to the scale of working. The effects of changes in material properties with depth and the greater volume of recharge and higher stress levels at the increased

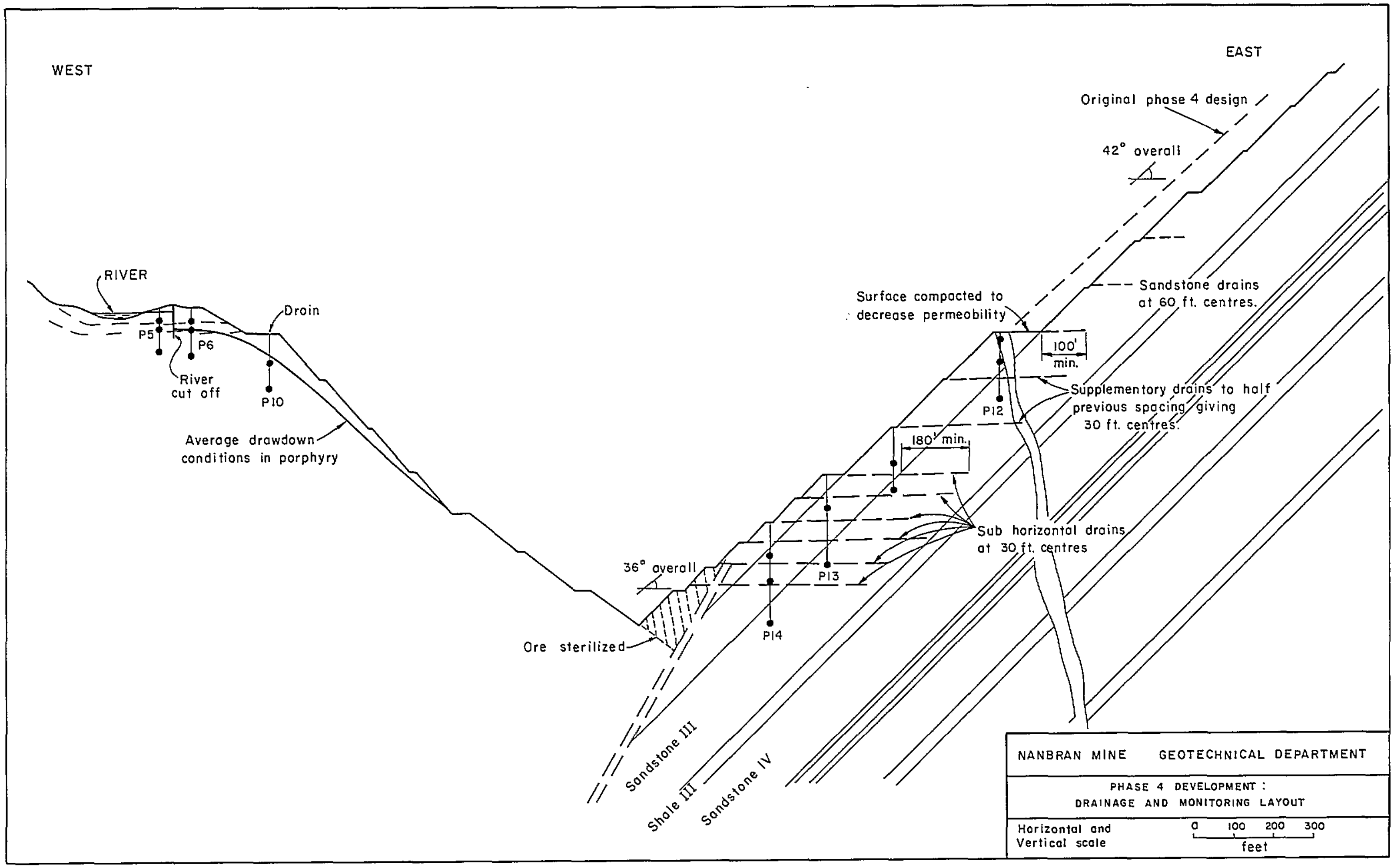


Fig 78 - Nanbran Mine phase 4 development: drainage and monitoring layout.

depth of working were evaluated. It was concluded that artificial drainage was not capable of improving stability sufficiently to allow the ultimate pit slopes in the deepest regions to be developed according to the original design ie, to the same overall slope angle as Phase 3.

463. Because this had, to some extent, been foreseen at the outset of Phase 3, when a calculated risk was taken that the slope design then adopted would be adequate for the whole ultimate slope, the planning department had for some time been working on modified mining plans. These, combined with the groundwater and other geotechnical information collected during earlier development and in the specific investigation, provided a large volume of analytical information. This was used to carry out an evaluation of the cost implications of alternative designs, and results of the cost/benefit analysis were used to derive a modified pit profile for the east wall.

464. The re-design is shown in Fig 78. It embodied the following modifications to the original design:

a. Complete removal of Shale II in the top 500 ft (150 m) of the slope, and installation in the lower half of the cut of three rows of drainage holes at 60 ft (18 m) centres and vertical intervals of 110 ft (33 m), extending a minimum of 100 ft (30 m) into the slope;
b. Treating the top surface of the berm in Shale II by adding a layer of compacted mill tailings, graded towards a bench drain to provide an impermeable top surface and good surface drainage.
c. Limiting excavation of Phase 4 slopes to provide final slopes in sandstone rather than in Shale II as was originally planned, and providing bench drains at 30 ft (9 m) centres and vertical intervals of 55 ft (17 m), extending 180 ft (54 m) into the sandstone. Bench layout was modified to include a 12 ft (4 m) wide berm every 55 ft (17 m).
d. Reducing the overall Phase 4 slope angle in sandstone and porphyry below the shear zone to 36°, compared with the 42° originally planned, and providing bench drainage at 30 ft (18 m) centres and vertical intervals of 55 ft (17 m), extending 180 ft (54 m) into the sandstone.

465. These measures were effective in achieving stability but some ore was lost as indicated in Fig 78. To mine this ore and maintain the required overall slope angles would have resulted in an uneconomic stripping ratio.

West Wall Overburden and Porphyry Slopes

466. No problems were experienced on the west wall either in the overburden or in the porphyry slopes during the Phase 4 excavations. The effectiveness of the cut-off was clearly revealed by the on-going monitoring. Drainage of the west wall was found unnecessary following a stability evaluation early in Phase 4.

OVERALL CONCLUSIONS

467. Throughout the section on Operating Stage, presenting numerous examples has been avoided because groundwater conditions vary considerably from site to site. Furthermore, a particular groundwater pressure distribution is of limited use in itself and becomes of value only when its significance to slope stability can be evaluated. This requires a thorough knowledge of all the geotechnical factors which must be considered in a general stability analysis.

468. The purpose of the groundwater reports has been to indicate in a general way some of the considerations frequently related to the stability of open pit slopes at various phases of development. The pertinent factors can be summarized as follows:

a. The need to understand the influence of groundwater conditions on slope stability.
b. The need to ensure that the layout of monitoring instruments, and types of instrument selected, are continuously reviewed to ensure their relevance to pit development.
c. The need to frequently re-assess the overall situation, so that effort can be devoted to aspects of greatest current significance. Where the importance of a particular aspect recedes with development, as in the case of the grout curtain, the frequency of monitoring should be re-assessed. This avoids a continually expanding workload, which leads in

turn either to unnecessary growth in staff or to ineffective control if the staff is not enlarged.

d. *The importance of considering all related groundwater and operating factors - for example, continuous rainfall recording - must be related to piezometric observations if any basis for predicting critical conditions is to be made. A technically sound solution to a drainage problem is only valid if it is feasible in terms of operating and financial constraints.*

e. *As a pit deepens, drainage requirements may become more extensive and therefore more expensive.*

f. *Results of empirical problem solving using test areas of the pit can often be more effective, both in cost and performance, than theoretically derived solutions. In any case, theoretical solutions need field testing before any large scale commitment to them is undertaken. The importance of monitoring in this respect is to give an accurate measure of their effectiveness.*

g. *Finally, it must be borne in mind that a point in pit development is frequently reached at which slope drainage can no longer provide the solution to stability problems, in terms of either cost or practicability. One of the main benefits of a comprehensive monitoring programme is that it should provide sufficient forewarning to allow proper evaluation of alternative solutions. In this way, the financial impact of changed conditions can be minimized.*

JEFFREY MINE

LOCATION AND HYDROGEOLOGY

469. The Jeffrey mine, owned by Canadian Johns-Manville Co Ltd is situated at Asbestos, Quebec. In 1977 it is currently the largest open pit mine in Canada, more than 900 ft (275 m) deep, and produces asbestos fibre. The main features of the pit are shown in Fig 79.

470. The principal rock types are ultrabasic peridotites and dunites underlain by the Caldwell slate series which form the north wall of the pit. Overburden consists of clay silts, silts, sand and sandy gravels to depths of 300 ft (90 m).

471. Groundwater has been recognized for many years as having a significant influence on slope stability. The overburden, particularly on the east side, appears to act as a major source of recharge into the underlying rock slopes.

Permeability Characteristics of the Rock Units

472. The ultrabasic rocks are generally without open fractures and are relatively impermeable with measured mass permeability values of less than 10^{-6} cm/sec. In the north wall slates, partially open fractures probably exist, giving rise to a moderate quantity of groundwater flow within discrete fissures. The measured mass permeabilities of the slate range from 10^{-4} to 10^{-5} cm/sec and are anisotropic - most of the flow is parallel to bedding.

473. Impermeable shear zones within the ultrabasic series, which are oriented approximately NE - SW, prevent significant flow in directions normal to their strike. Thus, where a shear zone is continuous over an entire face, as, for example, the contact shear zone on the north wall or the shear zone beneath the skipway on the south wall, it will act as a relatively impermeable barrier and tend to dam groundwater behind it.

474. The pit area can be divided broadly into two permeability zones - first, the north wall slates with an estimated average permeability of 5 x 10^{-5} cm/sec, and second, the ore and shear zones with an average permeability of 5 x 10^{-7} cm/sec.

Observed Piezometric Conditions

475. Results of piezometric monitoring in various parts of the pit have indicated a groundwater table some 75 to 125 ft (25-40 m) below surface. Considering the scale of the excavation, this indicates generally adverse groundwater pressures within the slopes.

Influence of Slope Drainage on Stability

476. The influence of groundwater on stability has been determined for the entire pit. The cost

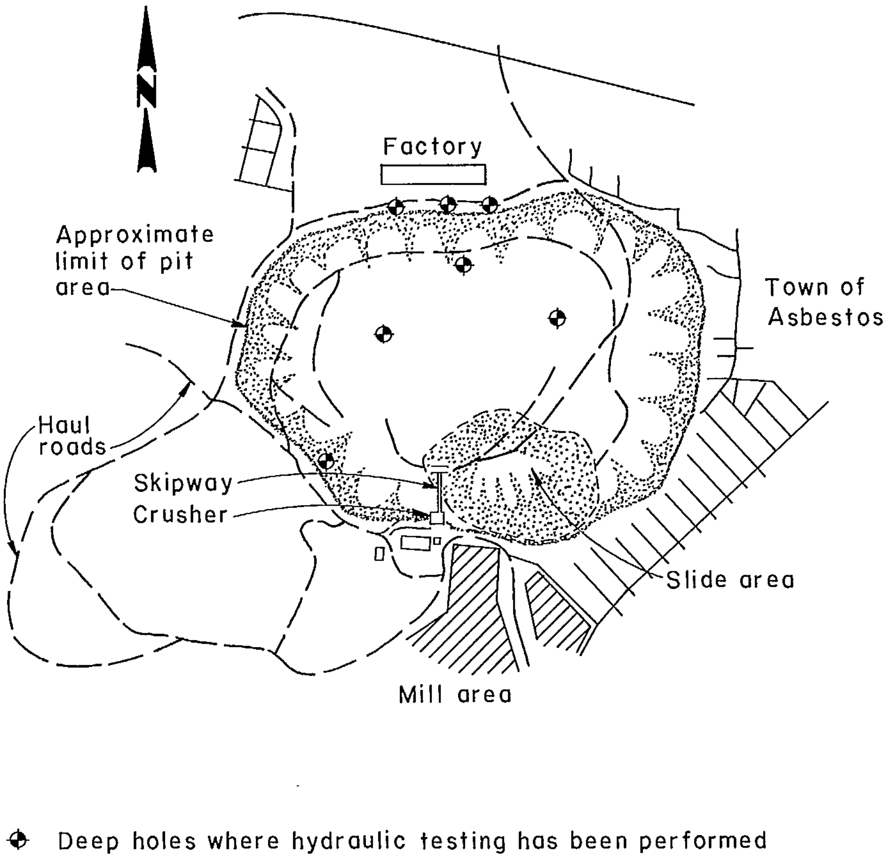


Fig 79 - Plan of Jeffrey Mine, Asbestos, Quebec.

saving in reduced waste through steeper slopes achieved by means of groundwater control would be significant. General hydrogeological considerations suggested that the low permeability and poor drainage characteristics of the ultrabasic material are limiting factors in the application of such stabilizing measures, but that a considerable measure of groundwater control could be achieved within the slate materials.

477. This case study gives brief details of a drainage trial in the north wall slates to ascertain response of the rock to drainage, and the stabilization of a major rock slide in the south-east corner of the pit through drainage.

NORTH WALL SLATES DRAINAGE TRIAL

478. A series of horizontal drains were driven into the north wall slates to determine the effect of horizontal drains on the groundwater conditions behind the north wall face. Groundwater conditions were monitored using four commercially available pneumatic piezometers; details of a typical installation are shown in Fig 80. This program was carried out during October, 1973.

479. On completion of the piezometer installations, monitoring of piezometric pressures was commenced to determine initial groundwater conditions. Readings were taken once or twice a day depending on how rapidly conditions changed.

480. Packer tests were conducted in each of the piezometer holes to determine permeability. These indicated a generally low permeability in the north wall slates of approximately 10^{-6} cm/sec. Figures 81, 82 and 83 show the packer test results for all three piezometer holes. BH-944 has a higher permeability region at roughly between 50 and 100 ft (15-30 m) deep from which the observed artesian water is thought to originate.

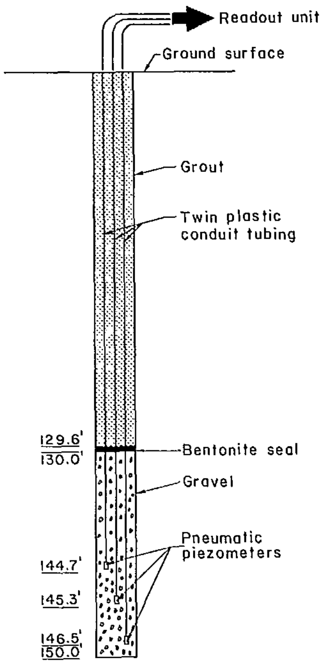


Fig 80 - Pneumatic piezometer installation for borehole 944, Jeffrey Mine.

481. Eight horizontal holes were driven into the north wall using an Atlas Copco air-driven, track-mounted drill with a 3 in. (7.5 cm) diameter tungsten carbide bit. The holes were inclined at 5° above the horizontal. Figure 84 shows the layout of drainage holes and their relationship to the monitoring piezometers.

Effect of Drains on Groundwater Pressure

482. Of the eight holes drilled, only drainage hole 2 significantly reduced the groundwater pressures recorded by any of the piezometer installations (Fig 84). A section through the three installations is given in Fig 85 and shows the interpreted phreatic surface before and after placement of drains.

483. A graph of piezometric responses versus time is shown in Fig 86. The greatest drop in groundwater pressures occurred in BH-945 near the completion of the second horizontal drain. Field observations have noted that the drop occurred

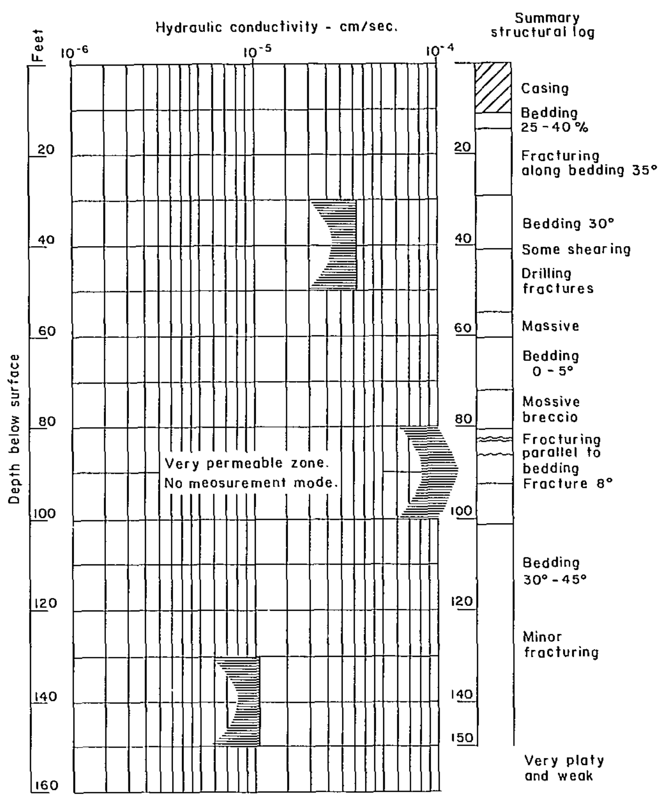


Fig 81 - Hydraulic conductivity log for borehole 944, Jeffrey Mine.

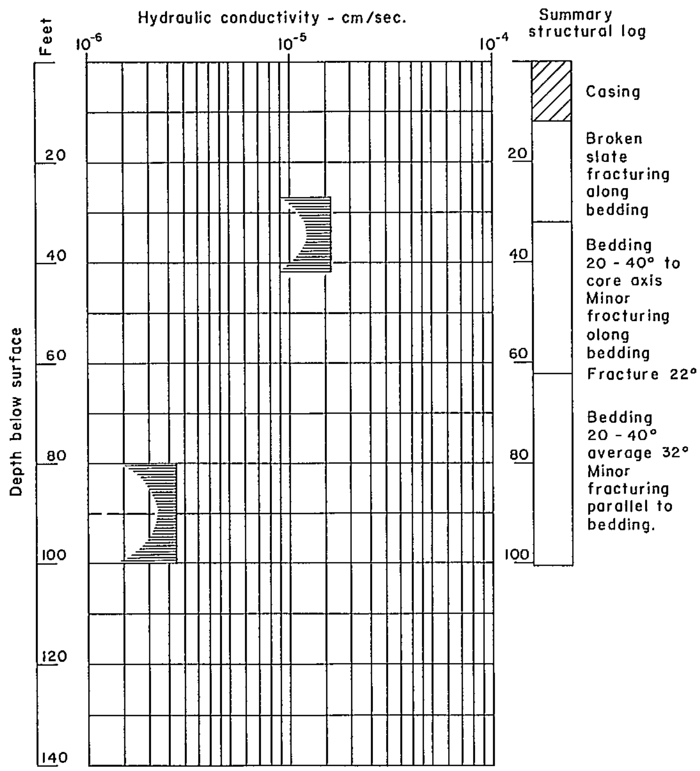


Fig 82 - Hydraulic conductivity log for borehole 945, Jeffrey Mine.

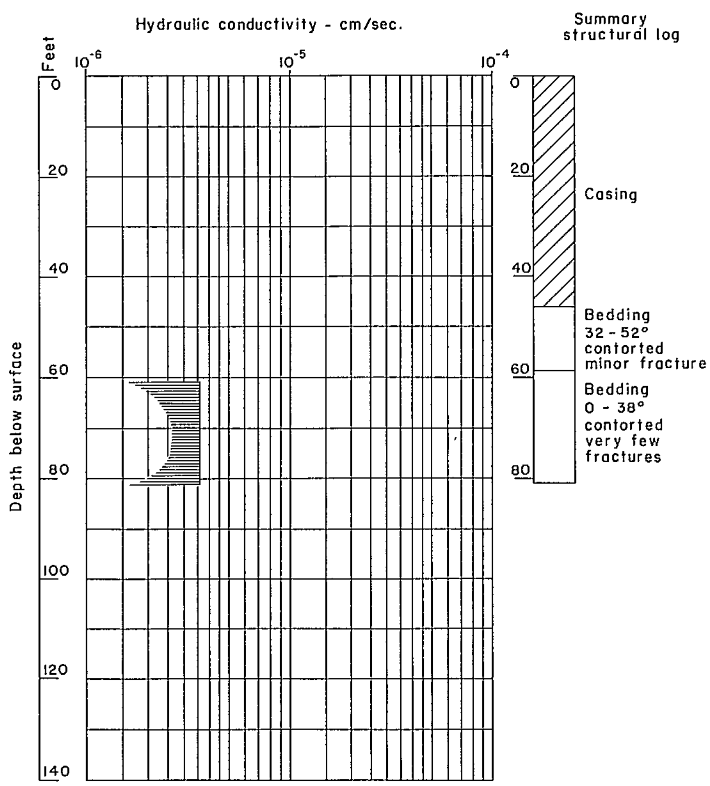


Fig 83 - Hydraulic conductivity log for borehole 946, Jeffrey Mine.

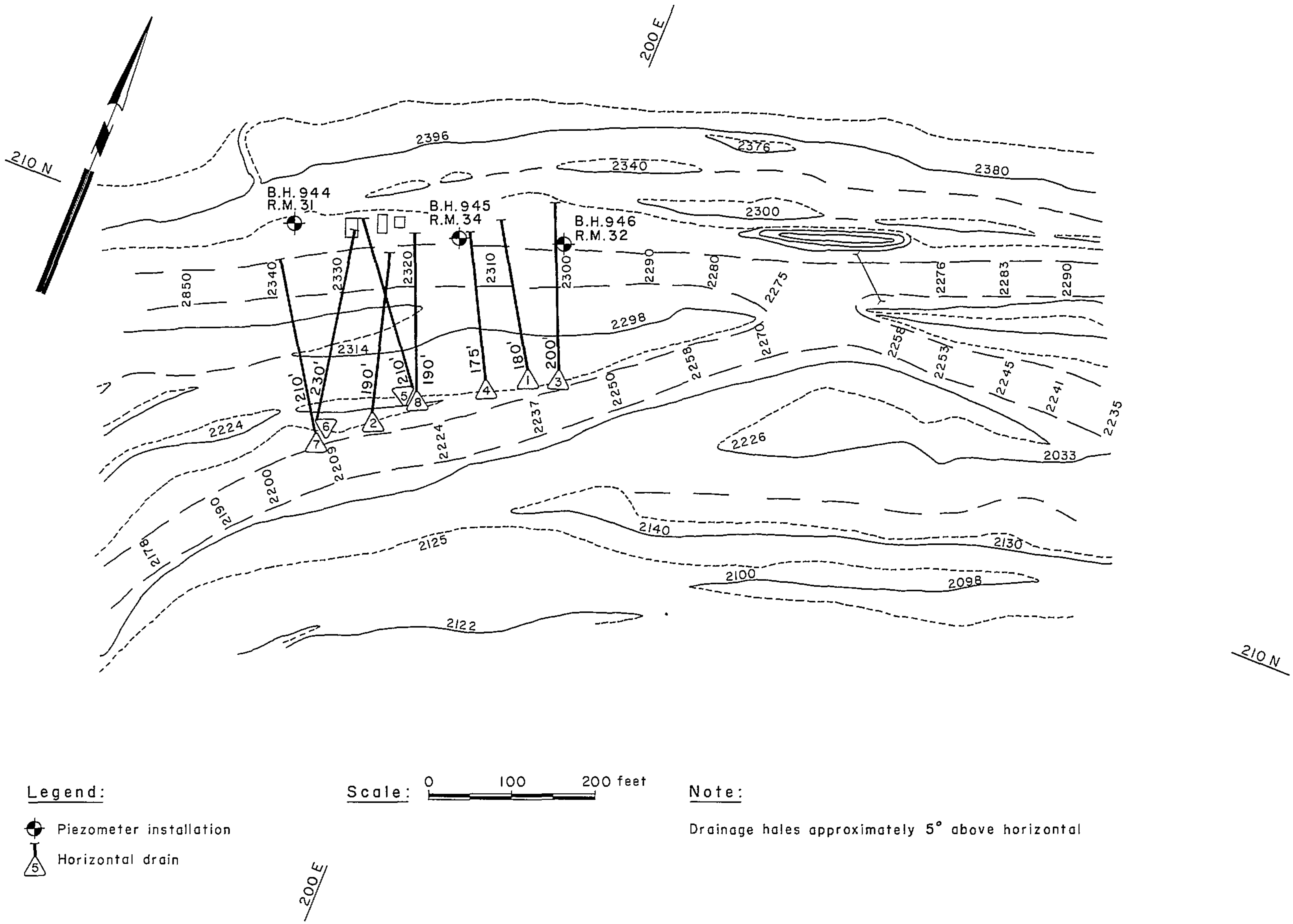


Fig 84 - Drainage test at North Wall, Jeffrey Mine.

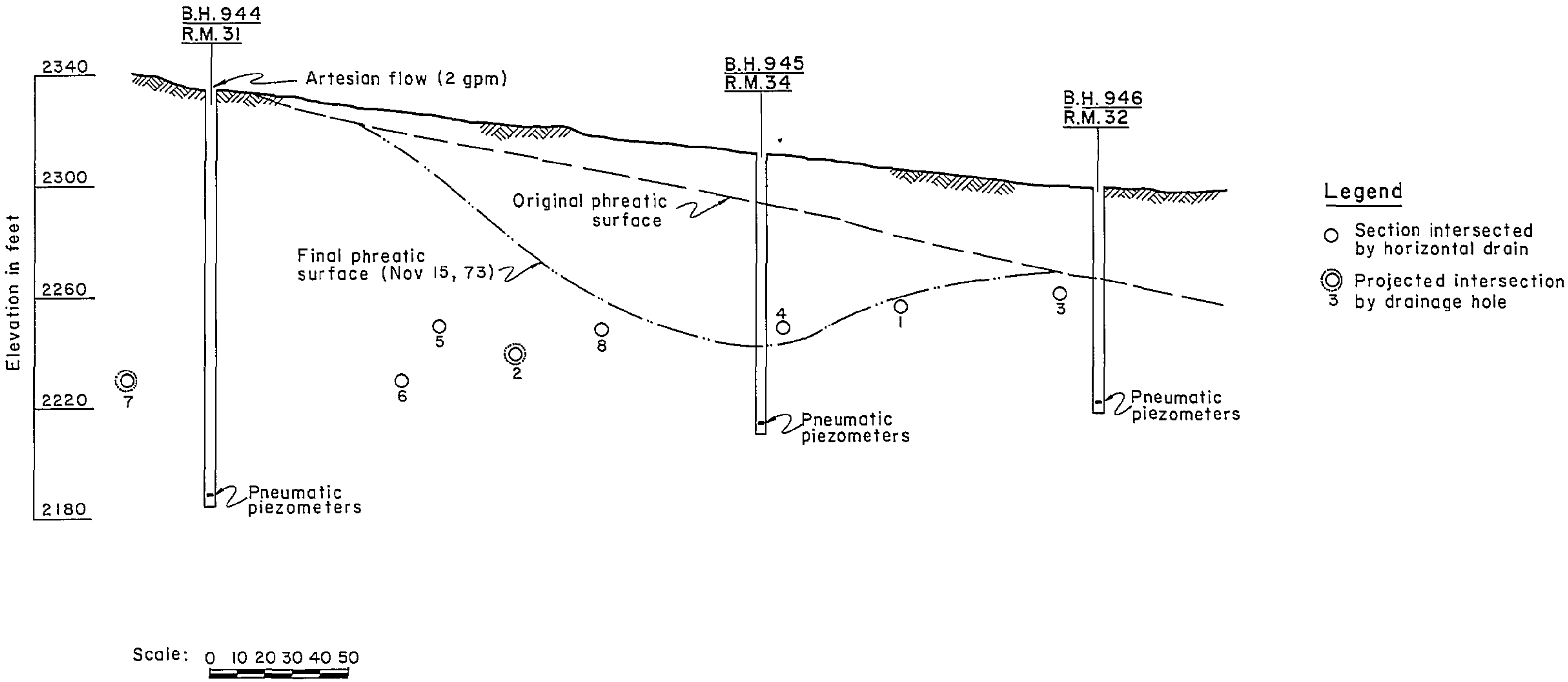


Fig 85 - Section through piezometer installations at the North Wall Drainage Trial, Jeffrey Mine.

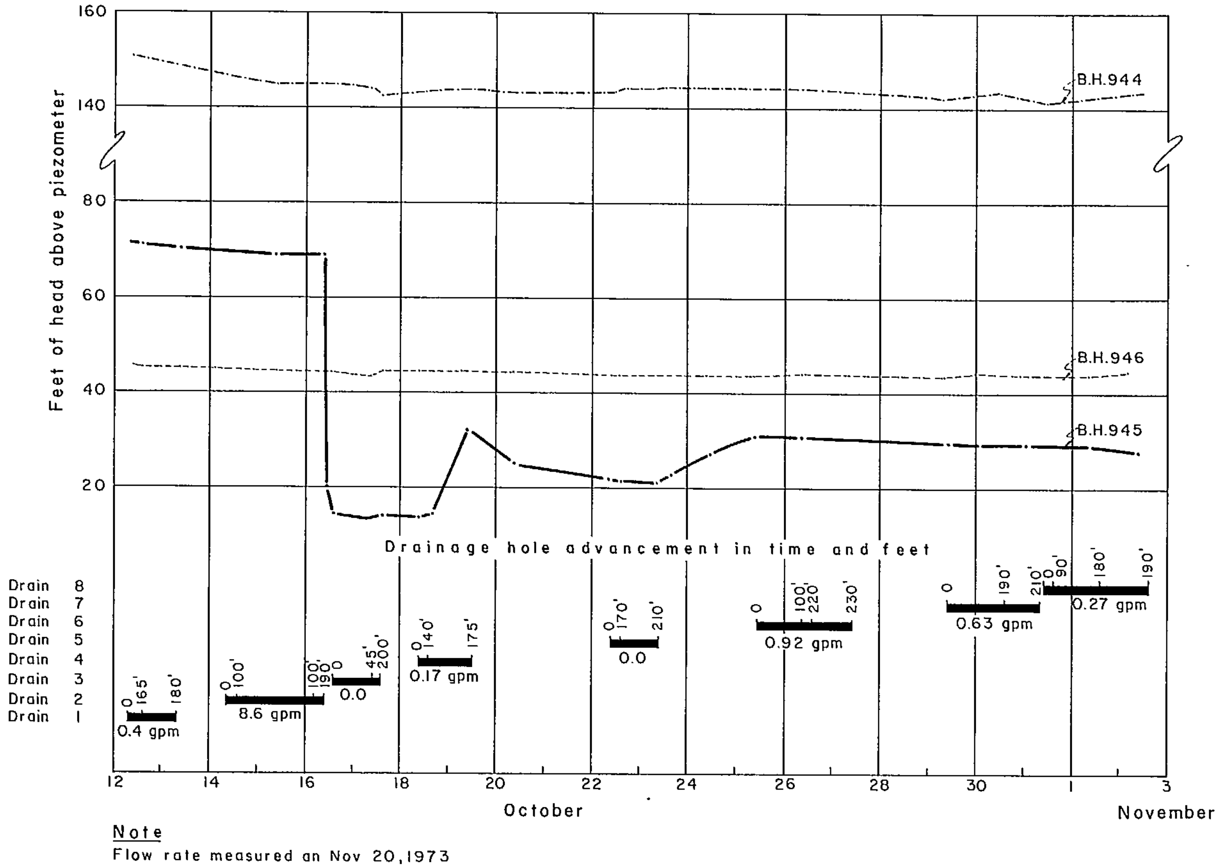


Fig 86 - Graph of piezometric response against time for the North Wall Drainage Trial, Jeffrey Mine.

almost immediately after drain hole 2 started producing water at approximately 15 gpm (70 l/min).

Conclusions and Recommendations

484. The piezometer installations indicated that relatively adverse groundwater conditions existed close to surface. The variation in piezometric response to horizontal drains indicated the complex nature of groundwater flow through the slate material.

485. Drain hole 2 intersected a more permeable zone approximately 185 ft (60 m) into the face, which effectively reduced the groundwater level in BH-945 and reduced the artesian flow from BH-944. Neighbouring drain holes failed to intersect this permeable zone or the permeable zone in BH-944. This suggests that these zones are localized and are not continuous within a given plane.

486. Based on this information, it was recommended that further drain holes into the north wall slates should be spaced approximately 150 ft (45 m) apart, be 200-300 ft (60-90 m) deep and be drilled directly into the face at an angle of 5° above horizontal.

STABILIZATION OF SOUTH-EAST CORNER ROCK SLIDE

487. This section describes the successful application of drainage to stabilize a major rock slope which became unstable close to an important plant installation. The slide was partially caused by adverse groundwater conditions.

General Description

488. The unstable area is shown in Fig 79. The slope consisted of a 600 ft (180 m) high rock section, overlain by some 200 ft (60 m) of overburden. This area of the pit was composed of relatively highly fractured, serpentinized

peridotite with complex major shear zones, generally dipping into the slope. At the time of the initial movements, excavation was proceeding around the toe. The overburden material is variable and contains several permeable sand/gravel lenses, which transmit significant quantities of groundwater. Direction of groundwater flow is towards the slope face.

Summary of Slide Data

489. The following summary is included to indicate the general nature of the investigation and stabilization measures carried out.

a. In the period 1965 - 1970 sloughing, ie, shallow slides, of the central, upper benches of the rock slope in this area took place due to gradual rock deterioration.
b. Before 1970, several relatively shallow slides had occurred wholly within the overburden due to adverse groundwater conditions.
c. During 1970, some apparent extension of the skipway rails was observed (Fig 79). The rails had been laid on a 45° fill slope and localized creeping of this fill material was suspected. Because of the critical role of this installation in mining operations, extensometers were installed from the ore pass bridge level and from the top of the slope to check movements at depth. The instruments became operational at the end of 1970.
d. In December 1970 and January 1971, a major overburden slide occurred to the north of and adjoining this area. Groundwater pressures were again shown to be a contributing factor.
e. In January 1971, a localized wedge slide of two benches in relatively competent rock occurred near the toe of the slope, below the main haul road crossing this area. The slide was sudden and no previous signs of distress had been observed. Figures 87 and 88 show a plan and cross section respectively of the slide area.
f. During February 1971, a massive, overall movement of the entire pit wall was suspected. A program was begun to monitor movement by

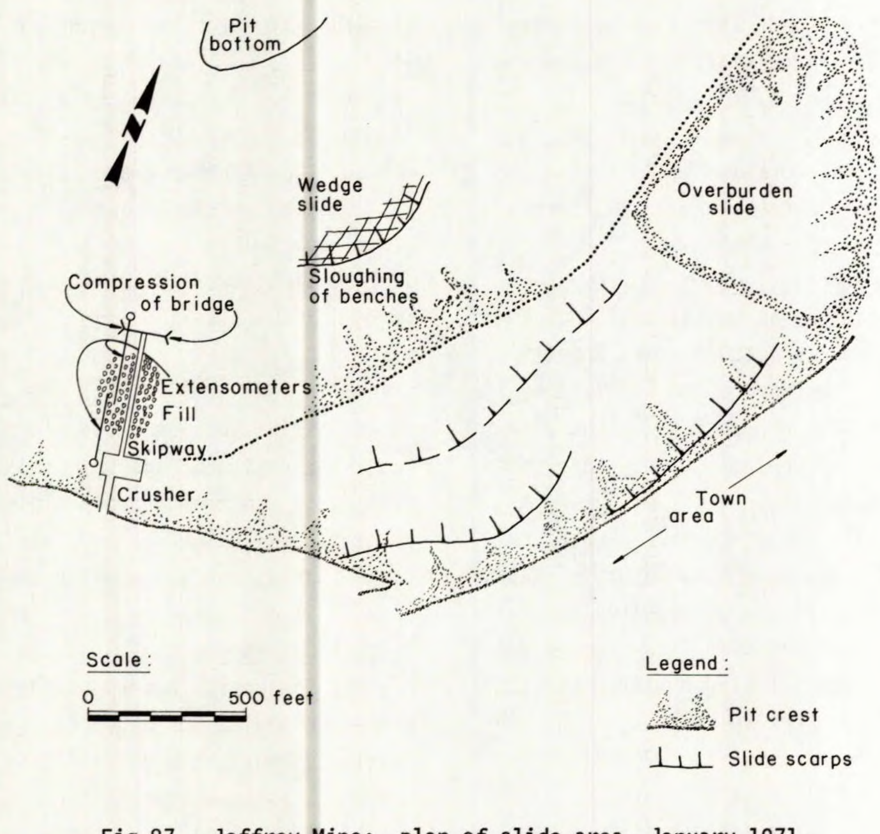


Fig 87 - Jeffrey Mine: plan of slide area, January 1971.

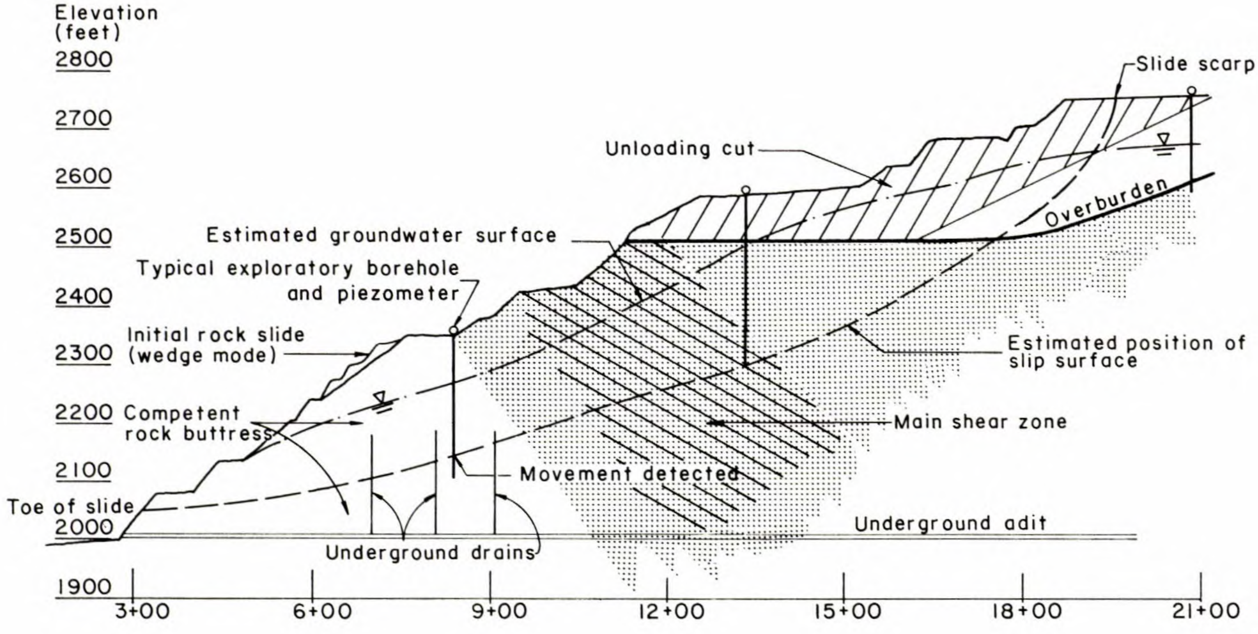


Fig 88 - Jeffrey Mine: section through axis of slide, January 1971.

surface displacement measurements and by borings to determine the depth to which movement was occurring. Piezometric conditions were also determined from these boreholes. The snow cover was up to 5 ft (1.5 m) during this period which made investigation difficult.

g. Measurements on the surface and in boreholes indicated movement of the entire slope, some 700 ft (210 m) in height, at depths to 250 ft (75 m). At this stage, further surface measurements were begun to define the extent of the slide more precisely.
h. A preliminary stability assessment using back analysis indicated that drainage of the slope area and stripping of overburden (ie, unloading) on the upper levels would improve stability.
i. During April/May, 1971, a period during which the main spring runoff occurs, major movements of the slope took place causing extensive damage to the skipway bridge and to the eastern approach haul road.

Stabilization Measures

490. The stabilization design consisted of:

a. large diameter horizontal drains from the orepass/bridge area into the slide zone;
b. drainage of the overburden materials to minimize recharge of the underlying rock;
c. drain holes into the base of the slide zone from an existing underground drift running under the central slide area;
d. removal of approximately 3 million tons (tonnes) of overburden.

The drainage and unloading programme was commenced in the early summer of 1971. Details are shown in Fig 88 and 89.

Primary Causes of Slide

491. The primary causes of instability were attributed as follows.

a. Mining operations at the slope toe gradually reduced the volume of a competent rock buttress which appeared to be retaining weak, sheared rock in the upper part of the slope (Fig 88). The buttress underwent significant deformation, initial distress being indicated by the wedge slide of January 1971.
b. Adverse groundwater conditions developed during the winter/spring break-up period. A major

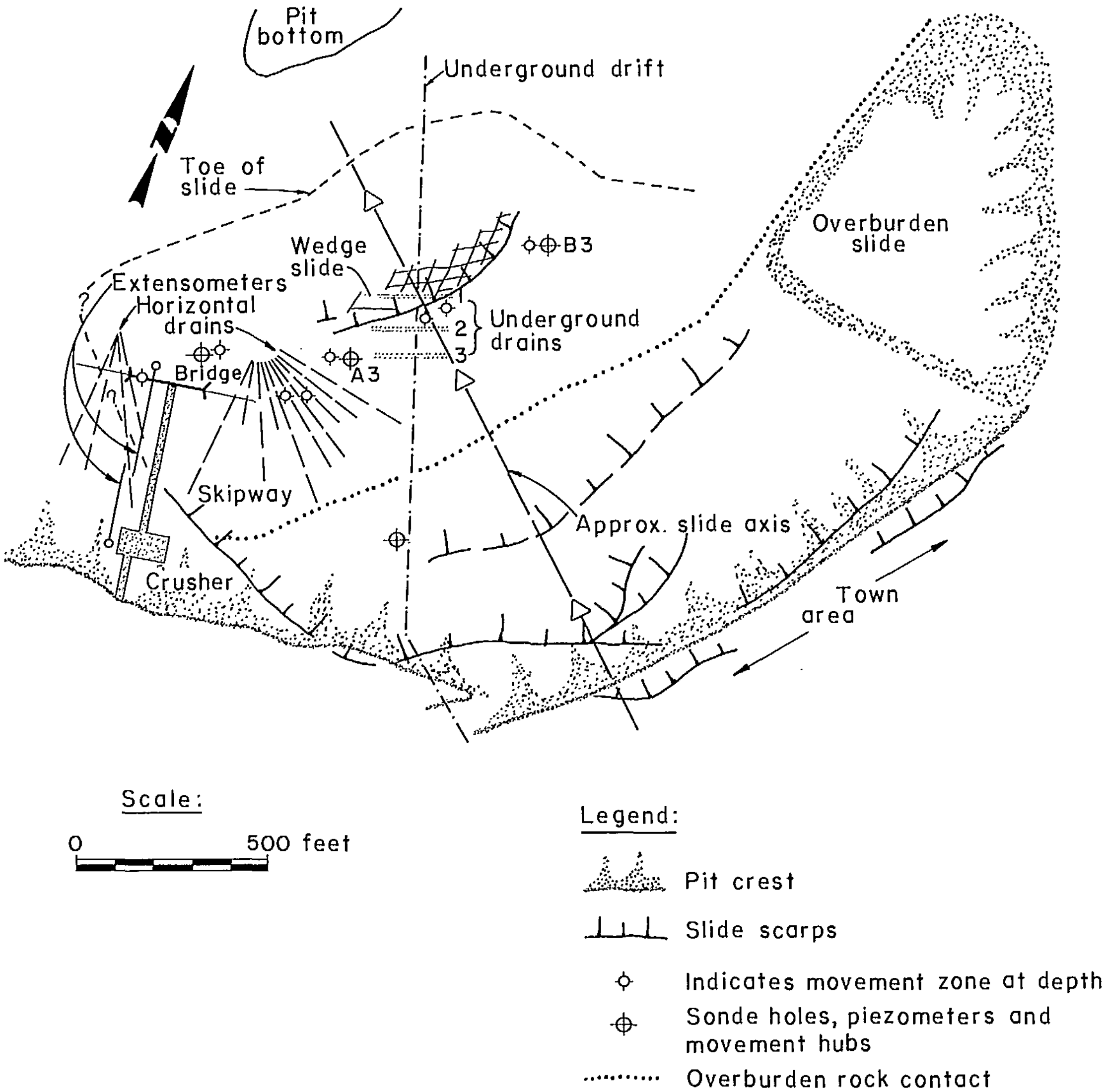


Fig 89 - Jeffrey Mine: slide area in February and March, 1971.

influx of groundwater both from the immediate overburden and from the slide to the north was thought to have occurred.

Details of Drainage Measures

492. The initial drainage measures in the area of the orepass/bridge structure consisted of two fans of horizontal drain holes drilled by a heavy duty diamond drilling rig. This equipment was chosen because of the difficult conditions in the highly variable and often sheared rock. The drains to the east of the bridge were drilled to and through the suspected slip surface.

493. On penetration of the sheared zone at the base of the slide, a considerable quantity of water under pressure was encountered. The water was contained within a permeable, dilatant zone at the base of the slide which had resulted from a movement of several feet. Previous drain holes into stable slopes in this area had resulted in only minimal flows.

494. One of the main holes in this series was temporarily closed off at the outlet and water pressures up to 75 psi (500 kPa) built up within a

matter of minutes, indicating high water pressures at a relatively shallow depth. After about three months, the installation was again closed off and pressures were found to have decreased to about 5 psi (35 kPa). Drainage of the overburden was also carried out and a considerable quantity of water was tapped and directed away from the lower slide area.

495. Twelve drainage holes were drilled from the underground drift. The drilling was carried out under the central base of the slide. All holes were drilled to intersect the slide zone as shown in Fig 90. Again a highly permeable, free draining zone several feet thick was encountered. Water under pressure initially flowed at 200 gpm (900 l/m) and dropped to about 20 gpm (90 l/min) after pressures dissipated. The relative permeabilities of the intact vs failed rock zone explained the negligible effect that the adit itself had on water pressure within the slide zone.

496. This example serves to demonstrate that, for effective drainage, the relatively permeable slide zone resulting from dilation of the rock mass during shear must be tapped to dissipate groundwater pressures. Once such zones have been intersected by drains, dissipation of pressures within the entire zone probably occurs because of the significant hydraulic conductivity of the slip surface.

Results of Stabilization Measures

497. Surface monitoring of the slide area showed a significant reduction in movement rates following completion of the initial drainage measures and overburden stripping, as shown in Fig 91. Movement rates further reduced after completion of drainage from the adit, resulting in negligible movement after August 1971. This is also indicated by the extensometer data in Fig 92.

498. The influence of groundwater on stability and the sensitivity of stability to minor changes in groundwater conditions is indicated by an increased movement rate during August 1971 following approximately 5 in. (13 cm) of rainfall within 24 hours (Fig 91).

499. Because drainage was installed during the summer period when groundwater conditions are generally more favourable, the success of the scheme could not be judged immediately. However,

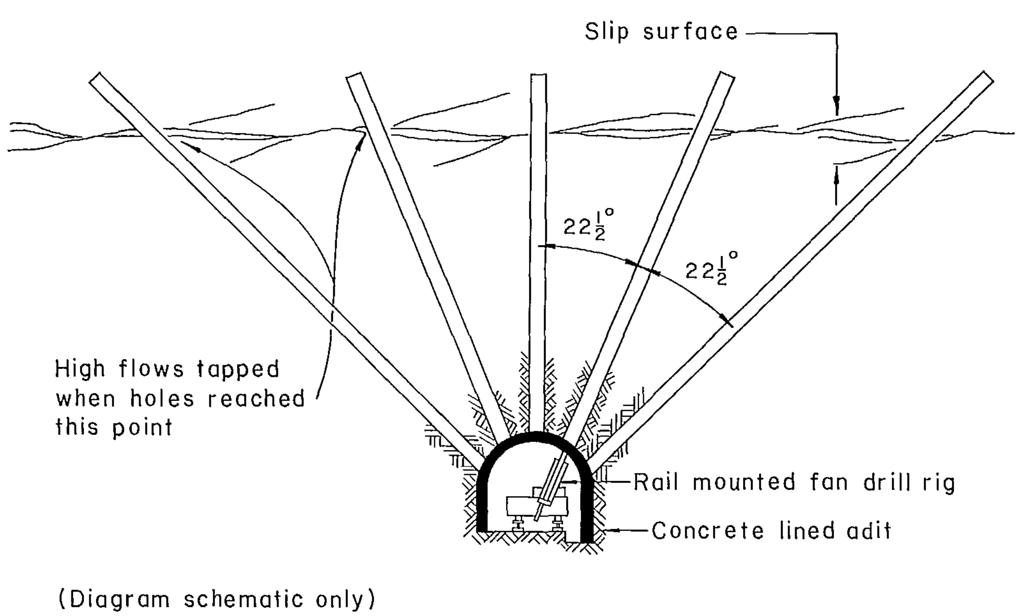


Fig 90 - Jeffrey Mine: cross section through drainage adit showing fanned drain holes.

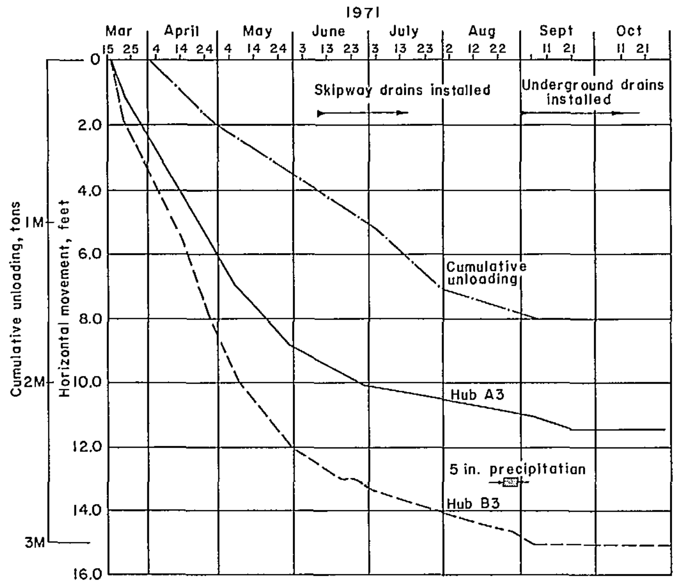


Fig 91 - Jeffrey Mine: surface movement parallel to axis of central slide area.

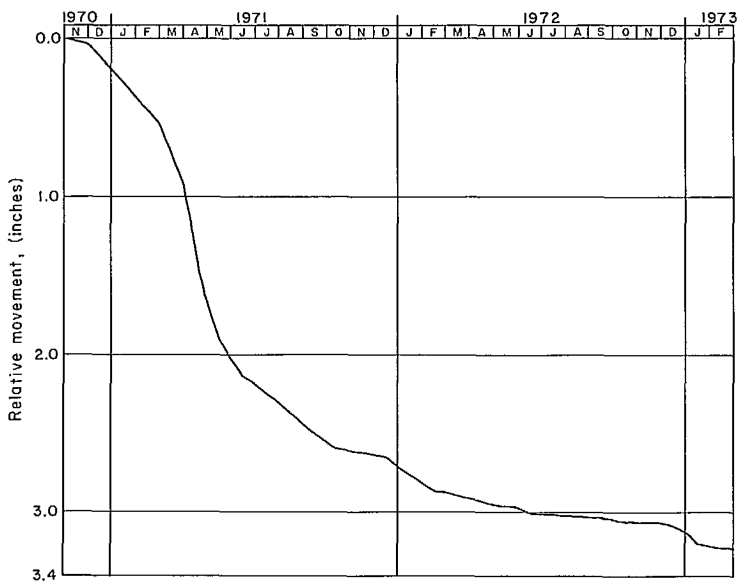


Fig 92 - Jeffrey Mine: extensometer results 1970-73: movement between surface and depth at the lower bridge anchor.

during the winter/spring period of 1972 and 1973 no re-occurrence of movement was observed and groundwater pressures within the slide zone remained minimal. It is thus concluded that drainage in this instance was of considerable benefit in controlling stability of a major rock slope and allowing mining operations within the slide area to be safely resumed.

REFERENCES

1. Muller, L. "The rock slide in the Vajont Valley"; Felsmechanik Ingr Geol., 2; pp 148-212 (English Text); 1964.

2. Londe, P. "La stabilité des massifs rocheux: Application aux barrages"; Annals Inst. Tech. Batiment Trav. Publ.; pp 1617-37; 1968.

3. Welsh Office. "A selection of technical reports submitted to the Aberfan tribunal", 2 vols, H.M.S.O., pp 240; 1968.

4. Legget, R.F. "Geology and engineering"; 2nd edition, McGraw-Hill; p 884; 1962.

5. Hazen, A. "Some physical properties of sands and gravels with special reference to their use in filtration"; 24th Annual Report, Massachusetts State Board of Health; 1892.

6. Sharp, J.C. and Maini, Y.N.T. "Fundamental considerations on the hydraulic characteristics of joints in rock"; Proc. Int. Soc. Rock Mech. Symp. Percolation Through Fissured Rock, Stuttgart; 1972.

7. Skempton, A.W. "Effective stress in soils, concrete and rocks"; Proc. of Conference on Pore pressure and suction in soils, Butterworths London; 1961.

8. Morgenstern, N.R. "The influence of groundwater on stability"; Symposium on Stability in Open Pit Mining, Vancouver; November 1970.

9. Sharp, J.C., Maini, Y.N.T., and Harper, T.R. "The influence of groundwater on the stability of rock masses: 1 - Hydraulics within rock masses"; Trans. Sec. A I.M.M.; v. 81; 1972.

10. Louis, C. and Pernot, M. "Analyse tridimensionelle des écoulements dans le massif de foundation du barrage de Grand Maison"; Proc. Int. Soc. Rock Mech. Symp. Percolation through Fissured Rock, Stuttgart; 1972.

11. Maini, Y.N.T., Noorishad, J. and Sharp, J.C. "Theoretical and field considerations on the determination of in situ hydraulic parameters in fractured rock"; Proc. Int. Soc. Rock Mech. Symp. Percolation through Fissured Rock, Stuttgart; 1972.

12. Sharp, J.C. "Fluid flow through fissured media"; Ph.D. Thesis, Univ. of London; 1970.

13. Snow, D.T. "Three hole pressure test for anisotropic foundations permeability"; Felsmechanik and Ingenieurgeologie, Springer, Vienna; v. 4, no. 4; 1966.

14. Louis, C. "Hydraulic triple probe to determine directional hydraulic conductivities of porous or jointed media"; Report D12, Rock Mechanics Project, Imperial College, London; 1970.

15. Maini, Y.N.T. "In situ hydraulic parameters in jointed rock, their measurement and interpretation"; Ph.D. Thesis, Univ. of London; 1971.

16. Sharp, J.C., Maini, Y.N.T. and Brekke, T.L. "Evaluation of the hydraulic properties of rock masses influenced by major heterogeneities"; 14th Symposium Rock Mechanics, Pennsylvania; 1972.

17. Londe, P. "The flow of water in rock"; Analysis and Design of Rock Slopes, Edmonton; Aug. 1971.

18. Hoek, E. and Bray, J. "Rock Slope Engineering"; Institution of Mining and Metallurgy, London; 1974.

19. Snow, D.T. "Fracture deformation and change of permeability and storage upon changes of fluid pressures"; Quart. of the Colorado School of Mines; v. 63, no. 1; Jan. 1968.

BIBLIOGRAPHY

The literature on groundwater is so vast that careful selection of material is necessary if the reader is not to be inundated. An excellent early - though completely valid - treatment of groundwater theory is contained in:

Hubbbert, M. King. "The theory of ground-water motion"; Journal of Geology; Volume 48 No. 8, pp 785 - 944, Nov - Dec 1940.

This treatise gives a complete development of Darcy's law, with particular emphasis on hydraulic potential and the definition of permeability.

A more recent publication, with less detail of groundwater theory but a good treatment of the effective stress principle, is:

Lambe, T.W. and Whitman, R.V. "Soil Mechanics"; John Wiley and Sons, Inc., New York, 1969.

Part IV of this book deals with the significance of water in steady-state flow conditions. Although concerned specifically with soils, the material is also relevant to rock.

Flow net construction and slope drainage are described in:

Cedergren, H.R. "Seepage, Drainage and Flow Nets"; John Wiley and Sons, New York, 1967.

Chapter 8 deals with slope stabilization through drainage; although soil slopes are primarily considered the material provides a good general background.

Information on the influence of groundwater in open pit mining is generally found in journal articles and papers, rather than in textbooks. A selected bibliography of material published since 1970, with abstracts, follows.

BIBLIOGRAPHIES

Van der Leeden, F. "Ground Water - A selected bibliography"; Water Info Centre, N.Y.; 116 p; 1974 (2nd edition).

This bibliography consists of an extensive list of references on the subject of groundwater, including the history and development of groundwater hydrology, exploration, mapping and data collection, geophysical prospecting, well logging, hydraulic characteristics of aquifers, water level, springs and geothermal resources, theory of groundwater flow, pumping tests and field observations, tracers, chemical quality and water standards, water pollution, radionuclides in groundwater, salt water intrusion, well design and drilling methods, well maintenance and simulation, pumping equipment, artificial recharge, river infiltration and storage, subsurface disposal, basin studies and water balances, phreatophytes and evapotranspiration, base flow and bank storage, groundwater models, land subsidence, groundwater law, management and conservation, education and water witching.

DRAINAGE/DEWATERING

Maini, Y.N.T. "Drainage of rock masses"; Paper no. 111-11 in Symposium.

An outline and review is made of research on the subject of the drainage of rock masses. It is covered re. the influence of surface topography of a fissure on flow characteristics (topography of a fissure, pressure flowrate characteristics of fissure, nonlinearity in the pressure flowrate relation), the difficulty of structural interpretation and its subsequent use in the numerical model, the influence of stress on permeability (mechanics of the stress-permeability interaction), and the influence of shear displacement on permeability.

Smith, J.B. "Mine dewatering with high pressure/ large capacity multi-stage centrifugal pumps"; Mining Technology; pp 478-485; Dec. 1974.

The practical aspects of dewatering with multistage centrifugal pumps are explained. Methods of establishing head requirements, resistance in pipe fittings, selection of pumps, parallel operation of pumps, and hydraulic balancing are discussed. The mechanical details provided generally apply equally to underground and open pit mine dewatering.

FLOW

Brekke, T.L., Witherspoon, P.A., Maini, Y.N.T. and Noorishad, J. "Coupled stress and flow analysis of fractured dam foundations and rock slopes"; Symposium on percolation through fissured rocks, Paper T4-J, 8 p.

The interaction between flow forces, body forces and boundary loads in structures in fractured rock masses is described. A method is given by which this interaction can be studied. In the method an iterative technique is used by which a compatible solution between stress and flow is obtained. Changes in permeability due to changes in effective stress and vice versa can be analyzed. Two examples of the method, one for dam foundation analysis and one for rock slope analysis, are given.

Han, C.Y. "The technique of obtaining equipotential lines of groundwater flow in slopes, using electrically conductive paper"; Imperial College of Science and Tech., London; 94 p; 1972.

The technique of obtaining the equipotential lines of groundwater flow in slopes of various angles using electrically conductive paper is described in this thesis. Homogeneous isotropic and anisotropic groundwater flows in slopes under normal drawdown condition and those saturated are considered. The equipotential lines obtained were used to determine factors of safety of some slopes.

Louis, C. and Pernot, M. "Three dimensional investigation of flow conditions at Grand Maison damsite"; Symposium on percolation through fissured rock, Paper T4-F, 16 p, (in French).

The hydraulic characteristics of the dam were determined from a statistical analysis of the fractures. The analyses is based on field measurements of the flow characteristics of each fracture and uses a weighting factor to determine the relative hydraulic importance of five different joint sets observed at the site. Investigations were made to analyze the influence of stresses on the directional hydraulic conductivity of the fractured system. All these data were used in a three-dimensional mathematical model that simulated steady flow with a free surface. A verification was made in the field of effects of the fractured system on flow conditions at the damsite.

HYDRAULIC PROPERTIES

Banks, D.C. "In situ measurements of permeability in basalt"; Symposium on percolation through fissured rock, Paper T1-A, 6 p.

Flow rates obtained from water pressure tests in moderately jointed basalts were correlated with joint parameters obtained from borehole camera surveys. The flow rates were predicted assuming each joint intersecting the test interval acted as an independent flow channel. Where the flow rate pressure response indicated laminar flow, the predicted flow rate, using the coefficient of laminar flow, was within an order of magnitude of the measured flow rate and a similar agreement was obtained by using the coefficient of permeability for turbulent flow.

Louis, C. and Maini, Y.N. "Determination of in situ hydraulic parameters in jointed rock"; Imperial College, Rock Mech. Res. Rep., 24 p; Feb. 1970.
The hydraulic parameters which are important in controlling the flow of water through jointed rock are discussed and theoretical and practical consideration is given to methods for the determination of these parameters in the field. Field testing requirements include either a pumping test or a dilution test. Practical suggestions regarding the performance of these tests in the field are given.

Maini, Y.N.T., Noorishad, J. and Sharp, J.C. "Theoretical and field considerations on the determination of in situ hydraulic parameters in fractured rock"; Symposia on percolation through fissured rock, paper T1-E, 8 p.
There are two kinds of features of heterogeneous rock that produce complex flow patterns - the fracture system and impermeable discontinuities such as faults and dykes. Factors are demonstrated which affect the determination of permeability of fracture systems which include systematic jointing or bedding planes in sedimentary systems, fractures of a more random nature in crytalline rock, shear zones and permeable faults. The detection, spatial orientation, and extent of impermeable barriers within the fracture systems and their influence on the overall flow conditions are considered.

Sharp, J.C. and Maini, Y.N.T. "Fundamental considerations on the hydraulic characteristics of joints in rocks"; Symposium on percolation through fissured rocks, paper T1- F, 15 p.
The basic hydraulic characteristics of the flow of water through joints in rock are examined. The applicability of classical hydraulic descriptions in relation to the flow of water through rough natural fissures is discussed. Laboratory tests to study the hydraulic flow regimes within joints are described. Empirical flow laws are derived for rough natural fissures, for the linear laminar, non-linear laminar and turbulent flow regimes at various pressure openings. The influence of applied stress on hydraulic conductivity is reviewed in relation to stressed zones within rock masses. The influence of normal and shear deformation on the hydraulic conductivity of the rock mass is shown.

HYDROLOGY

Legrand, H.E. "Overview of problems of mine hydrology"; Transactions of the Society of Mining Engineers, v. 252, pp 362-365; Dec. 1972.
An outline is made of some basic principles of groundwater hydrology that have application in mine hydrology and suggestions are made for an understanding of these principles so as to forstall internal and external problems. Described are the hydrologic setting prior to mining, induced changes in hydrology with mine development, some effects of geologic boundaries on mining hydrology, and some practical considerations. Although described in the context of underground mining, the principles apply equally to open pits.

Wright, A.P. "Design of a system for monitoring hydrologic effects of a proposed coal surface mine in Southwest North Dakota"; Preprint no. 75-F-339, Fall Mtg., SME, AIME, 18 pp; Sept. 10-12, 1975.
The selection of design criteria for a monitoring program of groundwater effects of a proposed coal surface mining operation and gasification plant is described. The monitoring program is based on existing hydrological conditions in the area, relevant legal criteria, the proposed mining and reclamation plans, and estimates of potential impacts of mining. Design of both surface water stations and of groundwater stations is described.

MINE CASE STUDIES

Atkins, J.T. and Pasha, A. "Controlling open pit failures of slopes at Shirley Basin"; Mng Engng, pp 38-42; June 1973.
Two types of instability were observed - toe instability and progressive bench instability. The causes of each are described. A program of slope stability research was undertaken and is described. A method of slide control by groundwater drainage using both horizontal drain holes and pumped wells was developed. Brief details of the installations are given.

Brashears, M.L. and Slayback, R.G. "Pumping test methods applied to dewatering investigations at Pine Point Mines, N.W.T., Canada"; Transactions of the Society of Mining Engineers, v. 252, pp 185-186; June 1972.
Pumping test methods were applied to dewatering design for a model open pit in 1968. Production wells cased with 14 in. pipe were drilled to a depth of 300 ft on the rim of the ore pit. After 2-1/2 weeks of pumping about 15 of the required 40 ft of dewatering had been achieved in the centre of the ore pit. Dewatering in open pit mines may be accomplished in a satisfactory manner by high capacity peripheral wells provided hydrogeological conditions are favourable.

Bullock, W.D. "Development of the Burnt Creek-Rowe mine complex on the Knob Lake iron range"; Canadian Mining Journal, pp 37-41, 47-49; Nov. 1972.
The Iron Ore Company of Canada operates a series of open pit iron ore mines in Northern Quebec and Labrador for direct shipping ores. A future plan to beneficiate ores from Schefferville at Sept-Iles is described as well as plant feed material. Dewatering methods in the open pits using specially constructed pump stands for sump pumping (French Mine) and deep wells, both within and on the periphery of the pit (Burnt Creek Mine) are briefly described; peripheral well dewatering is indicated to be the method most advantageous to slope stability. The design of the Burnt Creek open pit is described with respect to improved dewatering techniques, improved slope engineering and improved haul road location.

Charbonneau, D. and Morrison, D. "Mine dewatering at Knob Lake"; C.I.M. Directory, pp 113-115; 1977.
The aims of dewatering an open pit iron ore mine in northern Quebec include economics, bank stability and efficiency of operations, and the methods in use include ditches, sumps and wells. Well site selection, well drilling, developing or washing the well, pumps and sumps, ditches and dykes are described.

Garg, O.P. and Devon, J.W. "Practical application of recently improved pit slope design procedures at Schefferville"; 1st Open Pit Operators Confer., pap. 10, 10 p; May 1977.
An attempt was made to steepen the open pit angle at an iron ore mine in Quebec allowing for leached and fractured rock and groundwater. A monitoring program included shear strength determination, measuring orientation of joints, faults and bedding planes, collecting geological information from drilling, and determining permafrost distribution and water table location.

Robinson, C.S. "Ground water hydrology, Henderson Mine, Col."; A.I.M.E., Ann. Symp., Las Vegas, 55 p; Feb. 1976.
The hydrology investigation accompanied the development of the mine and involved records of the flow, chemical composition and temperature of the groundwater at and near the surface of the mine, from the entire mine and from portions of the mine and from exploration holes drilled in the mine, as well as groundwater pressures and changes in pressures with time. Geologic, hydrologic and chemical data were recorded.

Simpson, T.A. and Walker, R.A. "Slope stability and ground water control in Eufaula Bauxite District Alabama"; Trans. Soc. Mng. Engrs., v. 258, pp 265-273; Dec. 1975.
Hydrogeological methods were used to solve problems of slope stability in open pit mines in a bauxite district and hydrologic data were evaluated and used to design a system of dewatering wells. Walls of open pit mines in the

district were subsequently stabilized by the operation of the dewatering system. Some of the mines were used to develop a sequential series of guidelines that show how to effectively remove ground water from the formations surrounding the ore bodies in the district.

Vogwill, R.I.J. "Some practical aspects of open pit dewatering at Pine Point"; C.I.M. Bull., pp 7678; April 1976.
A description is given of all facets of open pit dewatering as well as possible future problems in scheduling dewatering, deep dewatering design and mining. Costs of open pit dewatering are described as well as general geology and hydrogeology, and aquifer analysis, monitoring of pit dewatering, well drilling and dewatering in semi-confined and confined water table areas.

PERMAFROST

Linell, K.A. and Johnston, G.H. "Engineering design and construction in permafrost regions - A review"; N.R.C., Div. Bldg Res., Tech Pap. 412, pp 553-575; 1973. (Similar paper with same title published in "2nd Int. Conf. on Permafrost"; USSR; 1973).
This review of the literature covers 190 references re. site selection and investigation for engineering purposes, principles of environmental engineering and protection for permafrost terrain, foundation design and construction (footings, pile foundations, ground anchors, control of frost heave, foundations in and near water bodies), roads, airfields, and railroads, excavation and underground construction, drainage, utilities, embankments and slopes, dams and reservoirs, petroleum production and pipelines, and research needs.

SLOPE STABILITY

Brawner, C.O. "Case studies of stability on mining projects"; Proc. 1st Inter. Confer. Stability Open Pit Mng, Vancouver, Nov. 23-25, 1970, pp 205- 226; 1971.
The case studies include failure of tailings dams due to an earthquake, stability of a slope in deep, overburden soil, pit slope instability in rock, pit slope instability at an open pit coal mine, investigation of seepage volume and influence of groundwater on stability, and preliminary design of slopes for a 2500 ft deep pit.

Hoek, F. and Sharp, J.C. "Improving the stability of rock slopes by drainage"; Proc. Symp. on Planning Open Pit Mines; Johannesburg; ed. P.W.J. van Rensburg; pub. S.A. Inst. Min. and Met.; pp 193-198; 1970.
The detrimental effect of water pressure due to groundwater flow upon the stability of slopes has been adequately demonstrated in geotechnical literature. The purpose of this paper is to explore the extent to which the stability of rock slopes can be improved by reducing the water pressure in the rock mass by drainage. The various methods of drainage are discussed and suggestions are given on the optimum location of drainage systems. Estimates of the increase in safe slope angle resulting from drainage are given in order that the opencast mining engineer may compare the cost of installing a drainage system with the economic benefit gained by steepening the slopes.

Lyell, K.A. "The Interpretation of water pressure factors for use in slope theory"; Proc. Symp. on Planning Open Pit Mines; Johannesburg; ed. P.W.J. van Rensburg; pub. S.A. Inst. Min. & Met.; pp 73-85; 1970.
A method of obtaining water pressure values and changes in water pressures in mine slopes is described, involving the fitting of analytical solutions to field measurements of water pressures made with piezometers. The installation and operation of piezometers and some factors affecting permeability are discussed. The approach used is illustrated with reference to design of steepened mine slopes at the De Beers Mine.

Morgenstern, N.R. "The influence of groundwater on stability"; Proc. 1st Inter. Confer. Stability Open Pit Mng, Vancouver, Nov. 23-25, 1970, pp 65-81; 1971.
The effects of water pressure on the stability of rock masses is considered. The concept of effective stress is introduced, and the physics of the

flow of water is outlined. The calculation of water pressure distributions is illustrated, and field permeability measurements are alluded to. Water pressure measurements are included.

Peuker, D. and Rechenberger, H. "Contribution on the effect of residual and surface water in open-cast brown coal mines"; New Mining Technology, v. 2, no. 10; 1972.
This translation is a condensed version of an extensive German language report by the groundwater research group of the state-owned brown coal mining industry of East Germany. The study subject was the effect of water on slope stability and includes various slope types, working faces (relatively low) slopes at ultimate pit limits (relatively high) and spoil slopes. Peripheral deep wells are the most common dewatering method. Also mentioned is horizontal berm dewatering and dewatering faces with horizontal drill holes. The economics and methodology of in-situ dewatering of coal are also discussed.

Sharp, J.C., Maini, Y.N.T., and Harper, T.R. "Influence of groundwater on the stability of rock masses - 1:hydraulics within rock masses"; I.M.M. Trans., pp 13-20; 1972.
Coverage is made of the basic aspects of groundwater flow and of their influence on slope stability. Quantitative studies of groundwater flow through discontinuous rock masses where the flow occurs in the fissures are reviewed. Sections cover the existence and effect of groundwater, the flow characteristics of fissures in rock, analysis of flow within rock masses, distribution of fluid pressures within rock masses, influence of major geological features, influence of recharge and discharge, role of fluid pressure on strength and stability of rock masses, and mobilizing forces resulting from groundwater.

Sharp, J.C., Hoek, E and Brawner, C.O. "Influence of groundwater on the stability of rock masses - 2:drainage systems for increasing the stability of slopes"; I.M.M. Trans., pp 113-120; 1972.
Drainage principles are outlined and the slope zone requiring drainage is discussed. The methods of slope drainage described include horizontal boreholes, pumped vertical wells, drainage galleries, and drainage trenches.

TRACERS

Greenfield, R.J. and Stoyer, C.H. "Tracing ground water by geophysical methods"; Fall meeting of the S.M.E., Preprint 73-F-346, 16 p; Sept. 1973.
As a support to hydrologic studies, geophysical studies were done to determine the geologic structure of an area contaminated by acid water. The geophysical work was used to define the type of information geophysical methods can give to determine which geophysical methods are most useful and to develop geophysical field procedures. Both direct current electrical resistivity and electromagnetic induction methods were used. The results and equipment used in the field studies are described.

APPENDIX A

PIEZOMETERS - CLASSIFICATION AND INSTALLATION PROCEDURES

INTRODUCTION

1. The installation of a piezometer should be regarded as part of an integrated system. The system commences with drilling and includes observation of water behaviour during drilling, detailed examination of core recovered, free standing water level measurement and permeability testing. It culminates with the selection and emplacement of a suitable tip and measuring system.

2. The successful installation of a piezometer system requires the detailed definition of the location and depth for which groundwater information is required. The general location must be chosen so that it provides groundwater pressure information which is relevant to the design or monitoring requirements. The precise position must allow for adequate hydraulic connection with the rock mass and allow the installation to be carried out effectively.

3. Ground and water conditions will affect both the type of instrument chosen and the installation procedure. These are dealt with in the following sections.

CLASSIFICATION OF PIEZOMETERS

4. A general classification of piezometers, listing the most important characteristics, is given in Table A-1 and illustrated in Fig A-1.

Response Characteristics

5. A Class Response is slow but it can be improved somewhat by using smaller diameter standpipe tubes. However, difficulties with readout

Table A-1: Piezometer types - general classification

Broad classification**	Class	Relative volume demand	Readout equipment	Major limitations	Major advantages
Open tube (standpipe)	A	High	Water level finder	Long term lag in most rock types. Tube must be straight for whole length. Difficulties likely in small diameter plastic tubes if water levels significantly below 100 ft, or dip less than ∿ 45°.	Inexpensive, simple to read.
Closed tube (hydraulic)	B	Medium to low	Usually Bourdon gauge or mercury manometer	Requires readout location not significantly above lowest water level. Therefore not suitable for general borehole use.	Relatively inexpensive. Suitable for measuring small negative pore water pressures.
Mechanical diaphgram (pneumatic or hydraulic)	C1*	Low to very low	Specialized pressure transmitter		Excellent long term stability. Can be made small. Installation simple.
	C2*	Negligible	Specialized pressure transmitter		Excellent long term stability. Can be made small. Installation simple.
Electrical	D	Negligible	Specialized electronic readout	Relatively expensive especially if cable lengths large. Some zero drift possible. Certain types may be susceptible to blast damage.	Frequency types (vibrating wire) ideal for remote monitoring. Installation simple.

* C1 Class - monitoring input closes bypass valve.
C2 Class - monitoring input opens bypass valve.
** see Fig A-1.

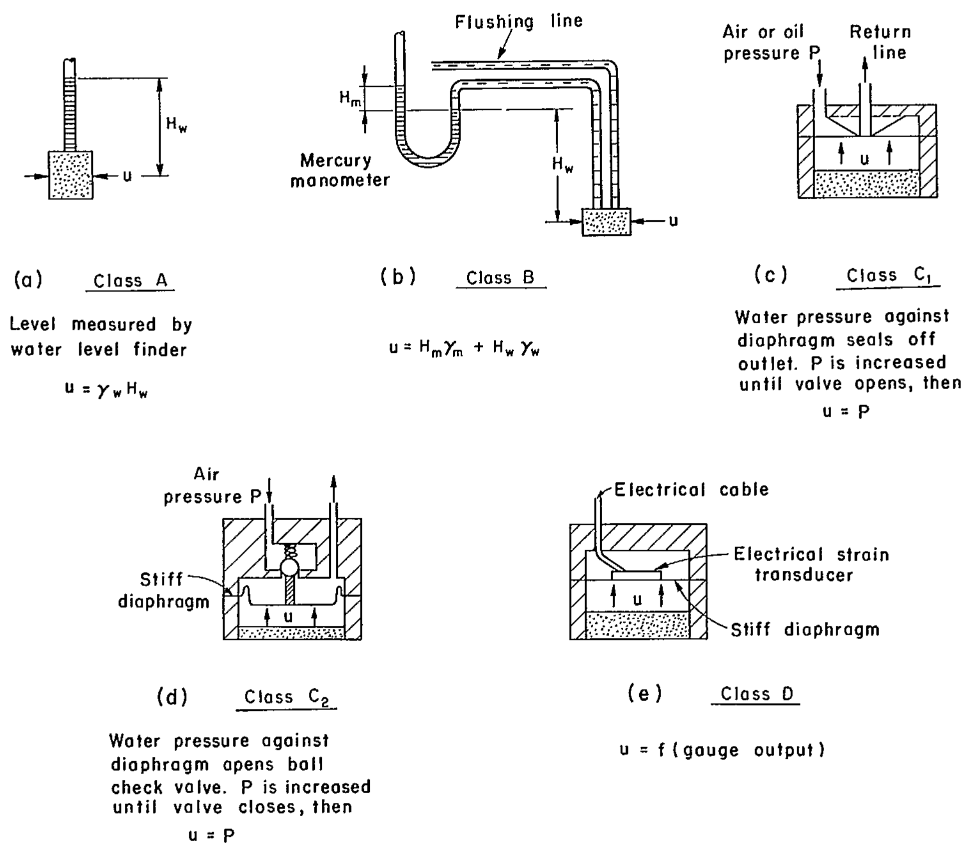


Fig A-1 - Piezometer classes showing principal characteristics.

equipment increase rapidly with diminishing size because of limitations to the size of the probe.

6. B Class Response is adequate for the majority of applications.

7. C1 Class Although apparently of very low volume demand, this type requires a volume change in the pore water when establishing the equilibrium pressures each time a reading is taken. This operating volume change varies from one instrument to another. If it is significant, it will give rise to large errors in pressure readings in low permeability systems. Instruments with these characteristics are therefore unsuitable for very low permeability materials.

8. C2 Class These are basically very low volume change devices and generally have very good response characteristics under all operating conditions.

9. D Class These are negligible volume change devices and therefore have excellent response characteristics under all operating conditions.

Readout Methods

10. A Class The water level in the standpipe is read by means of a dip-meter - a probe consisting of a graduated insulated cable, the lower end of which is weighted and terminates in two contacts. The contacts are connected to a resistance meter or to a lamp or buzzer. The cable is lowered down the standpipe until a change in resistance or the visual/audible indication is observed, indicating the water level has been reached. The instrument can be used by unskilled personnel.

11. B Class In operations where this type of

piezometer is suitable, the tubing from all piezometers is usually led to a single gauge house. A trained operator is required.

12. C Class These instruments employ supply and return air or oil tubes for pressure readout, and require specialized readout equipment. The only oil systems presently available are both Class C2. A trained operator is required.

13. D Class Vibrating wire strain gauges are most commonly used in the transducer. The operating method consists of impulsing a tensioned wire and measuring the natural frequency of the resulting vibrations. This frequency output is ideal for remote monitoring as it can readily be amplified for transmitting over any desired distance without affecting signal accuracy. A less popular alternative is the use of bonded or unbonded resistance strain gauge devices. A trained operator is required.

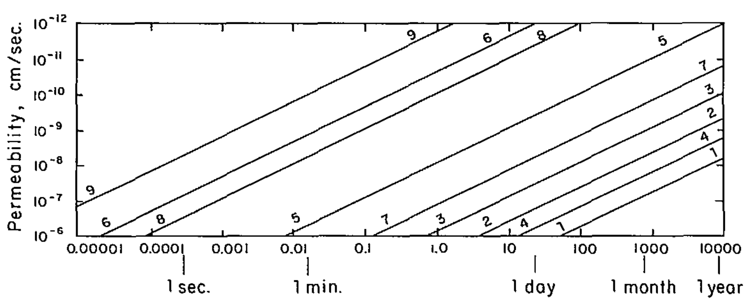
SELECTING A PIEZOMETER

14. The most important physical properties required of a piezometer installation are ruggedness, adequate accuracy and long term reliability.

15. Piezometers must be selected to suit the expected permeability of the rock mass. For accurate response to changes in groundwater pressure, the volume of water required to operate the device must be readily available from the rock mass without a significant pressure change. High volume demand piezometers are therefore unsuitable for measuring rapid pressure changes in low permeability materials.

16. The relative response characteristics of various piezometers are illustrated by the "time lag", ie, the time taken for the piezometer to reach equilibrium with the in situ groundwater conditions. The relationship between permeability and the time taken to reach 95% of the true pressure value is illustrated for various piezometers in Fig A-2.

17. The selection of a piezometer type depends on the expected rate of change of pressure with time. In many cases, pressure changes over a period of weeks rather than of days or hours are required, and the use of standpipe piezometers can be acceptable provided the limitation on response time is recognized.



Key	Class	
1	A	5 cm diameter open standpipe
2	A	Geonor borehole piezometer
3	A	Geonor borehole piezometer with sand filter
4	A	Borehole sand pocket 60 cm x 15 cm diameter + 15 mm standpipe
5	B	Borehole sand pocket 60 cm x 15 cm diameter + 3 mm hydraulic leads + Bourdon gauge
6	C2	Borehole sand pocket 60 cm x 15 cm diameter + diaphragm
7	B	Bishop hydraulic piezometer (300 m leads)
8	D	Penman diaphragm piezometer
9	D	U and A diaphrogm piezometer

Fig A-2 - Typical piezometer response characteristics.

18. Type A piezometers give satisfactory response times where water table variation is small or where permeability is relatively high. For example, a typical 99% response time for a 3/8 in. (10 mm) internal diameter standpipe piezometer having a collector zone 10 ft (3 m) long by 1.75 in. (44 mm) diameter and located in a material with permeability $k = 10^{-5}$ cm/sec would be 10 minutes; if $k = 10^{-8}$ cm/sec the time would be 7 days. To measure piezometric heads which vary significantly with time in low permeability environments, it is necessary to use piezometers with low volume demand, ie, Class D or those of low volume demand in Class C.

19. Limitations of depth must also be considered. Failure of connectors under the weight of the standpipe may limit the use of standpipe piezometers. Alloy standpipes can rarely be installed to depths greater than 1000 ft (300 m), and PVC standpipes to only about half that depth. PVC tubing may also collapse under external water pressure or be adversely affected by temperature effects during grouting. Classes B, C and D readout cables or tubes weigh less and can be installed in difficult conditions beyond 1600 ft (500 m).

20. The need for multiple installations in a single hole or in swelling ground may dictate the choice of a tip having electric cables or narrow hydraulic tubes in preference to standpipes which have a practical minimum diameter of about 1/2 in. (13 mm).

21. The readout facilities, in terms of access to the piezometers and availability of experienced staff, are also relevant. Where piezometers can be read directly at the top of the hole, pneumatic types are preferable. For centralized remote reading, hydraulic or electrical types are more suitable. With remote reading, it is necessary to provide suitable physical and visual protection for lines, etc, buried in trenches, to avoid accidental damage by excavators, graders etc.

Cost

22. In rock mechanics projects, instrument and installation costs are frequently substantially less than drilling costs, which alone may constitute 80-90% of the total cost of a piezometer system.

PIEZOMETER INSTALLATION

23. Although piezometers are simple devices, correct installation is essential for reliable results and meaningful predictions.

24. Installation details will vary with the instrument and ground conditions but a system involving the introduction of materials and equipment through the drilling rods, as with wire line drilling systems, is strongly recommended, even for shallow boreholes.

25. Two factors are important in installing piezometers:

a. If the piezometer is to measure a representative value of the groundwater pressure at a given depth, the borehole must not act as a possible flow channel, ie, the hole must be sealed along its length with a seal of about the same or lower permeability as the ground.

b. The piezometer tip itself must be exposed to the groundwater. Sufficient flow into the piezometer to satisfy the volume demand of the instrument within a reasonable response time must be possible. Hence, the higher the volume demand of the piezometer, the more care is necessary in providing an adequately permeable collector zone at its tip.

Equipment

26. The equipment necessary for most types of installation is as follows:

a. A power winch arrangement capable of handling the drill string, piezometer cables and packer inflation tubes. Generally, piezometers are installed with the drill rig in place and one or two small cable winches are used to handle piezometer and packer lines.

b. A grout pump capable of supplying approximately 10-15 gpm (45 - 70 l/min) of slurry materials (for bentonite and cement). It should be easy to clean.

c. The correct type mixing drum with calibration aids; a colloidal mixer is preferred.

d. Packer assemblies and control equipment, details of which will vary with the instrument, ground conditions and hole size.

e. Bentonite in powder or pellet form, cement, accelerator, diesel fuel, graded sand and gravel filter material, supply of clean water.
f. Piezometer and readout equipment.
g. Supply of compressed air or inert gas to 150 psi (1000 kPa).

Further details of individual components are given below under Installation Methods (para 30 et seq).

Size of Borehole

27. Borehole size, from the point of view of piezometer installation, is generally dependent on the depths at which the groundwater information is required and the number of instruments to be fitted in the borehole.

28. Depending on the depth required, conventional diamond drilling procedure is to start the holes in a large size and reduce the size with increasing hole depth as drilling conditions dictate. The normal diameter range is:-

PQ	HQ	NQ	BQ
4.83 in.	3.78 in.	2.98 in.	2.36 in.
(123 mm)	(96 mm)	(76 mm)	(60 mm)

For multiple piezometer installations this is convenient because more lines or standpipes and installation equipment are required at the top of the hole as the number of instruments increases.

29. Diamond drilling is generally necessary to provide core samples. Piezometer installation may take place in specially drilled holes or in exploration holes put down during the early stages of mine development. Because diamond drilling is expensive, it may be preferable in a working mine environment to use a percussion or rotary drill rig. Hole size is likely to be determined by the size of rig available, and is usually in the range of 3 to 10 in. (7 - 25 cm) in diameter.

INSTALLATION METHODS

Filter Technique

Standpipe Piezometers

30. Normal sizes of standpipe used range from approximately 1/2 in. to 1 in. (12 mm to 25 mm) inside diameter. Light alloy materials are preferred to galvanized steel for long-term installations. Connection strengths should be tested for suitability for deep installations. Rotation of threaded assemblies should be avoided. Flush coupled tubing is always preferred to minimize installation problems.

31. Various types of standpipe tip, most of which can be fabricated on site, are illustrated in Fig A-3.

32. It is recommended that all operations be carried out through the drill string to obviate blockage difficulties due to caving ground.

33. The prime requirement is to create a permeable zone around the piezometer tip, by placing a graded filter around it at the base of the hole or, in the case of multiple installations, on top of the previously grouted stage. Suitable size ranges and minimum layer thicknesses are shown in Fig A-4.

34. Precise gradings can be varied depending upon what is available locally. It is essential that the filter sands are clean, ie, they should contain virtually no silt or clay particles. Silt prevents grout from passing through the filter sands while setting, but must be pumped into the hole, thereby adding an operation. Alternatively the thickness of the layer of fine sand can be increased to, say, 4 ft (1.2 m) and the grout placed on it. The top portion of the sand therefore becomes blocked, but the silt pumping operation is avoided.

35. Generally, when piezometers are being installed over a prolonged period in the life of the mine, experimentation and local experience should be used to derive the filter configuration most convenient to the particular site, consistent with adequate long-term performance. Paragraph 63 et seq describes methods of checking performance. The detailed method of emplacement is as follows:

a. A 1 ft (0.3 m) minimum layer of filter material is placed initially to avoid problems due to residual fines which may have been left in the hole.
b. The standpipe is lowered into position.

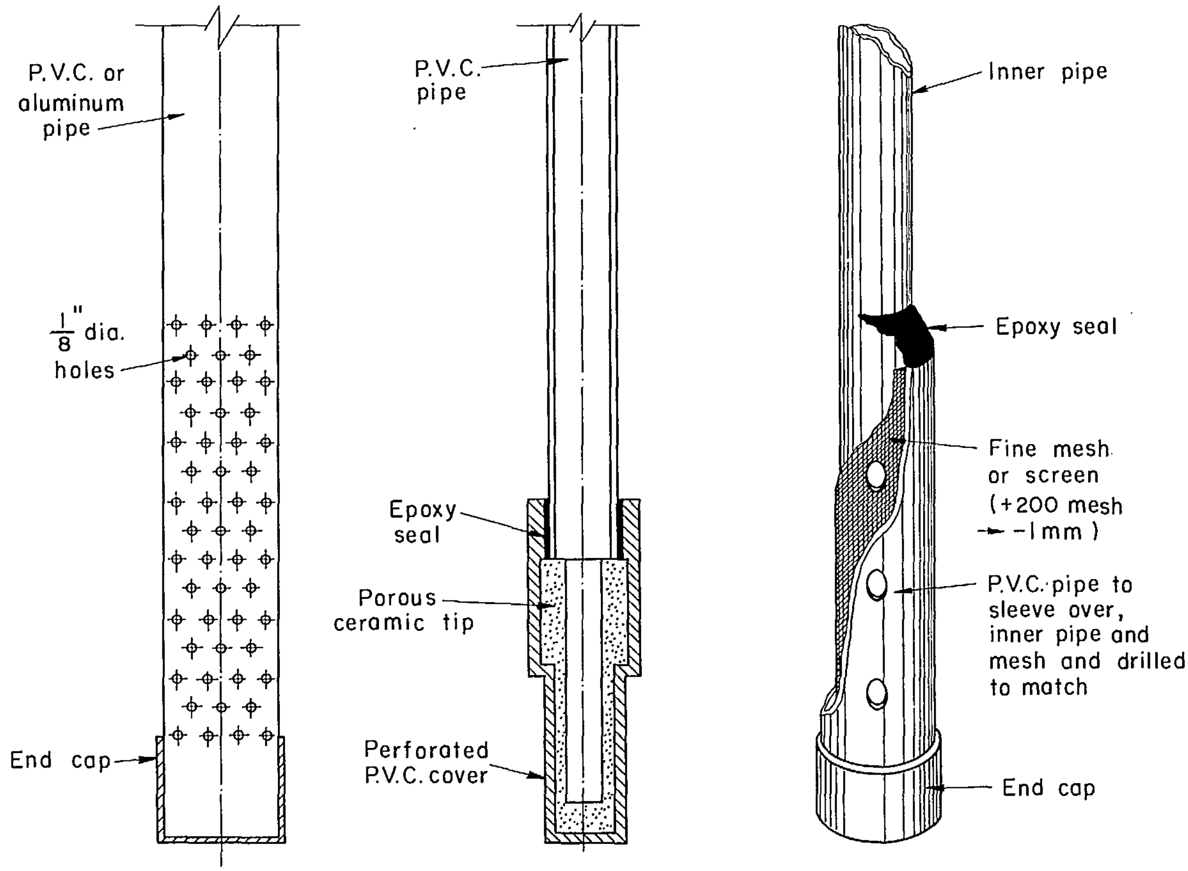


Fig A-3 - Simple types of standpipe piezometer tips.

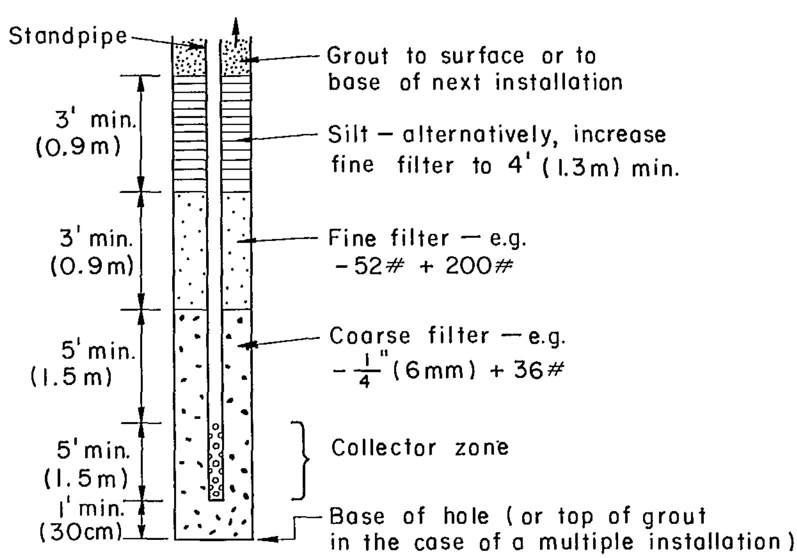


Fig A-4 - Filter details for piezometer tips.

Lengths of tube are progressively added at the top of the hole in the case of aluminum or rigid PVC pipes. If flexible PVC pipe is used the desired length is unreeled; note caution regarding possible pipe collapse.

c. The drill string is raised to the top of the proposed filter. If poor, caving ground is suspected at the piezometer tip, this operation can be delayed until after the filter has been placed. Careful tamping may later be required in this case. Coarse filter sufficient to cover the perforated zone of the tip plus 3 ft (1 m) for a combined minimum of 15 ft (1.5 m) is poured through the drill string to avoid blockage. The position of the top of the filter should be carefully checked after placing and should be compared with the measured volume using a weighted rod attached to the wireline string.

d. The drill string is then raised to the top of the proposed layer of fine filter and a layer of fine filter is poured, or preferably pumped, on top of the coarse filter.

e. After raising the drill string still further, and allowing the fine filter to settle, a similar amount of silt is pumped down.

f. The piezometer is then sealed in the borehole. Pumping is necessary to ensure that the sealant is introduced at the correct level. Two methods are possible, either

  i. use a separate tube to grout to the desired level after or during withdrawal of of drill rods, or

  ii. grout through the drill rods during withdrawal.

36. The sealant can be one of the following mixes.

a. <u>Cement-bentonite-accelerator grout</u> at a ratio of approximately 0.3 : 0.5 : 1 of water : bentonite : cement, and selecting accelerator quantities so that adequate strength and suitable setting times are obtained depending on requirements imposed by the particular site. Bentonite ensures low permeability and the accelerator decreases setting time which is particularly important for multiple installations in a single hole.

b. <u>Cement-accelerator-expander grout</u> similar to a. but using an expanding agent such as powdered aluminum to obtain a hard, strong seal, with no shrinkage.

c. <u>Bentonite</u> can be used in pellet form in holes less than 100 ft deep, and pumped as a bentonite/diesel fuel slurry to moderate depths.

37. If grouting is done through the drill rods, an adaptor to avoid filling up the standpipe during pumping may be necessary, as shown in Fig A-5.

38. The quantity of grout required should be determined from the borehole volume. Sufficient time must be allowed for the grout to be pumped into position, allowing for progressive withdrawal of the grout tube or drill rods.

39. The grouting operation must be carried out slowly. Where lengths of grout in excess of a few tens of feet are required, the operation must be done in stages; each stage being allowed to set overnight and any fluid (laitence) at the top of the grout column flushed out prior to grouting the next stage.

40. After placing, the grout will take several hours to settle. The position of the grout upper surface should be carefully checked if multiple installations are required. Additional grouting may be necessary to achieve the required level.

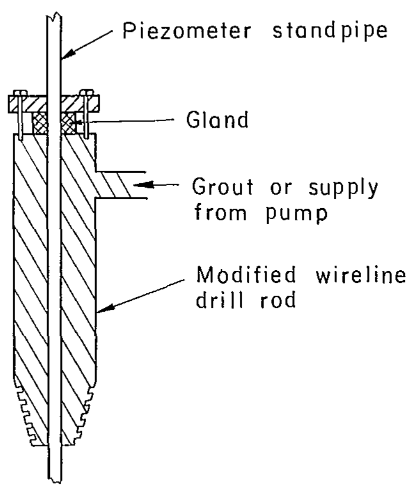


Fig A-5 - Standpipe adapter for grouting through drill rod.

41. Steps a - c are then repeated for each stage of a multi-stage installation after withdrawing sections of the drill string and grouting between each tip.

42. Some common problems which can develop are listed in Table A-2, along with some possible remedies.

Classes B, C and D Piezometers

43. The method of installing these types of piezometer, including the placing of a graded filter in the collector zone and sealing with grout is very similar to that for standpipes. This is described in para 30-42 and the details will not be repeated here. Certain differences in procedure are necessary as described below:

a. Handling readout tubes or cables, depending on the type of tip, is facilitated by the use of cable reels. Hand held reels may be used for limited depths. Power winches are necessary for installations deeper than about 100 ft (30 m) and are more convenient at any depth.
b. Stretching or kinking cables or readout tubes must be avoided when lowering instruments into the borehole.
c. In multiple installations, marked cables indicating the depth of reference number of each tip are essential to avoid errors during installation and monitoring.
d. In cases of artesian flow, it may be necessary to attach long cylindrical weights below the piezometer tip to enable it to be lowered to the desired depth.
e. With installations in deep holes, the strengths of the tubes and cables must be checked to ensure they are adequate.
f. The correct operation of the tip should be checked immediately after installation, before placing any backfill or grout.
g. When removing drill rods or casing over the readout lines a method of temporarily supporting the lines from the drilling mast will be required. One possible method is shown in Fig A-6.

Use of Borehole Packers as Grout Seals

44. The installation of a packer immediately

Table A-2

PROBLEM	REMEDY
Difficulty in introducing gravel and sealant due to artesian water conditions.	Use of packer necessary. Grout above packers, leaving void in collector zone.
Silting of piezometer tip due to inadequate amount of filter.	Pressurize standpipe and attempt to clear. Problem best avoided during installation by generous application of filter material.
Generation of heat from accelerator or expanding agents in grout, damaging PVC pipe.	Use alloy tubes or experiment with mix to avoid excessive heat generation.
Failure of connections leading to loss of standpipe in hole.	"Fishing" necessary, or if space permits, install new pipe and grout. Frequently occurs with PVC. Requires meticulous checking of each joint.
Failure of standpipe	Check strength of standpipe, use alloy or steel rather than PVC.
Loss of standpipe during rod pulling operation.	"Fish" and attempt to add extra lengths. Problem best avoided by ensuring standpipe is sitting firmly on base of hole or on initial gravel layer, and is free inside rods prior to pulling.

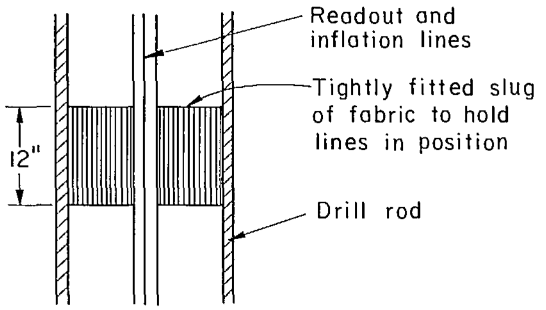


Fig A-6 - Possible support mechanism for read-out lines within drill rod.

above the required location for the tip enables a section of the borehole to be sealed off for grouting purposes. Expendable pneumatic packers are commonly used for this application. A typical pneumatic packer which can readily be fabricated on site, providing the rubber sleeve material is available, is illustrated in Fig A-7. The system shown was developed during the Pit Slope Project and is now available in several commercial forms.

45. A typical installation procedure is as follows:

a. The piezometer tip and packer unit support pipe are assembled with the piezometer standpipe, readout lines and cables passing through the packer unit and sealed internally with epoxy resin. Resin setting temperatures should be compatible with readout tubes and cables.
b. The packer unit is attached to a lowering line preferably of wire rope to eliminate stretch, and is progressively lowered through the drill rods or casing. All lines should be securely taped or banded together every 3-5 ft (1-1.5 m). If required, the unit can be weighted to facilitate lowering in artesian conditions.
c. When the instrument assembly has been placed at the required depth, the packer is inflated and checked, if possible, for sealing. This may be effected by pumping water through the standpipe or flushing lines and monitoring the piezometric pressure and return water flow.
d. Grout is then placed above the packer using the drill rods or grout tube as described for

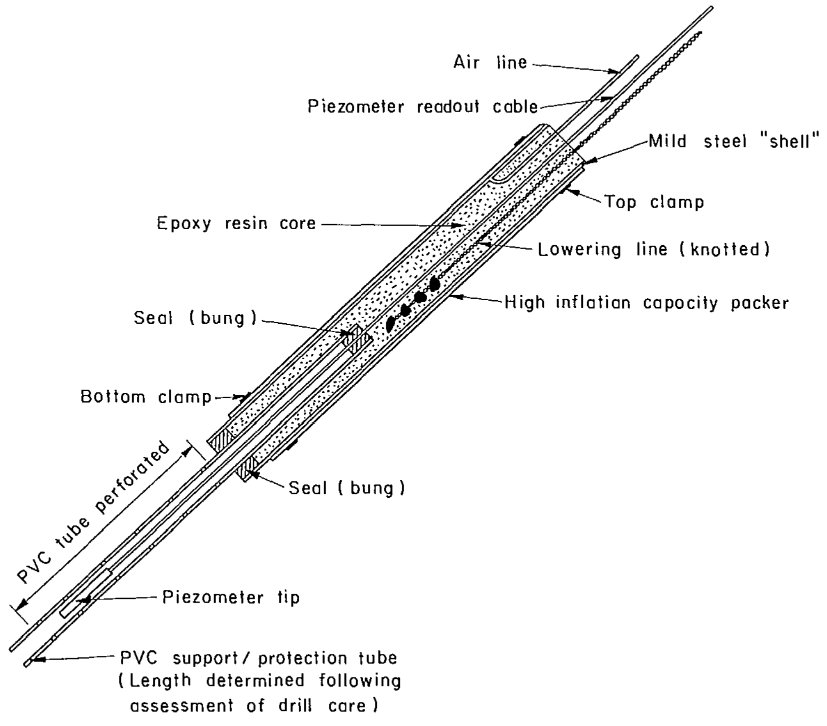


Fig A-7 - Typical piezometer packer assembly.

standpipe piezometers (para 35-40). Ample time should be allowed for the grout to set before deflating and disconnecting.

46. If multiple installations are required, this can be achieved by isolating packers below and above each piezometer, except for the initial unit at the hole bottom, which is assembled as a single borehole unit. In this case, grouting is complicated since grout and return pipes must pass through the packer units.

47. Alternatively, subsequent piezometers may be placed after the lower grout stage has set. This requires upper packers to seal against and around the readout lines to the lower piezometers, requiring very flexible packers or other special modifications.

Fully Grouted Procedure

48. If low volume response piezometers can be used for monitoring steady state groundwater conditions (Class C2 or Class D types), installation procedures may be simplified by using a fully grouted or continuous placement procedure.

49. In this procedure, the piezometer tips are normally taped together in a single string and are suspended at the required depths together with a grout pipe which is initially lowered to the hole bottom.

50. The grout mix should be carefully controlled and metered. Special mixing and watering equipment is required. The grout is placed as the grout tube, casing and drill rods - if applicable - are withdrawn. The grout mix is varied according to downhole depth so as to place a relatively permeable mix around the piezometer tips.

51. Pure sand containing a minimum of cement should be used near piezometer tips. A pure cement grout with a suitable expanding agent should be used between piezometers. The grout levels, especially for deep holes, should be checked during installation by allowing the grout to set at various stages, and then sounding.

52. In some special applications, after the grout has set, the grout and rock zone around the piezometer tip is fractured by applying a high hydraulic pressure through the tip. This creates a higher permeability collector zone. The tips must incorporate a flushing tube, which is used to apply the fracturing pressure.

Waste Dumps and Embankments

53. Piezometers can be installed in boreholes drilled in waste dumps and embankments using the methods previously described. However, there are frequently problems associated with drilling waste and embankment materials, because of the nature of the materials or difficulty of access. If the piezometers are installed prior to dumping, the cost of drilling is avoided. Two basic configurations are possible, as shown in Fig A-8.

54. In retaining embankments or waste dumps where tipping takes place in thin, nearly horizontal layers, the direct method may be used. The standpipe tube or, in the case of other types of piezometer tip, a suitable enclosing pipe is supported temporarily at each lift of tipping by timber or metal struts, sandbags, hand-placed rock or compacted fill, while tipping takes place around it. Careful control of tipping is required in the vicinity of the pipe. As tipping proceeds, additional lengths of standpipe, drill steel or conduit (as applicable) are added, the temporary support being applied each time. Joints must be water-tight.

55. Where end tipping is the mode of disposal, the remote method is necessary. The tubes from the piezometer tip are led to a remote reading location in a suitably protected steel conduit. This should be buried in a trench or covered with a minimum of 3 ft (1 m) of compacted fill to avoid damage from rocks rolling down the slope of the tip. In the case of large dumps - higher than 100 ft (30 m) - reinforced concrete protection may be necessary.

56. Classes C2 and D piezometers are extensively used in remote applications because of their remote reading facility. The decision to take a direct or circuitous route from the piezometer tip location to the readout location will depend on the amount of protection required for the portion under the dump, the topography of the site and other factors.

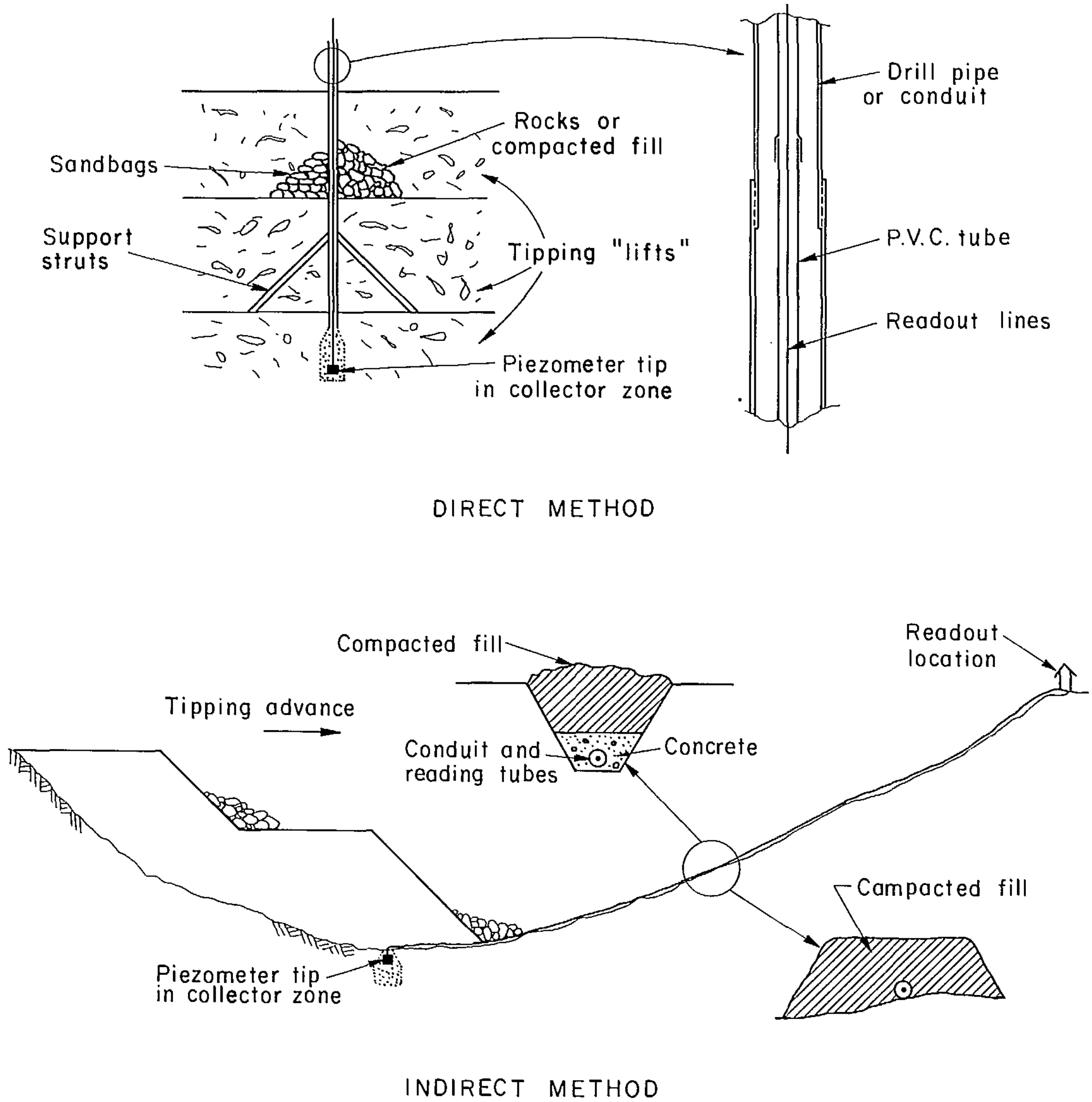


Fig A-8 - Piezometer installation in waste dumps.

Artesian Conditions

57. Artesian conditions, ie, strong upward flow of water in the borehole, makes the installation of piezometers by the normal methods already described very difficult, and sometimes impossible. Problems arise in sinking finer sizes of graded filter materials to the required depth, and grout is washed away before it can set.

58. It is possible to use a placement tube system through which the filter materials are forced into the collector zone location under pressure. However, the size of boreholes normally used and the weight of fill material restrict the size of placement tube to about 2.5 in. (6.5 cm) in diameter. Such tubes are prone to blockages caused by the "arching" of gravel under pressure, which restricts use of the system to shallow depths.

59. Successful installation under artesian conditions therefore normally involves either the

use of an expendable packer above the piezometer tip (para 44) or the application of an integral piezometer-packer unit. The unit illustrated in Fig A-7 incorporates a piezometer and can be used for this purpose; commercial units are also available.

60. Weights attached below the piezometer tip will be required in each case. Because caving ground in the hole is frequently associated with artesian conditions, the packers, piezometer tips and weights must be installed through the casing or the drill rods. This generally limits the number of installations that can be made in a single hole.

61. During installation, the flow from the hole should be observed before and after packer inflation. If the packer is operating effectively, flow from the borehole should cease or diminish substantially and the piezometric pressure will rise. If flow persists at a rate too great to allow grouting the packer must be deflated, the drill string, packer and piezometer tip raised to a higher elevation, the packer inflated and the flow observed. In cases of high artesian pressure this may have to be repeated several times to establish the top of the artesian zone.

62. Once the flowrate has been minimized, grouting and disconnecting the packer proceeds as described in para 44 et seq.

CHECKING INSTALLATION AND OPERATION

63. During installation of piezometers and while they are in operation, doubts may arise about the exact significance of the pressure being measured and whether the instrument is functioning correctly. Definite answers are frequently impossible but some helpful techniques are described below:

Checking Installation

64. Piezometric pressures should be measured during installation. Readings should be taken, where practicable, after lowering the instrument to the correct depth, after completing the collector zone, and after grouting. The pressure should change either upwards or downwards depending on the circumstances, and the time taken to show a change will depend on the volume demand of the instrument compared with the water volume available from the surrounding ground.

Checking Sealing

65. The effectiveness of sealing the tip can be assessed by measuring any change in piezometric pressure resulting from the introduction of water to the borehole above the grout. Unless the permeability is high and the hole is uncased, there should be no short-term response in the tip.

Checking Response

66. Whether or not a piezometer tip or standpipe is reacting to pressure changes may be determined by a falling head test through the standpipe or flushing tube, if fitted. In the case of a standpipe, the water level is increased as described in Appendix B, and the water level monitored regularly afterwards. It should return to the pre-test reading. If the imposed water level remains stationary, the standpipe is probably blocked. The drop in head with time should be recorded and retained as an initial calibration, against which later checks can be compared (para 70).

67. Some piezometer tips have a by-pass system for introducing a measured pressure on the tip. In this case, normal functioning is indicated if the pressure recorded by the tip is equal to the measured applied pressure.

Monitoring Long-term Response

68. In general, long standing and consistent records of instrument levels are of very great value in indicating proper functioning. Similarly, to determine whether a piezometer is operating properly, it is helpful to compare its behaviour with others in the same area; an array of instruments is therefore necessary for successful long-term monitoring. Piezometric pressures which are erratic and depart from general behaviour are usually the first indications of malfunctioning. Problematic pneumatic or electric types most frequently indicate zero or maximum readings which are indicative of blocked or broken

pipes and valves or short circuits respectively. In these cases very little can be done to overcome instrument failure. Where a long period of monitoring of 5 years or more is required, pairs of instruments can be installed to overcome the consequences of instrument failure.

69. Generally, in designing a piezometer layout for investigation or monitoring purposes, the likelihood of instrument failure must be recognized, and the number and frequency of tips in the layout specified accordingly, depending on the importance of the site and the length of time for which monitoring is required.

Assessing Deterioration

70. Periodic falling head tests (Appendix B) performed on the standpipe or piezometer tip will establish an apparent permeability of the tip and collector zone. If periodic tests, taken say every six months show that this is decreasing, allowances must be made for the increased time lag when drawing conclusions about groundwater pressure changes as indicated by the piezometric pressure recorded. Instruments which are partially blocked and therefore will not record absolute values may still be of value in indicating trends in groundwater pressure.

Rehabilitation Methods

71. Some pneumatic and electric piezometer tips have provision for flushing the tip with water under pressure to counter silting. Standpipes can similarly be cleaned by injecting water under pressure. Generally, however, flushing is of only temporary effectiveness once silt and clay have entered the collector zone, and must be repeated frequently. Attempts to clear partially blocked standpipe tips are rarely successful.

72. In some cases, it may be possible to convert a standpipe piezometer with a blocked collector zone into a class C2 or D piezometer. Several versions of miniature piezometer-packers have recently been introduced, having diameters down to about 0.5 in. (1 cm). These instruments can be lowered down a standpipe and sealed in place at the bottom. They will then accurately measure the pressure at the tip.

AVAILABILITY OF INSTRUMENTS

73. Many piezometers are manufactured commercially and are available through specialist suppliers. Instruments for specialized applications are sometimes developed by the relevant research departments of government and universities, which may be approached for advice and information and may have instruments and ancillary equipment available on sale or for hire.

REFERENCES

A-1 Terzaghi, K and Peck, R.B. "Soil Mechanics in Engineering Practice" John Wiley and Sons Inc., 1967. (Article 68 of chapter 12 provides a survey of tip types and recommendations on installation procedure).

A-2 U.S. Dept. of the Interior, Bureau of Reclamation "Earth Manual", U.S. Govt. Printing office, Washington, 1974. (Designations E-27 and E-28 in the Appendix give detailed guidance on piezometer installations in dam foundations).

A-3 Bishop, A.W., Vaughan, P.R., and Green, G.E. (1969) Report on speciality session "Pore measurements in the field and in the laboratory"; Proc. 7th, Int. Conf. Soil Mech. and Found. Eng. 3, 427. (A review of recent practice and instrumentation, related principally to earth structures).

A-4 "Suggested methods for determining in situ permeability, groundwater pressure and flow"; Int. Soc. for Rock Mechanics, Committee on Field Tests (Draft report, 1974; Final report in course of preparation).

A-5 Hanna, T.H. "Foundation Instrumentation"; Trans. Tech. Publications, Ohio, U.S.A., 1973; (Chapter 3 describes different types of piezometer tip, methods of installation, methods of recording and the protection of piezometers.)

APPENDIX B

FALLING HEAD PERMEABILITY TESTS

INTRODUCTION

1. Falling head tests are often used for preliminary groundwater evaluation. The test sections must be below the water table. The tests have the following characteristics:

a. speed

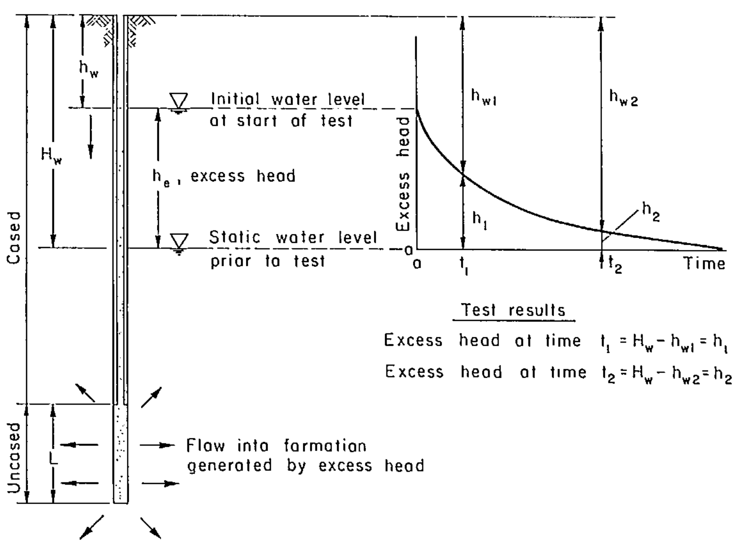
b. simplicity

c. low cost (no specialized equipment is required)

2. Permeability values obtained in the falling head test are less accurate than in other types of testing, because it is carried out under non-equilibrium conditions. The values may be lower than the true permeability due to air entrainment. In a falling head test, the permeability of the rock mass is determined from the rate of fall of an induced excess head. Generally, satisfactory results are obtained with an excess head equal to approximately half the distance between the static groundwater level and the ground surface, with a minimum of about 20 ft (6 m). Some experimentation may be necessary to determine the excess head which gives a convenient duration for the test.

3. Tests may be carried out in either an uncased hole or in an exposed test length at the base of a cased hole. Tests at successive depths to provide a "permeability profile" require drilling to be interrupted at each test location with a cased hole. Delay to drilling is minimal because the test is rapid. The same method may be applied to an uncased hole; alternatively, successive horizons may be tested after completion of the hole by using a double packer installation to isolate each test location. Staged tests may be used to produce a complete permeability profile. The use of a packer to seal the standpipe or casing above the test location is recommended. Results from tests without such a seal should be considered only approximate.

4. The principle of the test is shown in Fig B-1, and the test methods are described in detail below.



Permeability, $$k = \frac{2.3 \log_{10}(h_1/h_2)}{t_2 - t_1} \cdot \frac{r_c^2}{2L} \cdot \ln(R/r) \text{ m/min.}$$

$\ln(R/r)$ may be taken as 7

in which case

$$k = 0.133\ S\ \frac{r_c^2}{L} \text{ m/sec}$$

where S is the gradient of the log head time graph (Fig. B-3)

Fig B-1 - Principle of the falling head test.

EQUIPMENT

5. The basic equipment required for the simple forms of this test are as follows.

a. A drill rig - preferably a water flush type, to produce a borehole of suitable diameter to accept packers and standpipe, if applicable. Holes between 2 and 4 in. (5 - 10 cm) in diameter are frequently satisfactory, but larger holes may be used. For a given permeability the rate of fall of the water level will increase with increasing diameter.
b. A packer system, either pneumatic or mechanical, with packers at least 6 ft (2 m) long. Pneumatic types are generally less likely to jam in the hole. If a partly cased hole is used and the bottom of the casing is more than 6 ft (2 m) below the stable water level, an approximate value for permeability can be obtained without the use of a packer. This is sometimes sufficient for preliminary estimates but leakage past the casing can cause misleading results.
c. Hand or power operated cable drums to handle leads of the dipmeter or transducer, and of the packers.
d. Inflation control equipment for packers, and a source of compressed air or inert gas to 150 psi (1000 kPa) (ie, gas bottle or air compressor).
e. A water level indicator, normally battery-operated with markings on the cable to indicate depth, ie, a dipmeter. (Appendix A).
f. A stopwatch.
g. An alternative to e. and f. is a pressure transducer connected to a digital read-out system.
h. A pump or suitable tank arrangement and a supply of clean water. A capacity of about 50 gpm (225 l/min) is generally adequate for 2 - 4 in. (5 - 10 cm) diameter holes. Larger diameters or longer test sections will require a larger pump.

TEST CONFIGURATION

6. Two configurations are possible if a section of the borehole is to be tested as shown in Fig B-2. The packers are necessary to prevent leakage of water around the bottom of the casing/standpipe. If only an approximate average permeability of the hole is required, a partly cased hole without a packer, or an entirely open hole, can be used.

TEST PROCEDURE

7. The steps in the test are as follows:

a. Drill a hole to the depth required for the first test. Tests are preferably carried out in vertical or steeply inclined holes. Drilling muds should not be used for sections of borehole to be tested for permeability. Soluble oil - but not diesel oil - can be used as an alternative in these areas.
b. Flush out hole until circulating water is free from sediment.
c. Examine the core to determine the suitability of the test section. The examination should confirm that, where representative permeability values for a given rock type are required, no special features such as clay bands or large fissures are present; these would give rise to misleading results. The examination should also confirm the suitability of the location for the packer, ie, there should be no intense fracturing or soft ground, and the borehole wall should be smooth. If this is not the case, leakage is likely to occur, causing misleading results. Record a full geotechnical description of the test location.
d. Monitor groundwater level until it is stable and record depth, H_w, from the surface. This may take several hours (Fig B-1 and B-3).
e. Install the packer, as shown in Fig B-2, and inflate. The length of the test section may be any length in excess of about 10 ft (3 m).
f. Lower the dipmeter to the required level corresponding to the excess head, h_e.
g. Add water to the hole until the dipmeter registers consistently and start stopwatch. Alternatively, fill to slightly above the excess head level, and commence timing as the level falls past the probe, ie, as audible or visual signals are lost.
h. While the stopwatch is running, take readings of water level, h_w, with time, t, until it

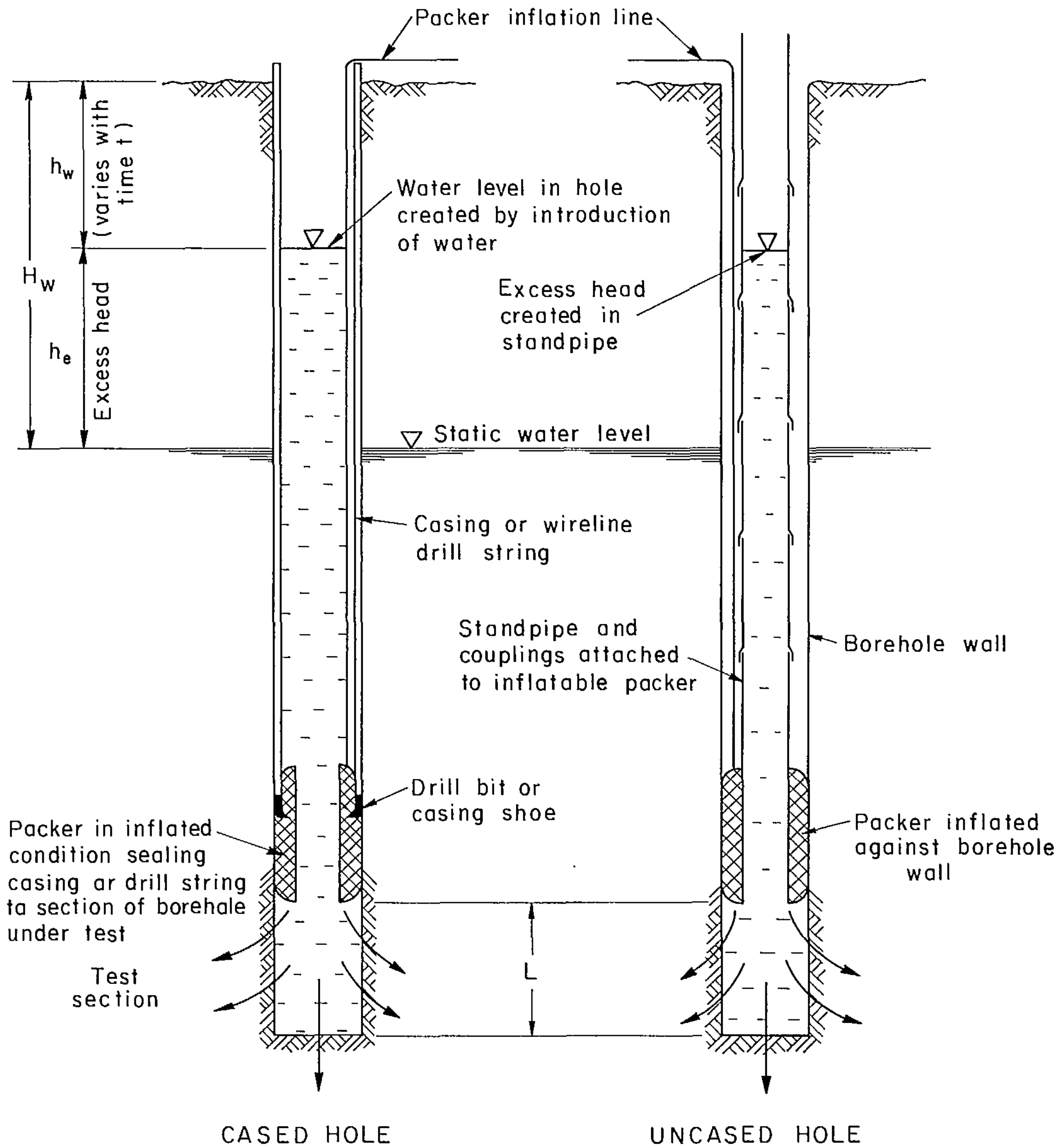


Fig B-2 - Configurations for falling head test.

reaches, or nearly reaches, groundwater level, H_W. Record results on a suitable proforma, an example of which is shown in Fig B-3. Sufficient readings should be taken to define a graph showing fall of level against time and at least two readings per minute are required in the initial period of testing. The duration of a simple test is frequently about 15 minutes and it is rare that readings beyond 30 minutes are necessary. It is usually easiest to pay out successive known lengths of dipmeter cable and note the time that the falling level passes the probe, as indicated by loss of signal.

i. Record the additional information required on the data sheet, eg, date and borehole number.

j. Check throughout the test to ensure that the original inflation pressure is maintained to confirm water-tight operation of the packer during the test.

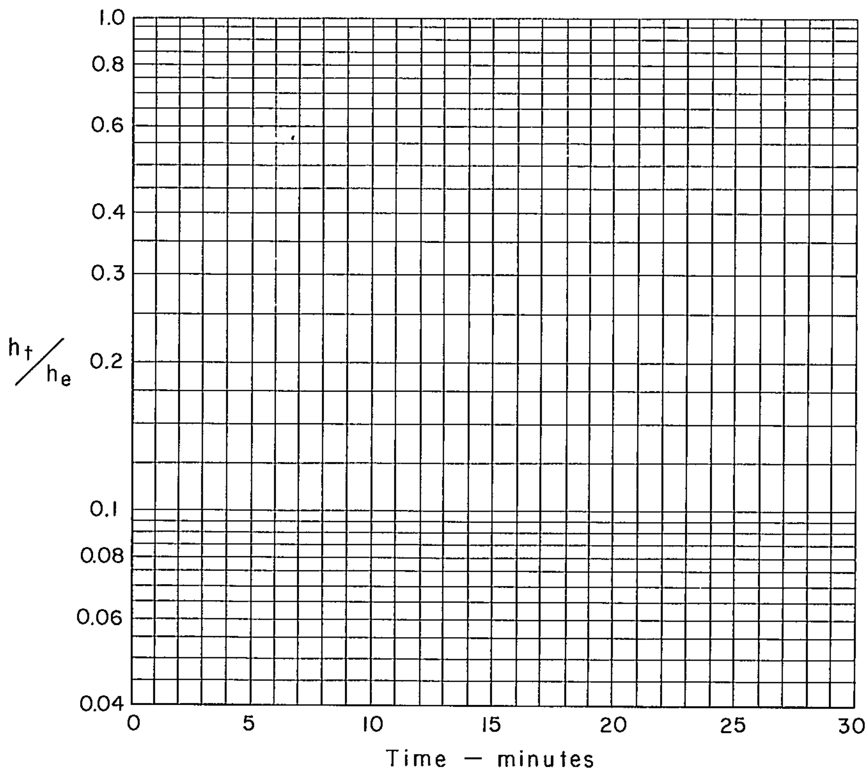
k. Remove probe; deflate and remove packer.

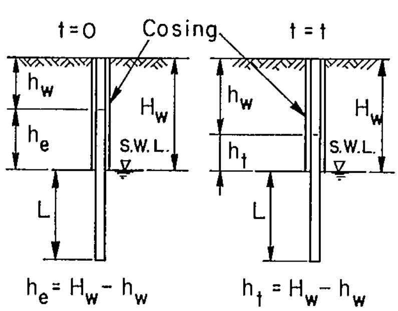
l. Drill the hole to the required elevation of the next test section and repeat steps a through l.

FALLING HEAD TEST	Test N°	Date	Project
	Engineer		Borehole
Borehole co-ordinates		Collar elevation	
Depth to top of test section	m	Length of test section, L	m
Depth of static water level, H_w	m	Radius of borehole, r	m
Excess head, h_e	m	Radius of standpipe or casing, r_c	m

Time, T (min)															
Depth to water, h_w (m)															
Excess head, $h_t = H_w - h_w$ (m)															
h_t / h_e															

Head - time graph (slope of graph is S)





Calculations

Permeability $k = 0.133\,S \left(\frac{r_c^2}{L}\right)$ m/sec.

Fig B-3 - Proforma for falling head test.

REFINEMENTS AND VARIATIONS

Use of Transducer

8. One frequent problem during testing is that the rate of fall of water level in the hole occurs too rapidly to permit accurate measurement and time recording with the stopwatch/probe system. One solution is to use a pressure transducer placed at the centre of the test section. This converts changes in pressure with time to head.

Isolating Intermediate Sections of Borehole

9. Another variation to the test is to seal off a test section above the bottom of the borehole, using two packers mounted on a standpipe perforated for the length of the test section (Fig B-4).

10. The method of installation is as follows:

a. Select test location and carry out steps b, c and d described in para 7.
b. Assemble the packers and standpipe. The inflation line for the lower packer can pass inside the pipe or be wrapped loosely around the perforated section.
c. Lower the packer assembly into position and inflate.
d. Carry out the remaining part of the test as described in para 7, steps f to k. This method is particularly suitable for existing boreholes.

Staged Testing

11. Where the casing is placed to a fixed

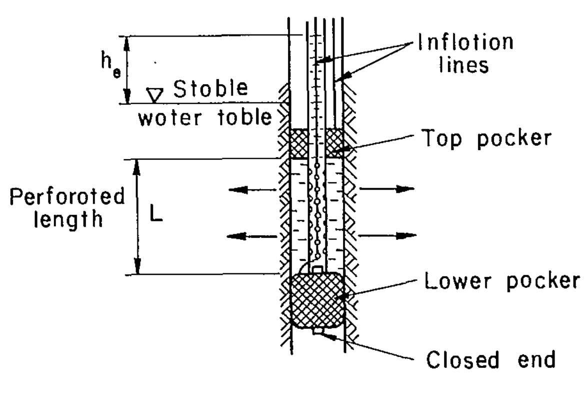


Fig B-4 - Isolating test section with packers.

depth, testing of overlapping sections of the borehole as it is advanced can be carried out to produce a permeability profile. The principle of staged analysis is shown in Fig B-5. Using this method, an average permeability value for, say, every 10 ft (3 m) section of borehole can be obtained.

LIMITATIONS OF TEST

12. Should it be found that the volume of water required to establish an excess head is beyond the capacity of the pump or the standpipe, or if the fall is so rapid that it cannot be measured accurately, it will be necessary to use the constant head test, described in Appendix C.

CALCULATION

13. The head at each increment of time is obtained by subtracting the measured depth to the water level at that time, h_w, from the recorded depth of the static water table, H_w. That is, for each increment of t,

$$h_t = H_w - h_w$$

14. For each increment of time, the ratio of h_t to the original excess head, h_e, is calculated.

ie, h_t/h_e for each increment of t.

15. Values of h_t/h_e versus t are plotted on a semi-log base. The gradient of the resulting curve, S, is measured; the average value of S should be used in the calculation of permeability.

16. The permeability, k, of an equivalent uniformly permeable mass is determined (B-1) from:

$$k = \frac{2.3 \log_{10}(h_1/h_2)}{t_2 - t_1} \cdot \frac{r_c^2}{2L} \cdot \ln (R/r)$$

in metres/min. This may be simplified to:

$$k = 0.019 \frac{S.r_c^2}{L} \cdot \ln (R/r) \text{ metres/sec}$$

where S = gradient of the head-time graph (Fig B-3) (calculations should be made for extremes of gradient but the higher value is likely to give more reliable results)

r_c = radius of standpipe or casing

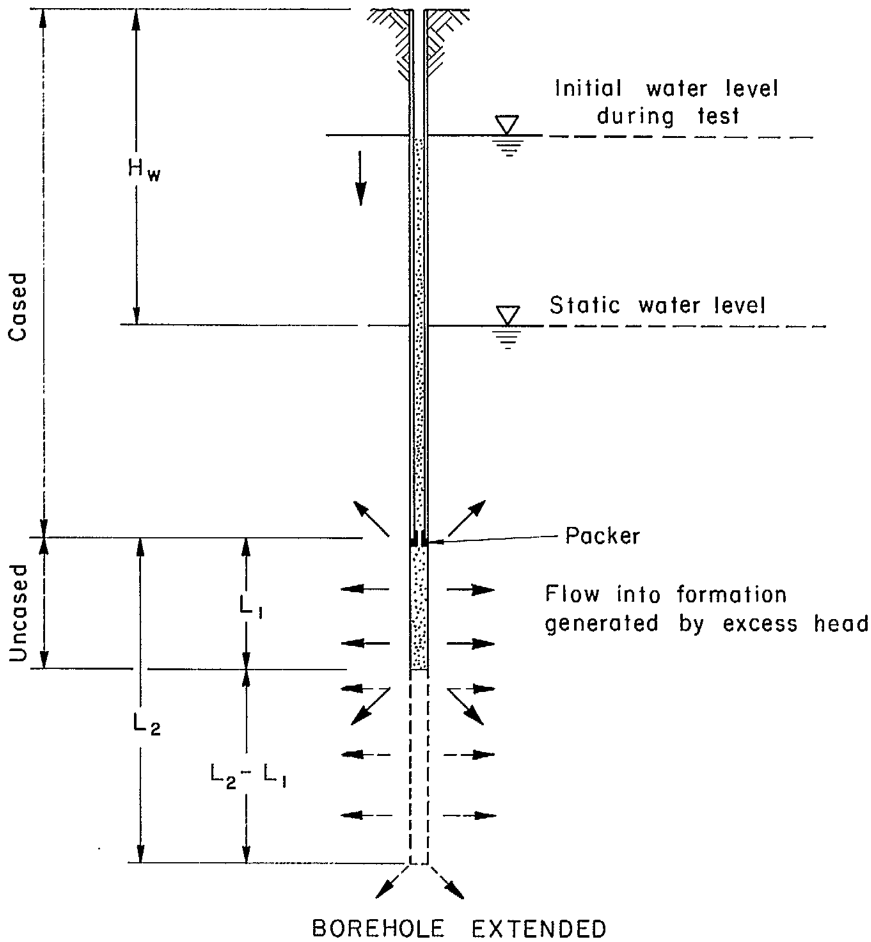
r = radius of borehole

L = length of test section

R = radius of influence, ie, the distance from the hole beyond which the excess head is ineffective. This may be assessed from neighbouring piezometers. Alternatively it is generally sufficiently accurate to assume that $\ln (R/r) = 7$

17. If $\ln (R/r)$ is taken as 7, the formula for a vertical hole may be simplified to:

$$k = 0.133 \frac{S\, r_c^2}{L} \text{ metres/sec}$$



Permeability of section $L_1 = k_1$

Permeability of section $L_2 = k_2$

Hence permeability of section $L_2 - L_1 = \dfrac{k_2 L_1 - k_1 L_2}{L_2 - L_1}$

Fig B-5 - Staged analysis for falling head test.

18. The formula above applies to a vertical hole. For inclined holes, each measured depth should be multiplied by sin α, where α is the inclination of the borehole to the horizontal, and the second term in the equation becomes:

$$\frac{r_c^{\,2}}{2\,L\,.\,\sin\alpha}$$

PRESENTATION OF RESULTS

19. Permeability values should be recorded on geotechnical borehole logs in a suitable histogram form. In this way the permeability of the various rock units can be readily correlated with other rock properties such as degree of fracturing, weathering etc.

20. Where closely spaced tests, or continuous permeability profiles have been carried out, a number of individual borehole logs/histograms can be plotted on geological cross sections (para 11). This results in an excellent visual presentation and greatly assists appreciation of the relationship between geology and permeability.

REFERENCE

B-1 "Suggested methods for determining in situ permeability, groundwater pressure and flow"; Int. Soc. for Rock Mechanics, Committee on Field Tests (Draft report, 1974; Final report in course of preparation).

APPENDIX C

CONSTANT HEAD PERMEABILITY TESTS

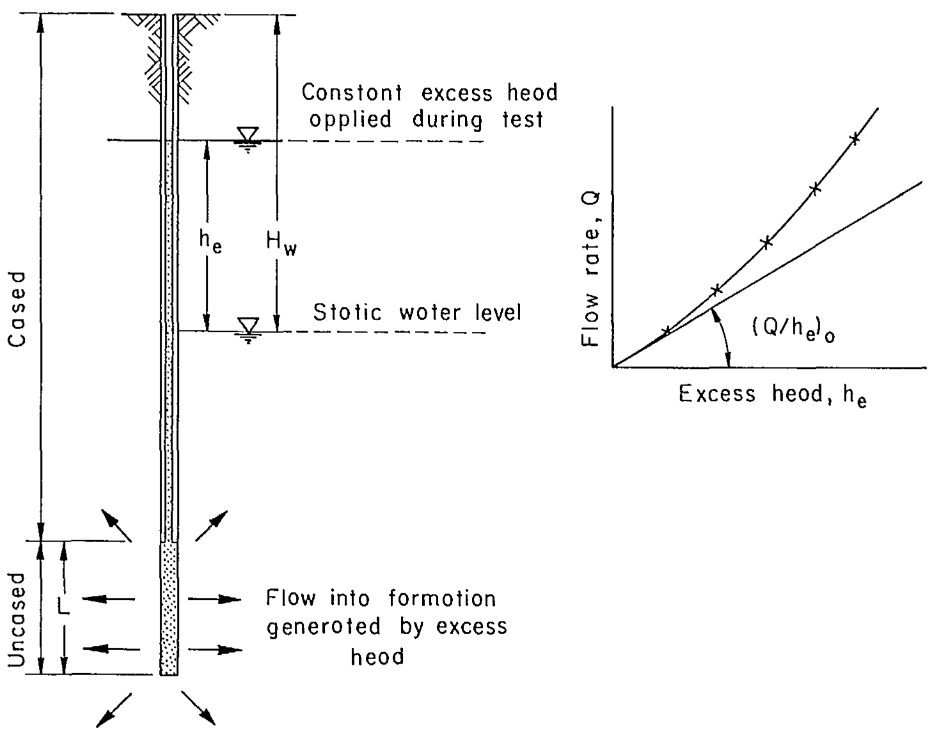
INTRODUCTION

1. Constant head tests are generally used to determine permeability values in deep or inclined boreholes. They require saturated conditions, ie, a test location below the water table.

2. Ancillary apparatus is necessary for supplying, controlling and measuring water flow. The tests take longer to perform than falling head tests, and are therefore more costly. However, the permeability values obtained are more accurate than those from falling head tests. A further advantage is that the pressure gradients applied during the test can be accurately controlled and can therefore be selected to correspond closely to the predicted field conditions, which is not always possible with falling head tests. Finally, because the range of excess pressures is applied cyclically, useful information may sometimes be obtained on the response of a fissure system to changes in groundwater pressure.

3. In general, constant head and pressure tests can be carried out for the same range of conditions as falling head tests, ie, at the base of uncased or partially cased holes, or at selected depths of an uncased hole by means of double isolation packers.

4. The test involves measuring the flow rates required to maintain a constant excess head above the water over a period of time. The concept of constant head testing is illustrated in Fig C-1.



$$\text{Permeability, } k = \frac{1}{2\pi L} \cdot \left(\frac{Q}{h_e}\right)_o \cdot \ln(R/r)$$

$\ln(R/r)$ may be taken as 7.

If Q is litres/min. and dimensions are in metres,

$$k = \frac{5.833}{\pi L} \cdot \left(\frac{Q}{h_e}\right)_o \cdot 10^{-5} \quad \text{m/sec.}$$

Fig C-1 - Principle of constant head test.

GENERAL

5. The constant test pressure can be applied either by maintaining a water level within a casing at a fixed height above the equilibrium groundwater level or by injecting water under pressure through a pipe to the test zone which is generally isolated by means of mechanical or pneumatic packers. These alternatives are illustrated in Fig C-2. For routine testing during drilling, the use of a single mechanical packer is often most effective although the relatively short sealing length of the packer may lead to difficulties in some rocks. A typical mechanical unit is illustrated in Fig C-3. The

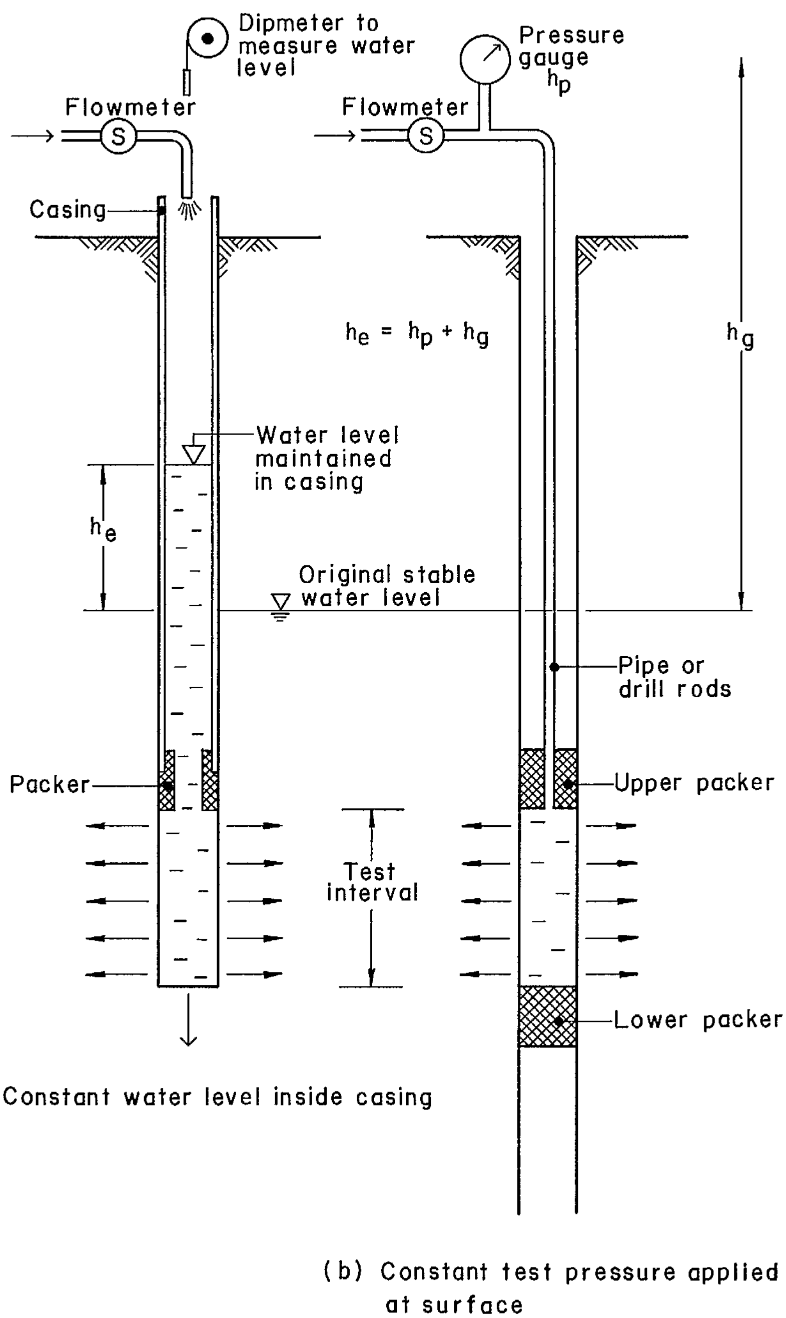


Fig C-2 - Alternative configurations for constant head test.

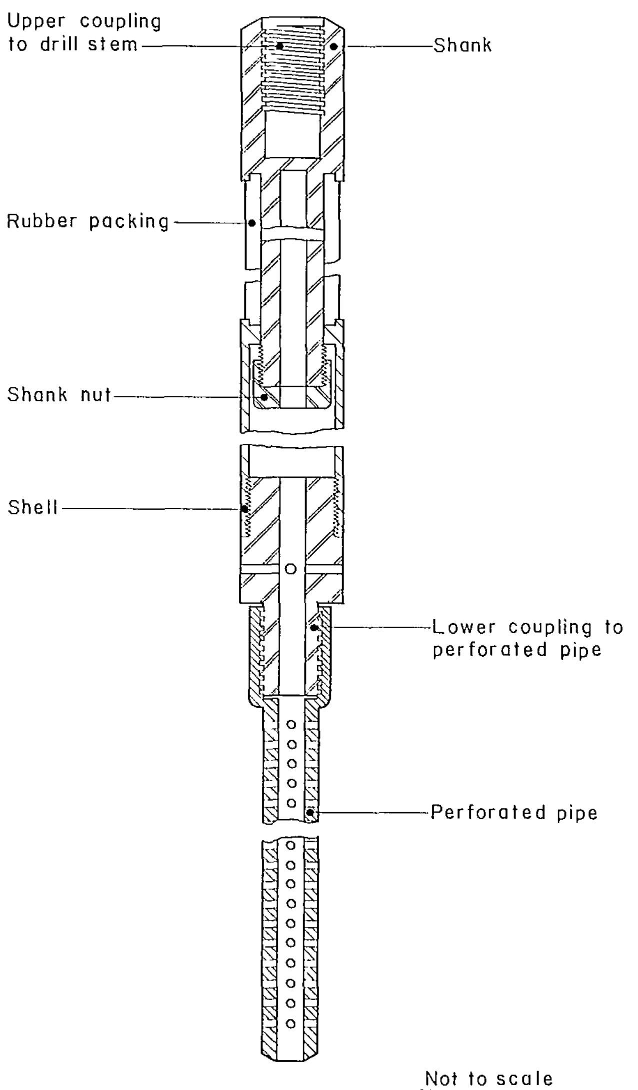


Fig C-3 - Single borehole packer unit (mechanical).

apparatus for maintaining a constant water level inside a casing is simpler than that used to maintain an externally applied pressure. Pressure applied from surface is invariably used for deep or inclined holes. The general procedures are identical in principle. The more complicated test in which pressure is applied from surface will be fully described.

EXCESS PRESSURE

6. It is particularly important in constant head permeability tests to ensure that excess pressures correspond closely to anticipated field pressures. When excess pressure is applied to the test section of the borehole, there is a risk that it may be sufficient to cause deformation of the fissures and provide misleading permeability values. It is necessary first, to select a maximum excess pressure for the test that will not cause this effect, and second, to ensure that at no time during the test, particularly when adjusting the excess pressure to the required value for the test, is the critical pressure exceeded.

7. In sound rock with fracture spacing greater than 3 in. (75 mm) and no infillings in the discontinuities pressures up to 100 psi (700 kPa) can be used with little likelihood that permeability characteristics will be altered. By checking flow rates up to this maximum pressure against the flow rate obtained with decreasing pressures, an indication of fracturing or the effect on fracture infilling can be obtained.

8. Recommended practice is to test at increments of approximately 25%, 50% and 75% of the full excess pressure before applying full excess pressure, and then to test again at equal decrements. If an ultimate excess pressure of, eg, 100 psi (700 kPa) has been selected for the test, the sequence would be to obtain averages of the flow rates for at least five consecutive minutes at 25 psi, 50 psi, 75 psi, 100 psi, 75 psi, 50 psi and 25 psi (175, 350, 525, 700, 525, 350, 175 kPa) and plot them on a flow vs excess head graph. Reasonable correlation between the flow rates for the increasing and decreasing stages of the cycle should be obtained. Figure C-4 (a) illustrates the desired result, and (b) and (c) possible results.

9. In cases where there are indications of flushing out of fissure infilling or swelling of materials within fissures, checks should be made to determine that pressures used during the test are similar to the anticipated field pressures. Washing out of infilling materials may be

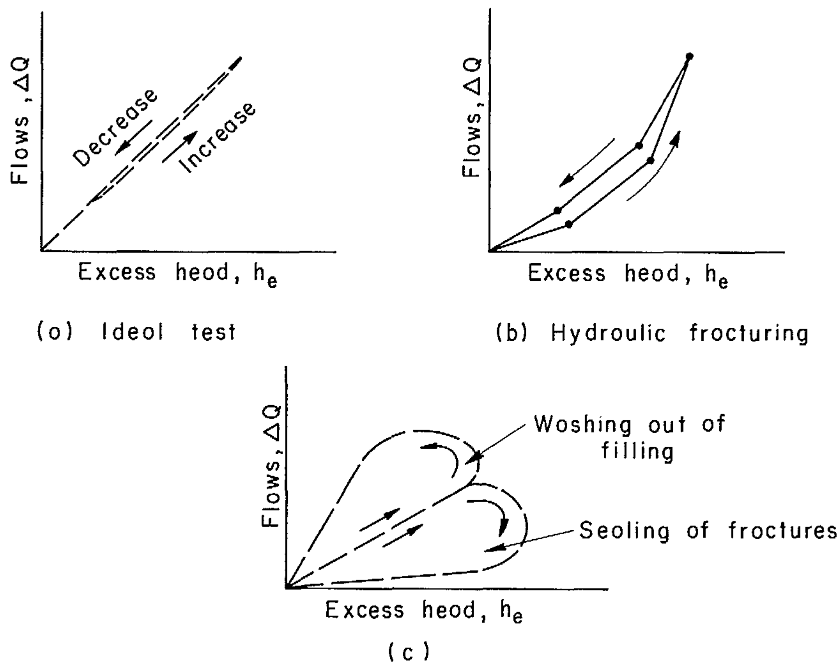


Fig C-4 - Schematic results from constant head test.

minimized by the use of lower flow rates, ie, using lower pressures. Swelling of material or silting up of fissures leading to reduced permeability is a problem unlikely to be solved by modifying the testing procedure. Permeability measurement in such material is unreliable.

EQUIPMENT

10. The general test configuration is illustrated in Fig C-5. Equipment required is as

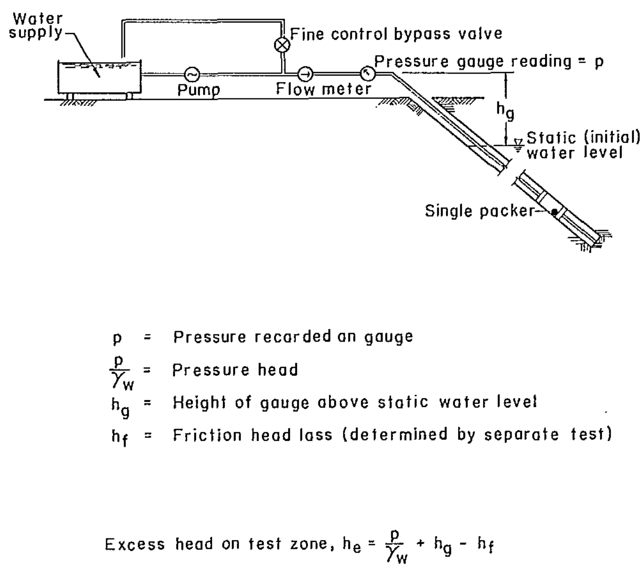


Fig C-5 - General layout of constant head test.

follows:

a. Items a - e (or g) and h as detailed for the falling head test (Appendix B).
b. A flow meter connected to the pump, capable of recording flows to ± 1 gpm (5 lpm) throughout the range of the pump, generally up to 50 gpm (250 lpm).
c. A pressure measuring device. Three possibilities in order of suitability are described below.
   i. The pressure at surface may be measured by a pressure gauge inserted in the water supply system between the flowmeter and the top of the borehole, as shown in Fig C-5. In this case the excess head at the test section is obtained indirectly from the gauge reading, using the relationship

$$h_e = \frac{p}{\gamma_\omega} + h_g - h_f$$

where h_e = excess head on the test section
p = gauge pressure
h_g = height of the gauge above the static water level
h_f = friction head loss
γ_ω = density of water

   ii. A pressure transducer and readout system may be used, with the transducer installed at the centre of the test section. This will measure the pressure on the test section directly.
   iii. A suitable device for reading the actual pressure in the test section during testing can be constructed as shown in Fig C-6. The piping used is 1/8 in. od, 3/32 in. id Nylex tube or similar, attached to the top packer so that the end of the pipe can exhaust into the test area. The test pressure is obtained by regulating the fine control valve until there is a very slow passage of compressed gas through the Nylex tube into the test section ie, one bubble per minute. The pressure indicated by the gauge with this system is then equal to the pressure of water in the test section of the borehole. The valve must be opened carefully to avoid excessive gas entering the test section.

11. Method i. requires a correction for friction loss in the pipes. This is obtained as follows:

a. Assemble the necessary supply system to reach the first test section - swivel joints, elbows,

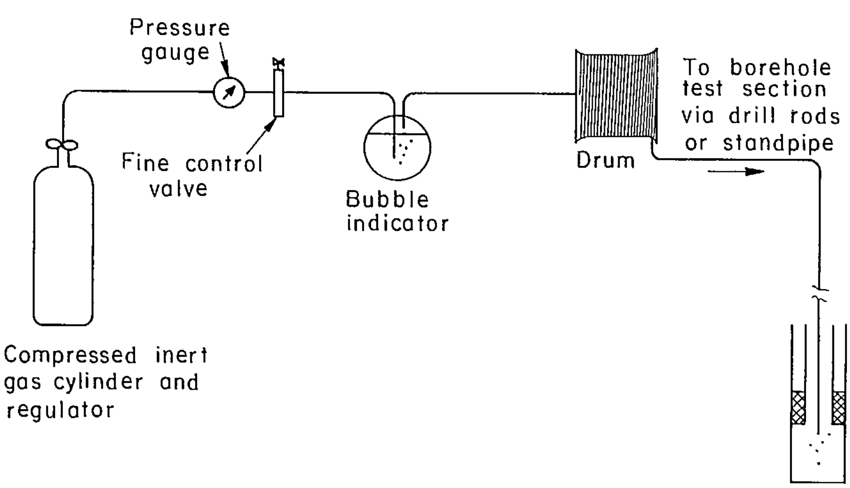


Fig C-6 - Constant head test pressure measurement using compressed gas.

drill stem or standpipe, and packers - on the ground with the discharge point at the same elevation as the pressure gauge.

b. Pump water through the system at a range of measured flows up to full pump capacity. Read and record the gauge pressure at each increment of flow. Convert pressures to heads of water.

c. Plot a curve of head loss versus flowrate. This curve can then be used to obtain the head loss, h_f, at the flowrate measured during the ensuing test.

d. Repeat the above steps for each proposed test depth.

TEST PROCEDURE

a. Follow steps a - e as detailed for the falling head test (Appendix B).

b. Connect supply pipework to drill string or standpipe. Check for leakage.

c. Start pump with by-pass valve fully open.

d. Gradually close by-pass valve until pressure in the test section is approximately 25% of the excess pressure required for the test. A single console containing the by-pass control valve, flow meter and gas-pressure needle valve with the bubble indicator, or transducer readout if applicable, is most convenient.

e. Maintain this pressure until a constant flow is obtained.

f. Record on a suitable proforma, such as Fig C-7. the pressure and flow rate at each minute for at least five consecutive minutes. Determine the average flow rate. Plot test pressure vs average flow. Fig C-7 applies to a test using a pressure gauge at the surface. It can be suitably modified for direct pressure measurement.

g. Record the other relevant data such as date, borehole number, etc, on the proforma.

h. Repeat steps d to f at each of the required pressures in the cycle as described above (para 8).

i. Deflate or remove packers and relocate at a new test location. This may be at greater depth in the same hole, after further drilling if necessary, or in a new hole. Drill rods are added or removed as necessary. Repeat steps b - h.

CALCULATION

12. All measured pressures should be converted to head of water, using the relationship

$$h = \frac{p}{\gamma_\omega}$$

where h = head

p = pressure

γ_ω = is the density of water

13. The values of stable flowrate, Q, are plotted against excess head, h_e, as shown on Fig C-1. The curve is extrapolated to the origin.

14. The gradient, measured from the origin, is determined, $(Q/h_e)_o$

15. The permeability, k, under constant flow conditions, assuming flow perpendicular to the borehole axis, is determined (C-1) from:

$$k = \frac{1}{2\pi L} \cdot \left(\frac{Q}{h_e}\right)_o . \ln\left(\frac{R}{r}\right)$$

where R = radius of influence (see Appendix B, para 16)

r = radius of the borehole (test cavity)

L = length of the test section.

16. If Q is in litres/min, all dimensions are in metres and if ln R/r is taken as 7, this may be simplified to:

$$k = \frac{5.833}{\pi L} \cdot \left(\frac{Q}{h_e}\right)_o \cdot \times 10^{-5} \text{ m/sec}$$

17. Correction of the calculation in the case of inclined holes is precisely the same as for the falling head test (Appendix B, para 18).

STAGED ANALYSIS

18. The description of staged analysis in Appendix B (para 11) applies equally to constant head tests.

CONSTANT HEAD PERMEABILITY TEST	Test Nº	Date	Project
	Engineer		Borehole

Borehole co-ordinates	Collar elevation
Depth of borehole m	Length of test section m
Length of casing m	Radius of borehole, r m
Depth of static water level, H_w m	Gauge height above S.W.L., h_g m

Test * pressure	Average flow		Increasing pressure							Decreasing pressure						
			Time - minutes							Time - minutes						
		Meter reading														
		Take														
		Flow														
		Meter reading														
		Take														
		Flow														
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		Flow														
		Meter reading														
		Take														
		Flow														

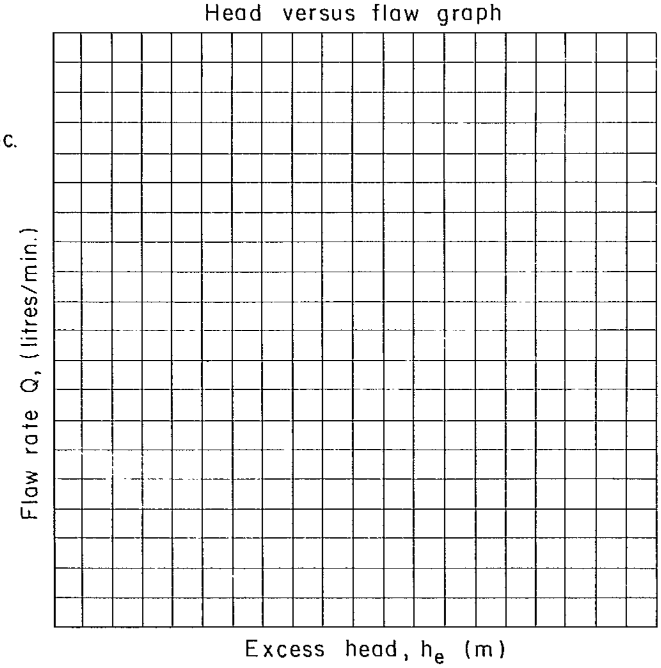
1. Gauge pressure h_p *												
2. Gauge height h_g (m)												
3. Friction loss *												
Flow (litres/min.)												
Excess head 1+2-3												

Calculations

Permeability

$$k = \frac{5.833}{\pi L} \cdot \left(\frac{Q}{h_e}\right)_a \cdot 10^{-5} \quad \text{m/sec.}$$

Head versus flow graph



NOTE
* All pressures to be expressed as heads (m H_2O)

Fig C-7 - Proforma for constant head test.

REFERENCE

C-1. "Suggested methods for determining in situ permeability, groundwater pressure and flow"; Int. Soc. for Rock Mechanics, Committee on Field Tests (Draft report, 1974; Final report in course of preparation).

APPENDIX D

WELL TESTS

INTRODUCTION

1. Well tests, which were developed primarily to determine the yields and capacities of aquifers for water supply, also provide a method of determining permeability. Groundwater up to 500 - 1000 ft (150 - 300 m) from a well centreline is frequently affected by pumping from the well, so the values of permeability obtained may be indicative of the average mass or bulk permeability of a large volume of rock.

2. A well test is expensive and is rarely justified solely for permeability measurements. It is common to use data from well tests carried out for water supply or groundwater control purposes to determine permeability. A large number of falling head or constant head tests can be carried out for the cost of one well test, and unless accurate large scale results are imperative, many small tests over a large area are often preferable.

3. The tests require the water to be pumped out of a well at a constant rate over a period of time while observations are made of the water levels in a number of nearby wells.

4. There are a number of methods for conducting the tests, and selecting the most appropriate depends on such factors as the type and nature of the aquifer, the expected permeability of the ground and the time available in which to conduct the tests. The details given in this appendix refer to well testing in an unconfined aquifer; other texts deal with confined (artesian) and partially confined (leaky artesian) conditions (D-1, D-2).

5. Test locations should be chosen with knowledge of the geology. Sufficient prior testing and exploration is required to ascertain the geological boundaries, such features as faults in the test vicinity, the approximate mass permeability of the rock, and standing water levels. Knowledge of these parameters ensures that a test location is not chosen in which the results will be unreliable or the test impossible. Generally, tests should not be made where the geology is complex or where permeability is highly anisotropic, because interpretation of results under such conditions is very difficult. Saturated rock reasonably close to the surface and a moderate permeability are required to give drawdown within an acceptable period of time.

6. The technique involves equipment and expertise which may not be available at some mines, and therefore, the test might best be carried out by a specialist contractor to the water supply industry.

EQUIPMENT

7. The following equipment is required:

a. A drilling rig capable of drilling holes of suitable depth and diameter for pumping well and observation holes. Depending on the required pumping rate, the former may need to be up to 12 in. (30 cm) in diameter. Cored or - if additional geological information is not required - percussion holes are suitable. Drilling mud or other additives should not be used.

b. Equipment and material for installing one piezometer in each observation hole. See Appendix A for selection of correct type of piezometer.

c. Pump capable of being controlled to provide a delivery rate constant to within 5% for drawing water from the well at the range of heads to be covered by the test, ie, from initial water table level down to the maximum anticipated drawdown elevation. All discharge pipes should have a gate or globe valve with which to regulate flow. The pumping rate should be checked at regular intervals throughout the test, preferably at about the same times that water level measurements are made. This will enable a close watch to be kept of the pump so that adjustments can be made if there is any change in delivery rate. Normally a 6 in. (15 cm) submersible pump is used but if the water table is close to surface and drawdown is not more than 15 - 20 ft (5 - 6 m), a suction pump can be used. The smallest submersible pump available for borehole applications is approximately 4 in. (10 cm) OD. Pump capacity must be at least 50 to 150 gpm (200 - 700 l/min).

d. Well screens (D-1) are required for tests carried out in overburden or soft rock.

e. Piping to convey pumped water out of the zone of influence of pumping to avoid re-circulation. Often 500 - 2000 ft (150 - 600 m) is required, depending on scale of test.
f. A method of measuring flow rate from the pump. For well tests run at rates of up to 100 gpm (450 l/min), the simplest method is to discharge the pumped water into a container that has a known volume - eg, a 45 gallon (200 l) oil drum - and record the time required to fill the container. If the pumping rate exceeds 100 gpm (450 l/min) it generally has to be determined in one of the following ways:
   - by standard orifice fittings on the discharge pipe,
   - by measuring the flow with a weir, or
   - by measuring the flow with a flowmeter on the discharge pipe.
g. Water level dipmeter capable of accurately measuring the depth of water in the piezometer standpipes and in the well (Appendix A).
h. A timing device, usually a battery operated clock, for accurate measurement of elapsed time, ie, between each measurement of the water level and commencement of pumping.

TEST CONFIGURATION

8. The layout of and section through a typical well test are illustrated in Fig D-1.

9. The diameter of the pumping well should be sufficient to permit insertion of a pump of suitable type - say 8 in. (20 cm) diameter for a 6 in. (15 cm) pump. It should penetrate the full depth of the aquifer and allow water to enter over

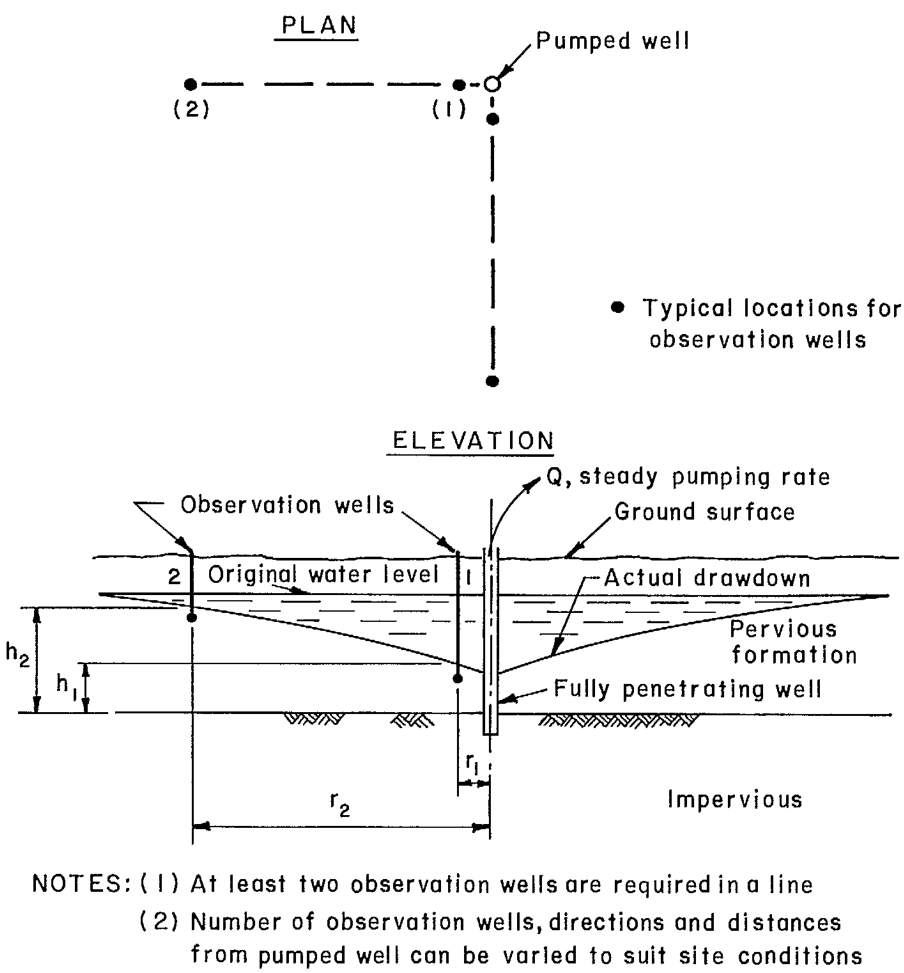


Fig D-1 - Typical arrangement of a well pumping test for determining permeability.

the full length of the well passing through the aquifer.

10. The observation holes need only be of sufficient diameter to permit installation of the standpipe piezometers, say 2.5 in. (6 cm), and of sufficient depth to cover the lowest water level that will be obtained during the test.

11. The tests may be conducted under non-equilibrium conditions; that is, when the drawdown of the water table has not yet reached a constant profile (D-3). This allows the tests to be completed in a considerably shorter time than otherwise would be required under equilibrium conditions and with no significant loss in accuracy.

PROCEDURE

12. The test procedure is as follows:

a. Drill the pumping well and install well screens, if required.
b. Pump with clean water to clear drilling debris. Develop the well by swabbing or surging at different pumping rates along the length of open screen. This increases efficiency of the well. If drawdown data from the pumping well is not used in the analysis, then development is unimportant.
c. Drill the observation wells and install piezometers (Appendix A).
d. Measure the static water levels in the well to be pumped and in all the observation wells every 2 - 3 hours for several days preceding the test to determine the magnitude of water level fluctuation. Record the depths of water and the exact times of the measurements.
e. Using the pump or a bailer (D-1), determine the extraction rate that will lower the water level by at least 15 ft. This is the pumping rate required for the test. If the permeability is high, it may not be possible to lower the water level by this amount in a reasonably short time. In this case the maximum pumping rate should be used. Pumping rate can be controlled by throttling the outlet valve or by recirculating a proportion of the discharge.
f. Allow recovery until all the water levels are as close as possible to those measured before the bail test. Start the test at the previously determined pumping rate and record the exact time (this is time zero, t = 0).
g. Read and record the depths of water in the well and observation holes and the times, for a series of time increments on a logarithmic scale, typically as follows:
   - as often as possible during the first minute;
   - every 30 seconds from 1 to 5 minutes;
   - every minute from 5 to 10 minutes;
   - every 5 minutes from 10 to 30 minutes;
   - every 10 minutes from 30 minutes to 1 hour;
   - every 15 minutes from 1 hour to 2 hours;
   - every 30 minutes from 2 hours to 4 hours;
   - every 60 minutes from 4 hours to 12 hours;
   - every 120 minutes from 12 hours to 24 hours;
   - if the test is longer than 24 hours, every 6 hours until 48 hours and then every 12 hours.
h. For the recovery test, stop the pump and start taking timed water level measurements immediately to determine the rate of recovery, following the same procedure as used in the drawdown part of the test. The recovery curve tends to mask any minor fluctuations of pumping rate during drawdown, behaving as if the rate were constant. Thus if the variation in pumping rate were less than ± 5 to 10%, an average rate can be used in the calculation. Recovery measurements should be taken until the water levels approach the original static levels.
i. Data, including date, time, hole number, location and ground-level for the pumping and observation wells should be recorded, together with the water levels at the pumping and observation wells against the corresponding elapsed times since pumping was started. The pumping rate at each time should also be noted. Figure D-2 is an example of a suitable pro-forma.

CALCULATION

13. For unconfined water table conditions, a correction must be made to the measured data before using a graphical solution for permeability. The correction allows for the fact that the saturated thickness of the aquifer is continuously decreasing during pumping and increasing during

WELL TEST	Test N°	Date	Project
	Engineer		Well N°

Well co-ordinates	Collar elevation
Depth of well 7 m	Thickness of saturated aquifer 2.44 m
Depth to static water level 4.56 m	Distance to pumping well 30 m
Total pumping time 350 min.	Pumping rate 473 l / min.

Notes :

Pumping level (m)	Drawdown S (m)	Pumping/ recovery time t/t'; (min)	Recovery level (m)	Residual drawdown S' (m)	$\frac{T}{t'}$	Comments
20.13						
20.13	0	0				
30.87	10.74	1				
32.72	12.59	2				
33.05	12.92	3				
33.46	13.33	4				
33.63	13.50	5				
33.68	13.55	6				
33.71	13.58	7				
33.77	13.64	8				
33.81	13.68	9				
33.84	13.71	10				
33.89	13.76	12				
33.93	13.80	14				
33.96	13.83	16				
34.01	13.88	18				
34.04	13.91	20				
34.10	13.97	25				
34.14	14.01	30				
34.19	14.06	35				
34.27	14.14	45				
34.31	14.18	50				
34.33	14.21	60				
34.37	14.24	70				
34.42	14.28	80				
34.45	14.32	90				
34.47	14.34	100				
34.54	14.41	120				
34.55	14.42	140				
34.55	14.42	160				
34.58	14.45	180				
34.61	14.48	200				
34.66	14.53	250				
34.74	14.61	300				
34.84	14.71	350				

Fig D-2 - Proforma for recording pumping test results.

recovery. The correction is:

$$s_{corrected} = s_{measured} - \frac{s^2_{measured}}{2m}$$

where s is the observed drawdown and m is the original thickness of the aquifer before starting the test.

14. This correction must be applied to drawdown measurements from all wells, but will be negligibly small for a thick aquifer of high permeability, for the early period of the test when drawdown is small, and for observation wells most remote from the pumping well. To save time, the correction should be applied to the latest data first.

15. The technique in applying the modified straight-line non-equilibrium equations (D-4), is to plot the corrected drawdown of the observation wells against the logarithm of elapsed time since pumping started.

16. This plot (Fig D-3) will give three straight line components for unconfined conditions if sufficient measurements are taken at the start of pumping.

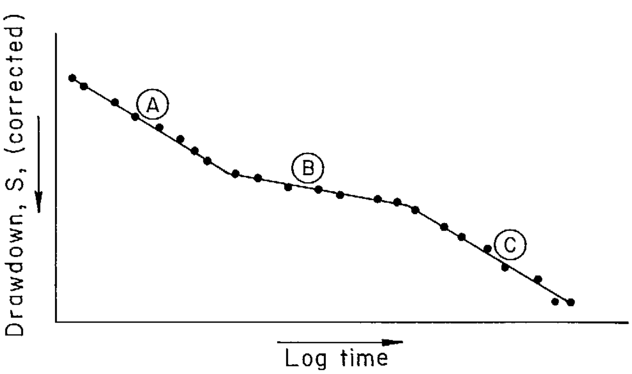


Fig D-3 - Typical drawdown results for unconfined well test.

17. The earliest component, (A), is due to the immediate response of the aquifer to release of pressure: at this stage the water pumped is obtained from relaxation of the aquifer before the start of gravity drainage.

18. The second component, (B), shows a decrease in slope because of replenishment by gravity drainage from the interstices above the depression cone.

19. The third component, (C), may start from several minutes to several days after pumping has begun. It is parallel to the first segment and starts when there is equilibrium between the gravity drainage and the rate of fall of the water table.

20. The straight-line plot shown on Fig D-4 represents the third segment of this drawdown plot for an observation well 30 metres from the pumping well. Fig D-5 shows the corresponding recovery curve. These results are used as examples of the method of calculating permeability.

PUMPING DRAWDOWN TESTS

21. The equation used in the modified non-equilibrium method may be written:

$$k = \frac{2.3Q}{4\pi m\Delta_s}$$

where k = coefficient of permeability
Q = constant pumping rate
m = thickness of saturated aquifer
Δ_s = corrected drawdown per log cycle of time (Fig D-4)

22. If Q is in litres/minute and m and Δ_s in metres then:

$$k = \frac{3.05Q}{m\Delta_s} \times 10^{-6} \text{ metres/sec.}$$

From the graph it is seen that the drawdown curve stabilized after about 15 minutes at a drawdown rate, Δ_s, of 0.53 metres/log cycle.

23. Using this figure, a pumping rate, Q, of, 473 litres/minute and a saturated aquifer thickness, m, of 2.44 metres, the permeability is given as:

$$k = \frac{3.05 \times 473}{2.44 \times 0.53} \times 10^{-6}$$

whence $k = 1.1 \times 10^{-3}$ metres/second

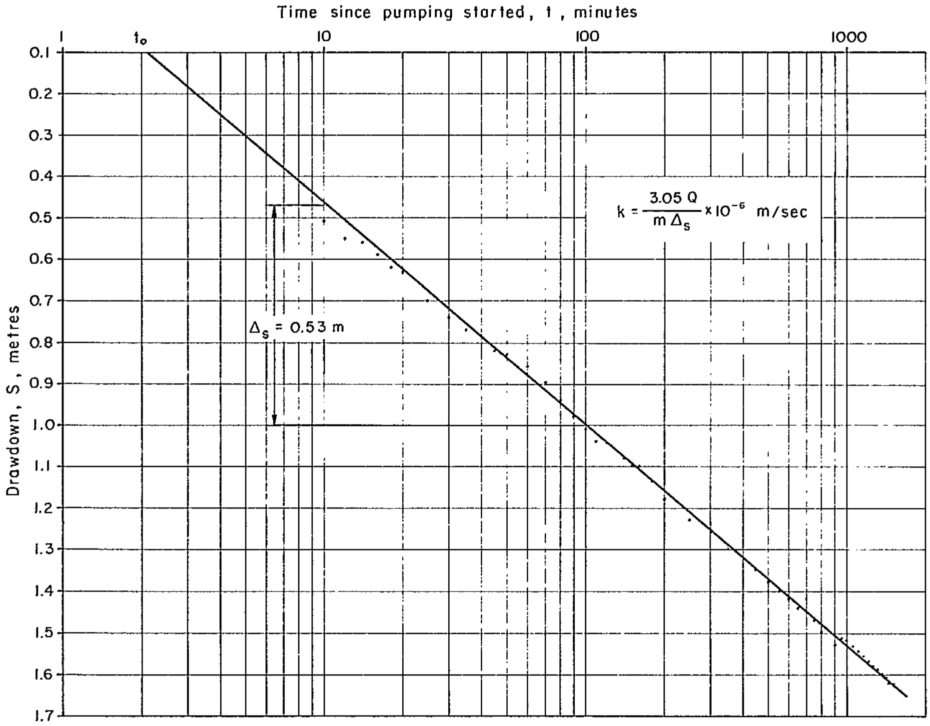


Fig D-4 - Time-drawdown curve for observation well.

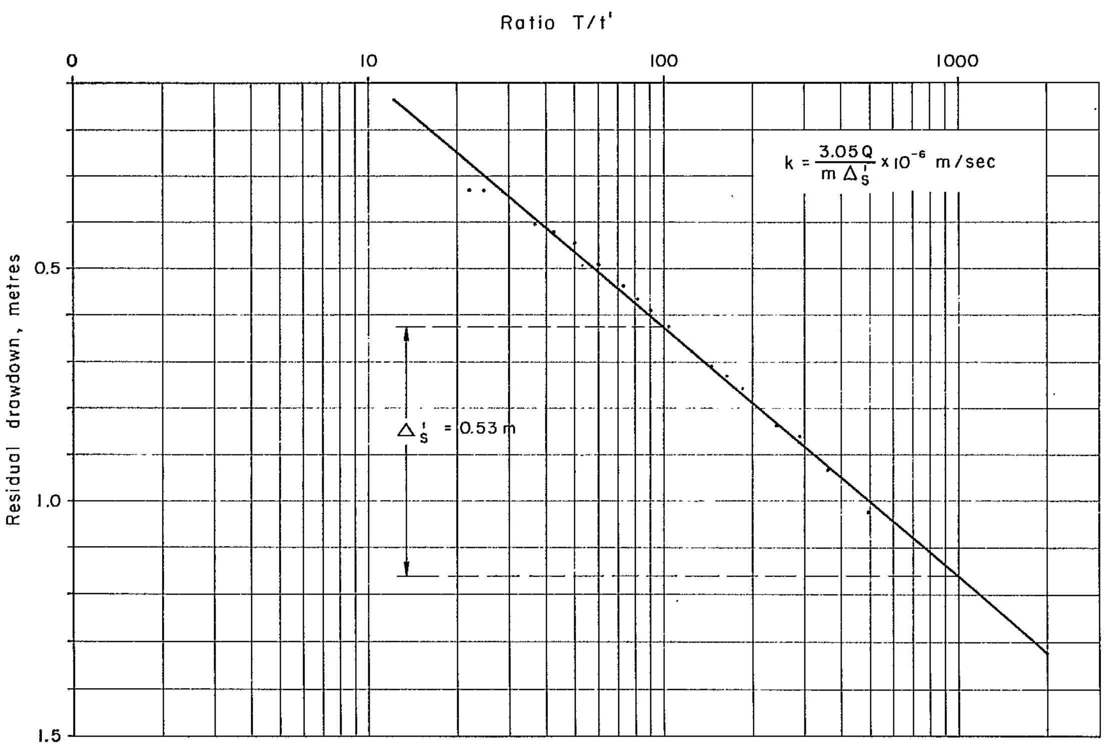


Fig D-5 - Recovery curve for pumping well.

To check that the data used for the best-fit straight line are not significantly affected by the middle segment of the drawdown curve, the time after which gravity drainage is in equilibrium with the fall of the water table must be determined (D-5). Only the portion of the data later than the time determined should be used in the permeability calculation.

RECOVERY TESTS

24. The equation used in the calculation may be written:

$$k = \frac{2.3Q}{4\pi m\Delta_s'}$$

where k = coefficient of permeability
Q = original constant pumping rate
m = thickness of saturated aquifer
Δ_s' = residual corrected drawdown per log cycle of T/t' (T = duration of pumping and t' = time at which recovery level is measured, Fig D-5)

If Q is in litres/minute and m and Δ_s' in metres then:

$$k = \frac{3.05Q}{m\Delta_s'} \times 10^{-6} \text{ metres/second}$$

The form of the above relationship is similar to that for the original pumping test.

DISTANCE DRAWDOWN METHOD

25. The coefficient of permeability may also be calculated from measurements of the drawdown at two or more observation wells at the same instant of time.

26. The procedure requires that the drawdowns be observed at the end of a particular pumping period in two or more wells at different distances from the pumping well (Fig D-6).

27. The coefficient of permeability is calculated from:

$$k = \frac{2.3Q}{2\pi m\Delta_s}$$

where k = coefficient of permeability
Q = constant pumping rate
m = saturated aquifer thickness
Δ_s = corrected drawdown per log cycle of distance from pumped well (see Fig D-6).

28. If Q is in litres/minute and m, Δ_s in metres, then:

$$k = \frac{6.1Q}{m\Delta_s} \times 10^{-6} \text{ metres/second}$$

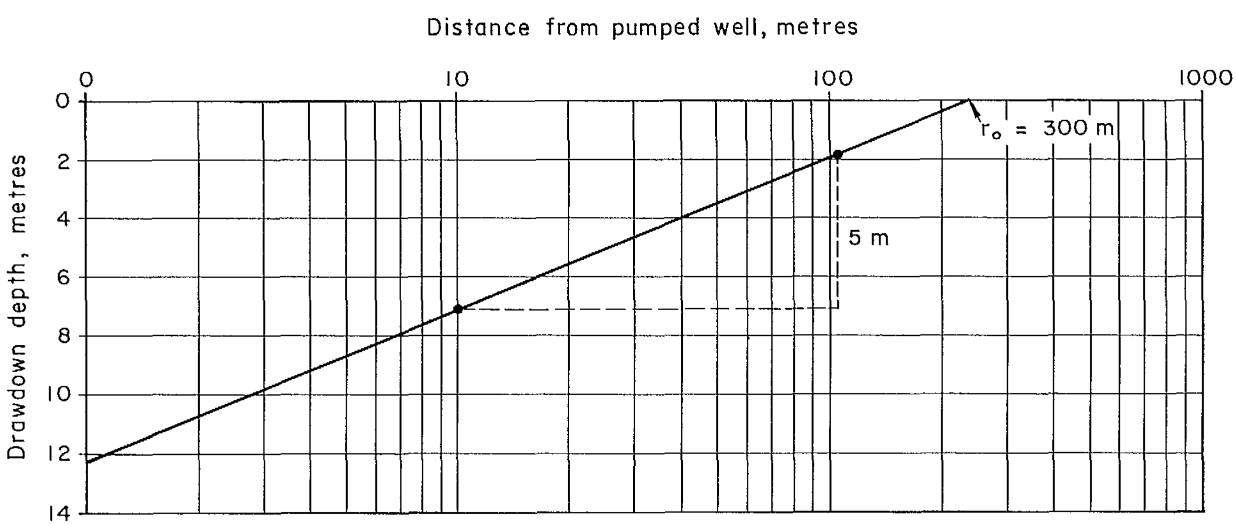


Fig D-6 - Example of semi logarithmic plot of drawdown vs distance.

The above calculations should be carried out for data recorded at all observation wells, ensuring that the appropriate portion of data as already described is used.

PARTIALLY PENETRATING WELL TESTS

29. Alternative methods of calculation apply to a partially penetrating well. This may be defined as one which extends through less than 80% of the full thickness of the aquifer.

30. If the penetration is more than 80% of the aquifer thickness, the effects of partial penetration are negligible, and the solutions for full penetration, already described, can be applied.

31. Similarly if data is being used from an observation well that is at a distance of twice the aquifer thickness or more from the pumped well, the effects can be ignored and the full penetration solutions used.

32. The permeability for the case of partial penetration of an unconfined aquifer is as follows (Fig D-7):

$$K = \frac{Q}{4\pi(h_1 - h_w)} \cdot \left(\frac{2}{h_s} \cdot \ln \left(\frac{\pi h_s}{2r_w} \right) + \frac{0.2}{H} \right)$$

where k = the coefficient of permeability

Q = constant pumping rate (steady-state condition)

h_1 = groundwater elevation at a distance from the well equal to twice the saturated thickness

h_w = the height of water in the well above the base of the aquifer

h_s = the depth of penetration of the aquifer

H = the initial height of the groundwater table above the base of the aquifer

r_w = the well radius

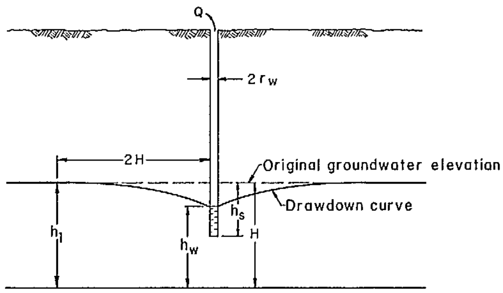


Fig D-7 - Partially penetrating well test.

A modified equation for the partial penetration of a confined aquifer, which is less common in mining, is given by Todd (D-1).

33. Working through the data from the previous example and assuming H = 100 metres,

$$Q = 473 \text{ litres/minute}$$
$$(h_1 - h_w) = 1.6 \text{ metres}$$
$$h_s = 2.44 \text{ metres}$$
$$2r_w = 0.2 \text{ metres}$$

$$k = \frac{473}{4\pi .1.6} \cdot \frac{2}{2.44} \cdot \ln \frac{2.44\pi}{0.2} + 0.002$$

$$= \frac{473}{4\ .1.6} \cdot 2.99$$

$$= 70.3 \text{ litres/m}^2\text{/min}$$
$$\text{or } 1.17 \times 10^{-3} \text{ metres/second}$$

34. This result corresponds closely to that obtained from the equation used for a fully penetrating well in an unconfined aquifer. At many mine locations, the partially penetrating case may be more applicable to local conditions. The methods described apply to steady-state conditions. Methods applicable to non-equilibrium conditions are given by Kruseman and De Ridder (D-5, pp 146-155).

REFERENCES

D-1. Todd, D.K. "Ground water hydrology"; John Wiley & Sons, Inc., 1959.

D-2. "Suggested methods for determining in situ permeability, groundwater pressure and flow"; Int. Soc. for Rock Mechanics, Committee on Field Tests (Draft report, 1974; Final report in course of preparation).

D-3. Theis, C.V. "The relation between the lowering of the piezometric surface and the rate of duration of discharge of a well using groundwater storage"; Am. Geophys. Union Trans.; v. 16, pt. 2; 1935.

D-4. Cooper, H.H., and Jacob, C.E. "A generalized graphical method for evaluating formation constants and summarizing well-field history"; Am. Geophys. Union Trans.; v. 27, no. 4; 1946.

D-5. Kruseman, G.P. and De Ridder, N.A. "Analysis and evaluation of pumping test data"; Int. Inst. for Land Reclamation and Improvement, Wageningen, Netherlands; 1970.

D-6. "Ground Water and Wells"; Johnson Division, sion, UOP Inc., St. Paul, Minn.; Fourth Printing, 1975. (This publication provides extensive practical guidance on well installation, testing, equipment selection and on methods of analyzing results).

APPENDIX E

THE USE OF ADITS FOR GROUNDWATER STUDIES

INTRODUCTION

1. Tunnelling is an expensive operation and can rarely be justified solely for the purpose of groundwater studies. During various stages of mine development, however, adits or tunnels to prove or sample ore and to assist general mining operations may be driven. If adit development is to be below the water table and sufficient time is available for detailed planning and preparation, such an operation provides an opportunity to undertake groundwater studies at marginally extra cost. The changes in the surrounding water tables related to adit progress can be recorded to:

a. provide information on the large scale behaviour of water within the rock mass,
b. estimate rock mass permeability, and
c. indicate likely response of the rock mass to drainage.

2. The advantages of adit studies are:

a. scale; the integrated behaviour of a volume of rock extending many hundreds of feet around the adit may be observed directly.
b. direct measurement; groundwater response is measured directly, whereas the use of analytical methods eg, finite element studies, (Supplement 4-1), involve both assumptions and predictions.

3. Effective methods of monitoring groundwater pressure changes are necessary and suitable preparations, often taking several months, are required. Groundwater tables and representative permeability values for the rock units in the area of the adit need to be known before and during excavation, which requires establishing a piezometer network and falling head (Appendix B) or constant head tests (Appendix C). Water flow into the tunnel must be recorded and some method of measurement has to be devised in conjunction with tunnel excavation. Hence, groundwater studies using this method will need close liason between the planning and operating departments. It may take from several months to one or two years to complete the study and to obtain all results.

EQUIPMENT

4. Equipment requirements will vary according to circumstances. The following details are supplied as guidelines:

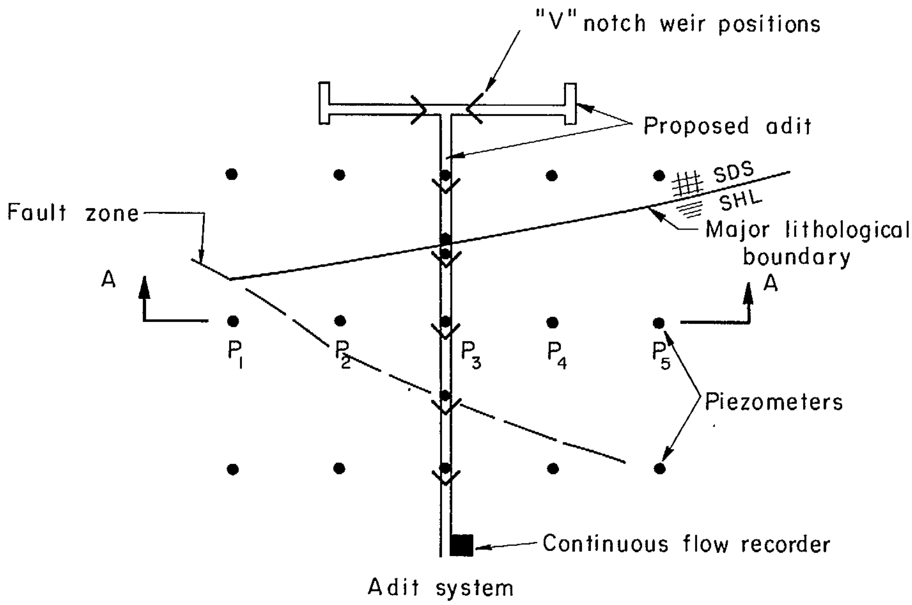
a. One or more drill rigs capable of drilling 2 to 4 in. (5 - 10 cm) diameter holes to the required depths. The number of rigs necessary will depend on the time available for the establishment of the piezometer network.
b. Equipment and materials to establish a piezometer network, as described in Appendix A.
c. A selection of 'V' notch weirs capable of measuring flows from 20 gpm to, say, 2000 gpm (1.5 - 150 l/s), depending on length of the proposed adit and on local ground and rainfall conditions. For example, flows from an exploration adit at a mine in New Guinea where rainfall was approximately 200 in. (500 cm) per year, peaked after heavy rainfall at approximately 2300 gpm (175 l/s). The adit was about 4000 ft (1200 m) long, approximately 8 ft by 6 ft (2.5 x 1.8 m) in cross section, and was driven through highly fractured and faulted andesites.
d. Rain and snow gauges to determine the precipitation at the site.
e. Plans of the proposed adit and the facility to update them as construction progresses. Details required will be topographical, geological, and geotechnical.

TEST CONFIGURATION

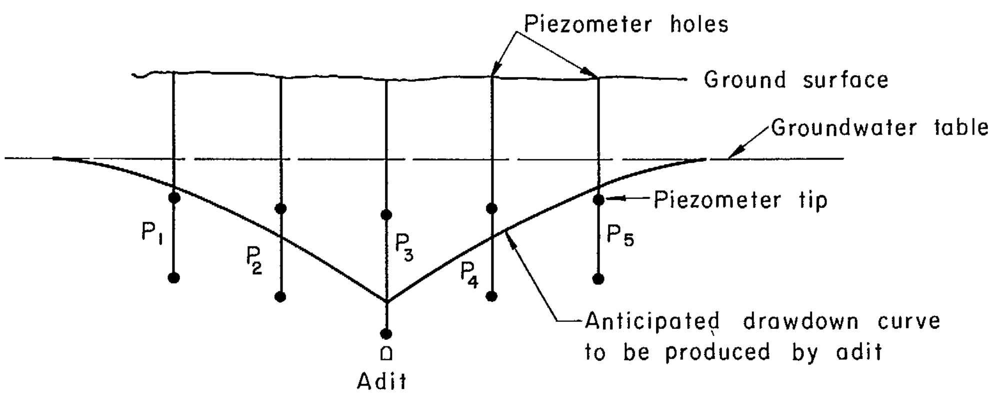
5. The principle to be followed is to establish a piezometer network which takes account of the structural and geological features of the rock mass. This is used to monitor the groundwater pressure changes in the area above and around the adit. A possible configuration is shown in Fig E-1.

6. The piezometric monitoring should commence as early as possible before the start of adit development to determine the undisturbed groundwater condition and the magnitude of seasonal variations. Monitoring should continue throughout and after the excavation phase, until conditions reach equilibrium.

7. Correctly installed standpipe piezometers are normally suitable unless the depths or permeability conditions dictate otherwise (Appendix A).



(a) PLAN VIEW



(b) SECTION A-A, SHOWING RELATIONSHIP BETWEEN PIEZOMETERS

Fig E-1 - Possible configuration for adit groundwater study.

8. The second important element is the monitoring of water inflow to the adit. Flow rates are usually measured at several sections by means of calibrated 'V' notch weirs, which can easily be instrumented to produce a continuous record.

9. Local rainfall details are necessary for interpretation of results, so the establishment of a rain gauge is required.

METHOD

10. The use of an adit for groundwater studies can be separated into three distinct stages:

a. planning and preparation,
b. data collection,
c. interpretation of results,

11. Table E-1 illustrates these stages and their relationships with the activities of other

Table E-1: Inter-relationship of tasks in an adit test

	GEOTECHNICAL GROUP	PLANNING GROUP	CONSTRUCTION GROUP	MAIN ACTIVITIES
Planning stage	1. Aquire or provide: — topographical maps, geological maps, seasonal rainfall data. 2. Design piezometer and V notch layouts	1. Design and schedule, construction details of adit. 2. Plan mining and metallurgical studies. 3. Incorporate piezometer and V notch layouts in design.	1. Install raingauge. 2. Commence piezometer installation.	Install raingauge Piezometer installation V notch weirs & sumps at face Monitor rainfall Monitor piezometers Monitor flows
Data collection stage	3. Commence collection of details of: daily rainfall, piezometer levels, adit flows, progress records of adit, geological/geotechnical log of adit.	GEOTECHNICAL GROUP 4. Commence interpretation of data.	3. Commence construction. 4. Complete piezometer installation. 5. Complete adit construction.	Construction of adit Interpret results and calculate permeabilities
Interpretation stage		5. Complete interpretation.		Time lag to equilibrium

groups normally involved in the adit development.

PLANNING STAGE

12. Information required to plan the layout of the piezometer network and installation sequence is:

a. topographical map,
b. geological map and cross sections, showing hydrogeological data,
c. details of the proposed adit - location, depth, construction, schedule,
d. details of seasonal variations in rainfall.

13. Liason will be necessary between the designers and construction schedulers for the following purposes:

a. to install piezometers before adit construction commences within their area of influence. A schedule is acceptable and sometimes necessary in which piezometers are still being installed during construction but in areas not yet affected by the excavation.
b. to measure water seeping from the face of the excavation. It will be necessary to agree on a method which is reliable under construction conditions. Weirs in side channels or drains which lead to sumps at the face often work well. These must be specified in the adit design. If a contractor is employed, provision for the construction of weirs and for access and measurement by mine staff should be made in the contract.

DATA COLLECTION

14. Data to be collected routinely are as follows:

a. rainfall; during the initial piezometric monitoring period and during the construction stage, rainfall will need to be measured daily.

b. piezometer pressures; these must initially be measured daily. Once information is available about rates of response, it may be possible to measure piezometers outside the influence of the excavation less frequently.

c. adit flows; flows are best measured at the portal by a continuous recorder. If this is not possible, flows measured manually each shift are acceptable. The disposition of notch weirs along the adit should be influenced by the geology and scale of the adit. There should be a 'V' notch on the downstream side of each cross section of piezometers and on the downstream side of any major lithological boundary or feature such as a fault or major dyke. The addition of electrical level detectors to each 'V' notch, connected to a multi-channel continuous recorder may be justified in large tests by savings in man-hours for the periodic reading of each 'V' notch.

d. progress records; the tunnel advance should be measured daily or per shift and up-to-date plans and records maintained. Geological/geotechnical logs of the adit should also be maintained, together with records of the location of visible differences in inflow.

15. All the above data should be plotted on a progress log, which should be kept up-to-date as construction proceeds.

INTERPRETATION OF RESULTS

16. The process of interpreting results will vary from test to test but, essentially, the data collected has to be assembled in a form which enables piezometric conditions to be related to flow rates at a given time, as well as to the geological conditions encountered.

17. As examples, Fig E-2 and E-3 illustrate piezometric behaviour and changes in flow rate that took place under certain geological conditions during an adit test.

18. By examining differences in the flow rate data between the portal, tunnel face, piezometer cross sections and the geological boundaries, the average flow rates originating in various sections of the tunnel at each stage of adit development can be determined. These data can be combined with the changes in groundwater levels monitored by the piezometer arrays and used for the determination of approximate rock mass permeability values, as described below.

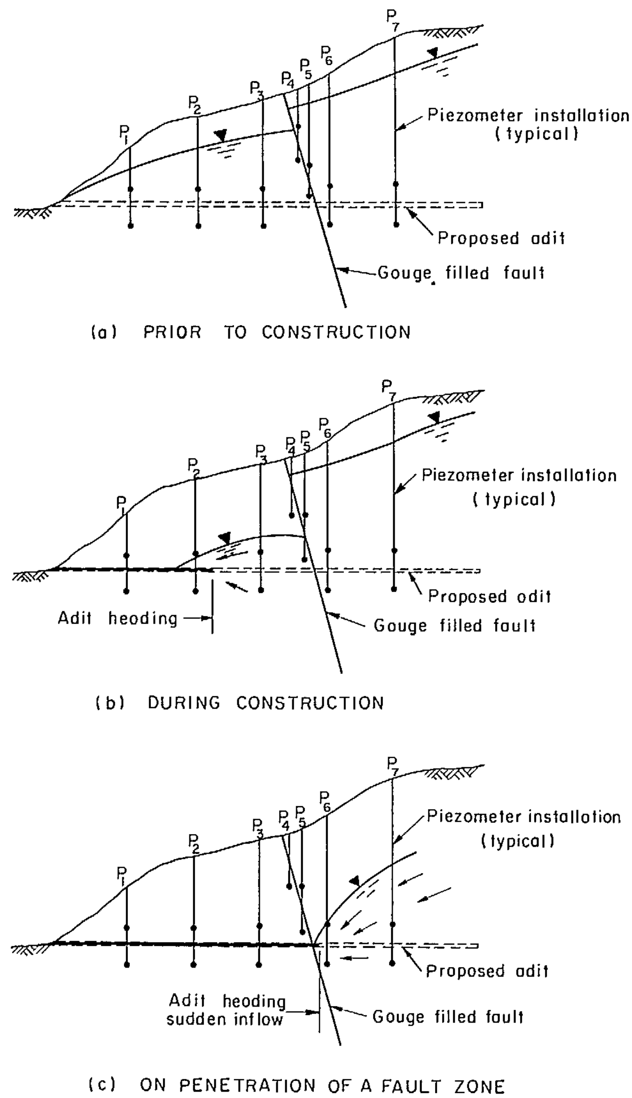


Fig E-2 - Piezometric results during adit construction.

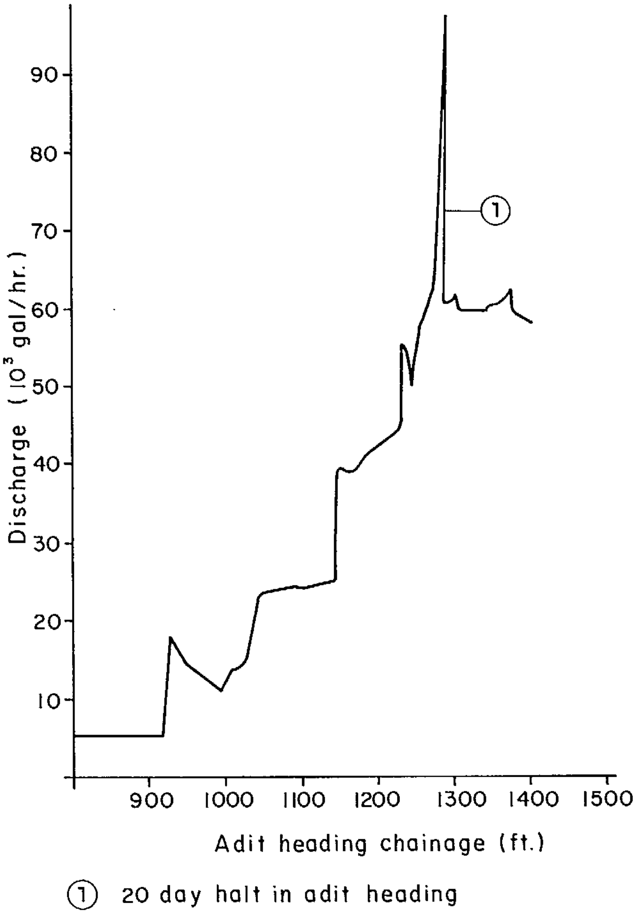


Fig E-3 - Discharge vs adit advance.

19. The relationship between precipitation and adit flows may provide information on near-surface permeability characteristics although these will generally be masked by changes in inflow resulting from tunnel advance during excavation. After completion of tunnelling and the establishment of equilibrium conditions, some indication of the transient response time of the rock mass can be gained. Where the geology is comparatively straightforward, comparison of flows from, and piezometric responses in, individual structural or lithological units may be possible: this enables comparative permeabilities to be judged, and features which have a significant effect on groundwater movement to be identified (see para 105-111 in main text).

20. The response to drainage, in terms of depression of the water table and the time taken to reach equilibrium, of rock units in the adit will indicate how successful large scale drainage measures would be in reducing groundwater pressures in pit slopes. It would also show any key structures or rock units that a drainage installation would need to intersect for maximum effectiveness.

DETERMINATION OF PERMEABILITY

21. There are four possible methods for the calculation of permeability. In increasing order of accuracy, they are:

a. approximate solutions based on drawdown,
b. flow net sketching,
c. electrical analogues,
d. numerical modelling methods.

22. If similar drawdown conditions are obtained along the length of the tunnel, the approximate mass or bulk permeability of the rock within the area of influence of the adit can be assessed by applying one of a number of standard drawdown solutions. An example of such a solution is shown in Fig E-4.

23. An approximation may be obtained by considering a two dimensional "slice" of the adit, orthogonal to its axis, and applying the formula obtained for flow to a partially penetrating slot in unconfined conditions (E-2), which is:

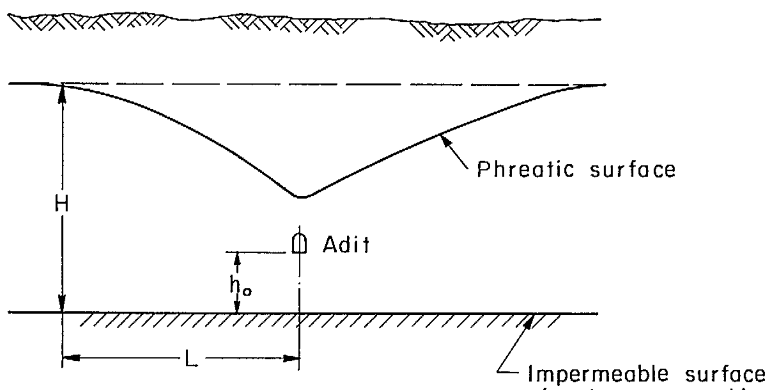
$$k = \frac{Q}{x} \cdot L \cdot \frac{1}{(0.73 + 0.27 \frac{H - h_o}{H})} \cdot \frac{1}{(H^2 - h_o^2)}$$

where k = permeability
Q = flow rate originating from the length of the adit being considered
x = length of adit being considered (ie, the "thickness" of the "slice")

and the remaining terms are as defined in Fig E-4.

24. Such methods are acceptable only for first order estimates of permeability. The accuracy of the result obtained is affected by the accuracy with which the flow originating in the section of the adit being considered can be measured in isolation: this in turn depends on the arrangement of 'V' notch weirs and piezometers.

25. The degree of uniformity of permeability



L is the distance from the centerline of the adit at which the original phreatic surface is undisturbed.

CROSS SECTION THROUGH ADIT (TWO DIMENSIONAL TREATMENT)

Fig E-4 - Approximate permeability assessment from adit test.

also affects the results. The formulae above relate to homogeneous masses. Zones of different permeability which intersect the adit more or less orthogonally may be dealt with separately by the two dimensional approach if sufficient monitoring points are available. Large scale permeability variations having the same trend as the adit prevent the development of normal drawdown and may give misleading results.

26. The use of flow nets, electrical analogues and numerical modelling methods are dealt with in Appendices F and G and in Supplement 4-1 respectively. Each can be applied to the interpretation of an adit test. Electrical analogues and numerical modelling can be used to study 3-dimensional inhomogeneous conditions. In each case, the procedure is to use the measured piezometer data to produce a flow net or groundwater pressure distribution. The measured adit flows relevant to the section being considered are used with the pressure distribution to determine the bulk permeability of the rock mass.

CONCLUSION

27. Determination of permeability by an adit test requires a great deal of preparation, data collection and processing of results. The long lead time before results are obtained must be considered and close liason with several groups involved in the project is necessary. Costs in the order of hundreds of thousands of dollars would normally be expected with this test. Such costs must, however, be related to the cost effectiveness and reliability of groundwater characteristics determined by other methods.

28. The primary advantage of this test is the provision of reliable data at a scale representative of large-scale open pit slopes.

REFERENCES

E-1. Harper, T.R. "The transient groundwater pressure response to rainfall and the prediction of rock slope instability"; Int. Journal of Rock Mechanics and Mining Sciences; v. 12, no. 5 and 6; June 1975.

E-2. Mansur, C.I. and Kaufman, R.I. "Dewatering in Foundation Engineering"; ed. G.A. Leonards; McGraw-Hill Book Co.; 1962.

APPENDIX F

CONSTRUCTION AND USE OF FLOW NETS

INTRODUCTION

1. Flow net sketching is a convenient analytical method for determining the discharge rates and groundwater pressures in homogeneous, isotropic soil and rock masses. Simple permeability variations due to changes in lithology can be dealt with. Flow nets are of great value in providing the engineer with a "feel" for the pattern of behaviour of groundwater and its response and sensitivity to external changes.

GENERAL PRINCIPLES

2. The flow of water through a soil or rock mass depends on differences of total head or pressure existing within the area being considered. In simple terms, a flow net is a pattern of curvilinear squares, formed by flow lines and equipotentials, which express graphically how flow flow takes place. A flow line describes the path followed by a molecule of water when flow is taking place. Equipotential lines, which occur at right angles to flow lines in isotropic materials, join points of equal hydraulic potential, ie, they are contours of hydraulic potential. Flow nets are therefore grids of intersecting flow lines and equipotential lines, as shown in Fig F-1.

3. Flow net construction is described below in terms of a vertical section of a pit slope, which is an appropriate form for stability analysis. Transformations to evaluate axisymmetric (two-dimensional) conditions for curved slopes can be made.

4. Flow nets can also be constructed with equal facility in a horizontal sense using contours of water table elevation ie, equipotential lines, as the basis. In this form, they are useful in feasibility studies to indicate regional groundwater movement, large-scale permeability variations and likely inflows into the pit from different directions.

USE OF FLOW NETS

5. Flow nets can be used for the following purposes:

a. to calculate groundwater pressures for slope stability analysis,
b. to estimate the volume of seepage,
c. to calculate the hydraulic gradient.

FLOW NET CONSTRUCTION

6. For the simplest case, it is assumed that the soil or rock conditions are homogeneous and isotropic, and the geometry of the body through which groundwater flow takes place has been defined.

BOUNDARY CONDITIONS

7. The first stage is to define the boundary conditions. These are the limits to which flow occurs and are represented by the phreatic surface and the physical boundaries of the area being considered, such as an impermeable base or cut-off

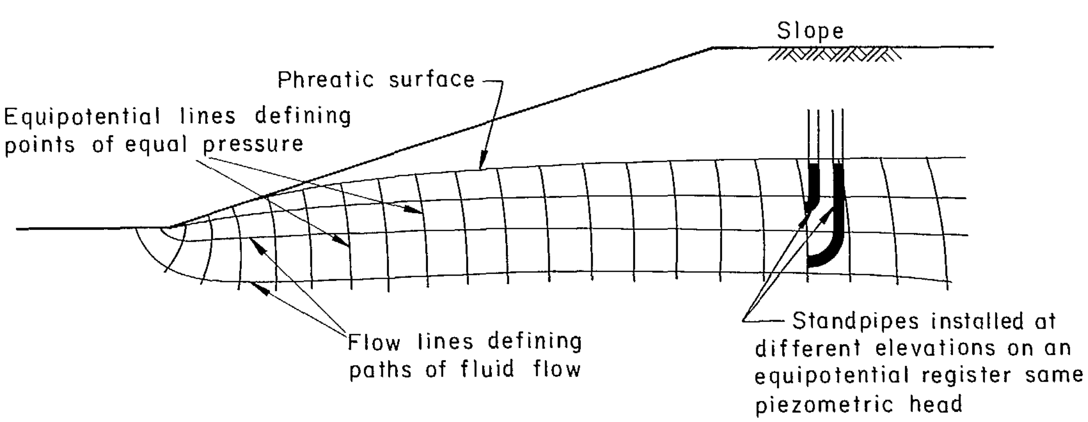


Fig F-1 - Flow lines and equipotentials.

zone, the slope surface and the upstream boundary. For a typical slope, these conditions are illustrated in Fig F-2.

8. The position of the phreatic surface is of particular importance. It is normally estimated from borehole or piezometric measurements, evidence of face seepage and other field observations.

DRAWING FLOW LINES

9. These are constructed starting at the upstream boundary. The number of flow paths, N_f, is normally set at between 3 and 5. An equal width between flow lines at the upstream boundary is chosen. The flow lines are drawn to be smoothly converging towards the toe, which is a focus for flow, and because the pit floor beyond the toe will be an equipotential line, the flow lines should exit at right angles to it as shown in Fig F-3.

PLOTTING EQUIPOTENTIAL LINES

10. The phreatic surface is now divided into a series of equal head drops starting at the upstream boundary, as follows (Fig F-4):

a. the distance between the phreatic surface and the flow line immediately below is measured at the upstream boundary, (a)
b. the first equipotential line is drawn from the phreatic surface at the same distance, (b), from the upstream boundary. This forms the first square in the flow net grid.

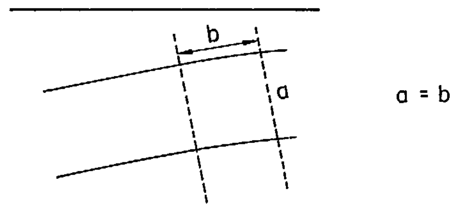


Fig F-4 - Constructing equipotential-flow line "squares".

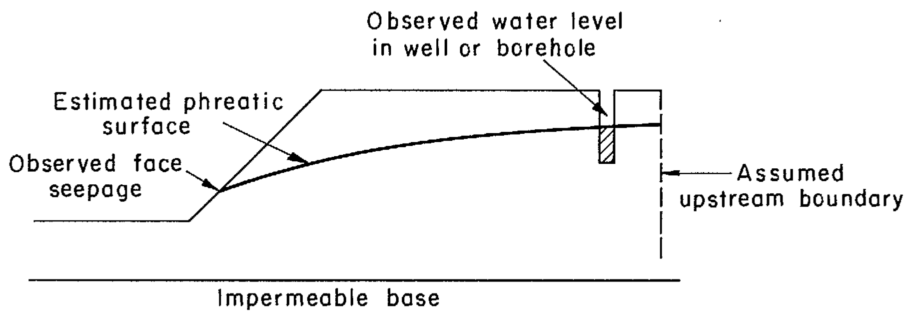


Fig F-2 - Boundary conditions.

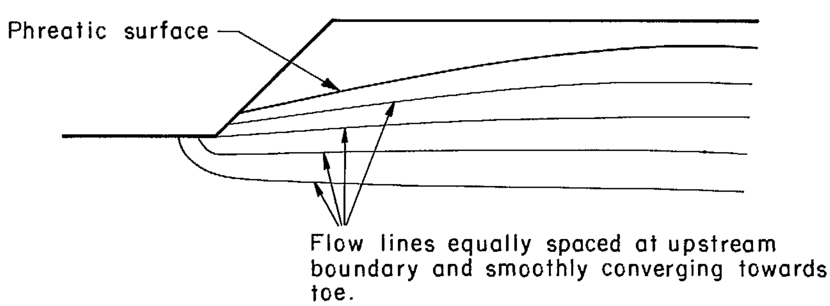


Fig F-3 - Constructing flow lines.

c. the head drop along the phreatic surface for that square is measured, and equal head drops are then measured out on the phreatic surface down to toe level.
d. the intersection points of these head drops with the phreatic surface form the starting points for all other equipotential lines. They are drawn to form curvilinear squares with the flow lines, as shown in Fig F-5.

11. It will generally be necessary to make several successive modifications to the construction of the flow net to end up with a well ordered net. A common problem is caused by making the phreatic surface horizontal at the upstream boundary. For flow to occur across this boundary to the slope face, the surface must slope in the direction of flow.

12. The first attempt will frequently be relatively crude and the squares large and few in number. A completed flow net is shown in Fig F-6.

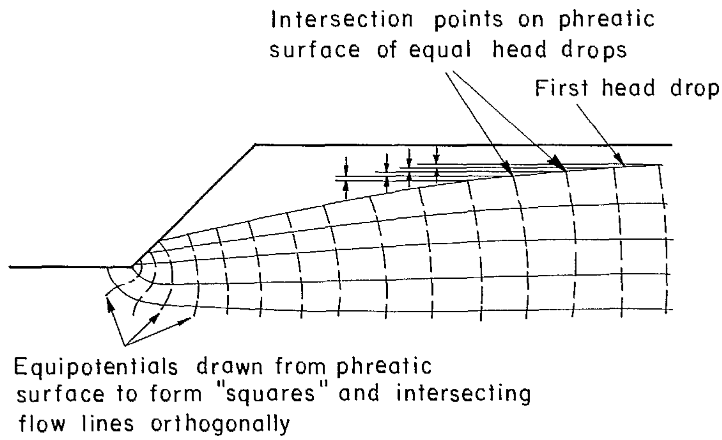


Fig F-5 - Completing flow net.

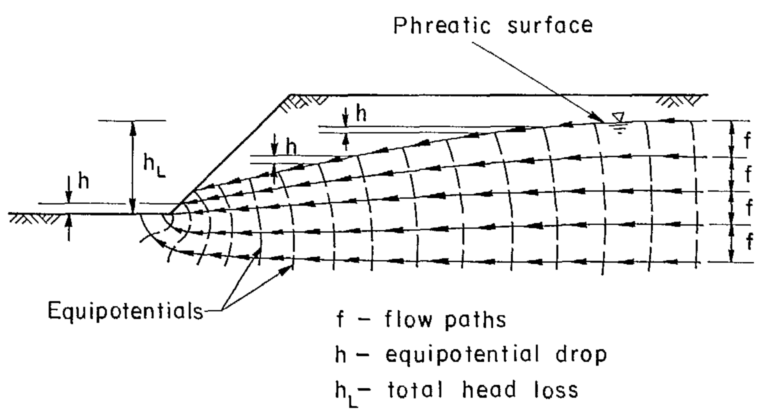


Fig F-6 - Completed flow net.

CALCULATIONS

Groundwater Pressure

13. In estimating the groundwater pressure at a point on a potential slip surface for stability analysis, the effect of flow on the pressure distribution must be borne in mind.

14. In Fig F-7, the pressure head, p, at point P on the potential slip circle is as shown. The pressure is obtained from where the equipotential line meets the phreatic surface and not from the level of the phreatic surface vertically above the point being considered.

Flow Rate

15. To estimate seepage volume, the values of parameters obtained from the flow net are substituted in the following equation.

$$\text{Rate of flow, } q \text{ (volume/time)} = k \cdot h_L \cdot \frac{N_f}{N_e}$$

Where N_f = number of flow paths (Fig F-6)
N_e = number of equipotential drops
k = coefficient of permeability
h_L = total head difference or head loss (Fig F-6)

Note: For anisotropic permeability cases, k is replaced by the effective permeability, $\overline{k} = \sqrt{(k_h k_v)}$; this is described below.

Hydraulic Gradient

16. The value of the hydraulic gradient for any square or group of squares in the flow net is equal to the total head loss in the square or group of squares divided by the length of the flow path (ie, by the length of the squares).

COMPLEX PROBLEMS

17. For more complex problems involving two or more materials of different permeability or a material with anisotropic permeability, techniques can be applied to consider the differences in flow rate and flow direction that will take place. The solution can easily become complex and proficiency in compiling simple flow nets is necessary before other problems are attempted.

Anisotropic Permeability

18. The drawing of the cross section of the slope must be transformed prior to drawing the flow net, using the square root of the permeability ratio.
For example, if $k_h = 3k_v$

where k_h is the horizontal permeability, and
k_v is the vertical permeability

then all horizontal dimensions on the cross section would be multiplied by

$$\sqrt{(k_v/k_h)} \qquad \text{or} \qquad \frac{1}{\sqrt{3}}$$

19. The flow net is constructed on the transformed section, as described above. When completed, the cross section and flow net are re-plotted

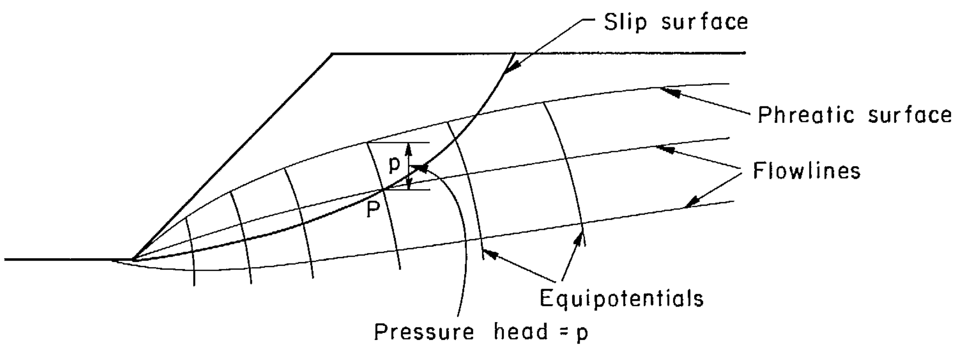


Fig F-7 - Determining pressure from flow net.

to the correct scale. These three steps are illustrated in Fig F-8.

20. When drawing flow nets of anisotropic rocks and soils, the following facts should be considered:

a. The cross section must be transformed before the flow net is constructed.
b. If the horizontal permeability is greater than the vertical, the transformed section will always be shrunk to a narrower horizontal dimension. If the reverse is the case, it should be lengthened horizontally, ie, the cross section is always contracted in the direction of greater permeability.
c. On the redrawn natural cross section, the flow net will not be composed of squares but of rectangles elongated in the direction of greater permeability. Flow lines and equipotentials will not cross at right angles.
d. Hydraulic gradients for anisotropic conditions must be determined from flow nets after redrawing at the natural horizontal scale, because the distance over which a given amount of head is lost can only be measured on a true section.
e. Seepage quantitites can be calculated by counting the number of flow channels and equipotentials in the normal manner, but using an effective permeability, $\overline{k} = \sqrt{(k_h k_v)}$ in the equation in para 15.

Materials of Different Permeability

21. When considering materials of different permeabilities, it is necessary to consider the transfer conditions between each rock or soil type. When water flows across a boundary between dissimilar soils the flow lines are refracted in a similar manner to light rays. When water is flowing from a material of high permeability into a material of lower permeability, flow takes place

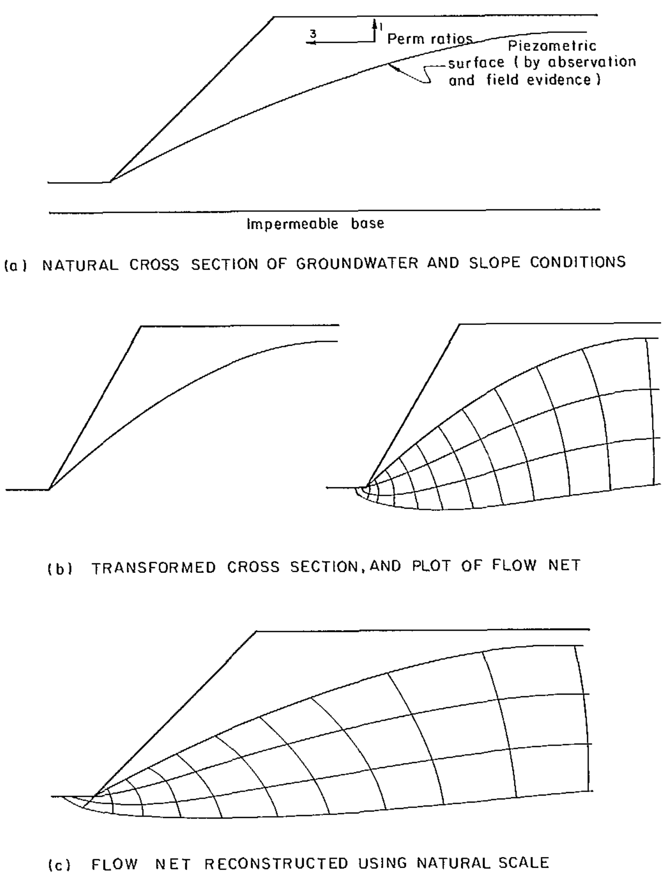


Fig F-8 - Transforming sections for anisotropic permeability.

within the material of higher permeability for the greatest possible distance. Conversely, if water is flowing from a material of low permeability into one of higher permeability, the flow deflects as soon as possible into the higher permeability material. Flow lines bend to conform to the relationship

$$k_1/k_2 = \tan \beta/\tan \alpha$$

where α and β are as shown in Fig F-9.

22. When water flows from a rock or soil of low permeability into one of higher permeability, the distance between adjacent equipotential lines is increased, because lower pressure gradients are required to maintain the flow. Conversely, if flow takes place from high to lower permeability, the reverse is true (Fig F-9).

23. On the downstream side of the boundary, the squares of the flow net either elongate or shorten, depending upon the ratio of the permeabilities, according to the relationship (Fig F-9)

$$c/d = k_2/k_1$$

24. In plotting flow nets which cross a boundary between different permeabilities, the following procedure defines the altered pattern:

a. Knowing the permeabilities k_1, k_2 and the angle of incidence, α, calculate the refracted angle, β, from the relationship

$$\tan \beta = \frac{k_1}{k_2} \cdot \tan \alpha$$

b. Plotting the refracted flow lines at the correct angle will determine the distance, d, between them.

c. The distance between equipotentials is then given by

$$c = \frac{k_2}{k_1} \cdot d$$

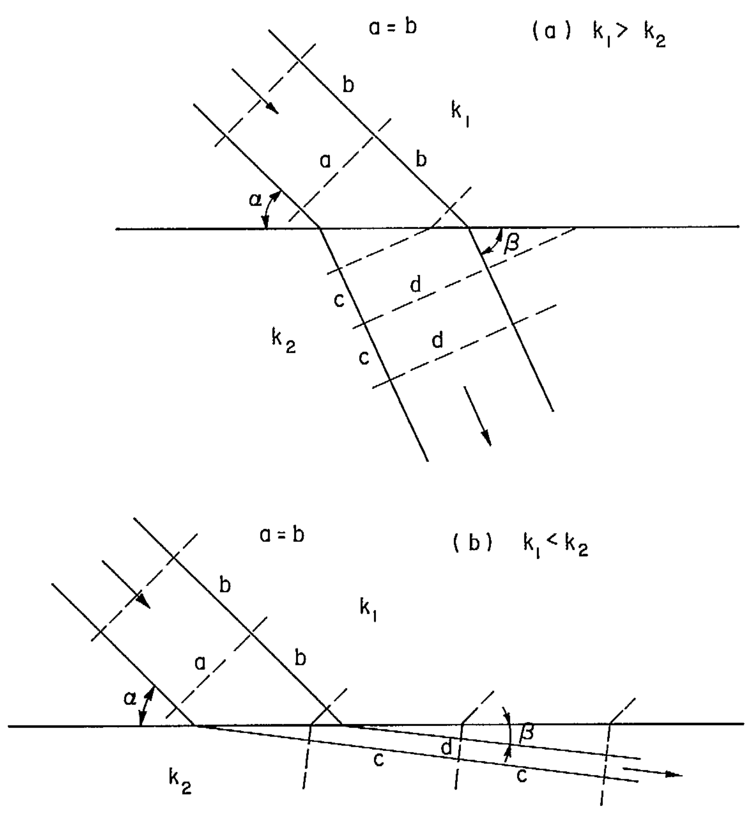


Fig F-9 - Refraction of flow lines at material boundaries.

25. When plotting flow nets of this type, it is necessary to check regularly that the above principles are being conformed with, by measuring the dimensions and angles of the component squares of the flow net. A typical flow net constructed using this approach is shown in Fig F-10.

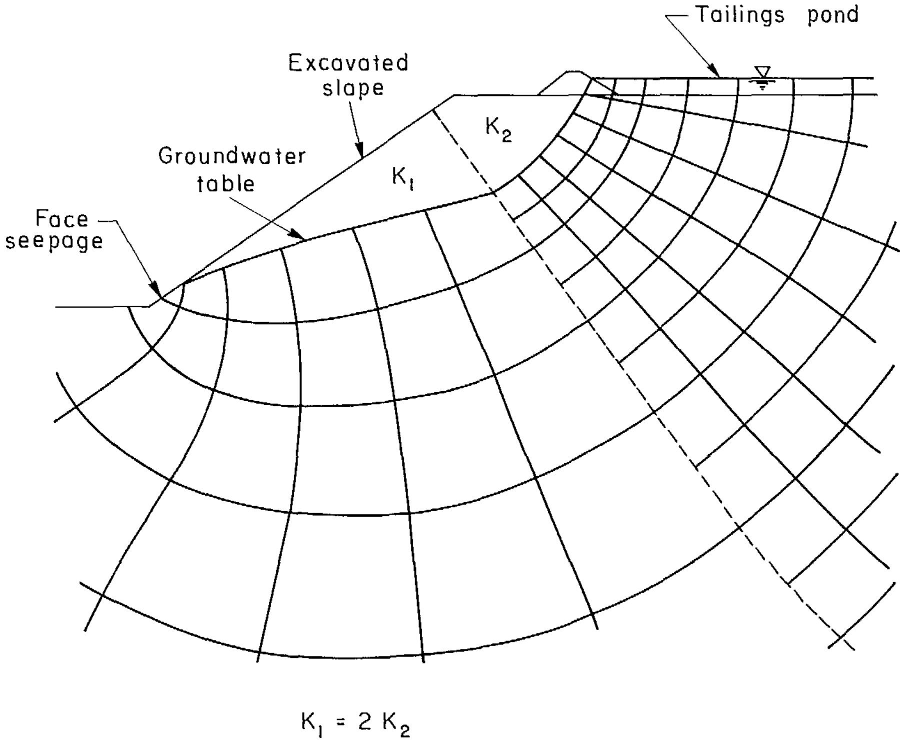


Fig F-10 - Completed flow net for slope with different materials.

REFERENCES

F-1. Cedergren, H.R. "Seepage, drainage and flow nets"; John Wiley & Sons, Inc., 1965; (An extensive description of the construction of flow nets and their application to filter design and seepage control in dams, foundations, slopes and structures. Chapter 7 describes transformation of axisymmetric flow nets for estimating flow rates into excavations).

F-2. Lambe, T.W. and Whitman, R.V. "Soil mechanics"; John Wiley and Sons Inc., 1969 (Chapter 18 contains useful examples of flow net construction for cut-offs, earth and gravity dams).

F-3. Smith, G.N. "Elements of soil mechanics for civil and mining engineers"; Crosby Lockwood and Son Ltd., London, U.K., 1965; (Chapter 2 includes a useful summary of Casagrande's parabolic construction for seepage through an earth dam).

APPENDIX G

ELECTRICAL RESISTANCE ANALOGUES

INTRODUCTION

1. Electrical resistance analogues are based on the similar mathematical forms of Ohm's law for the flow of electricity and Darcy's law for the flow of water through a porous medium. The analogy is:

Electricity	Fluid
Current, I	Flow per unit area Q/A
Potential difference, V	Hydraulic gradient, i
Conductance, 1/R	Permeability, k
Ohm's law, I = V(1/R)	Darcy's law, Q/A = ik

The analogue is set up so that the equations of electricity flow correspond to the equations of water flow for the slope being considered. Electrical measurements corresponding to the parameters of water flow are then made.

2. This appendix describes two types of electrical resistance analogue - conducting paper and conducting ink. Other types (G-1) include:

a. The discrete resistance analogue, in the form of a network of variable resistances arranged in a rectangular mesh. The resistances can be varied to model various permeabilities. Known voltages can be applied to any points of the network to represent boundary conditions and equipotentials obtained by inspection of the potentials at nodal points. They may be used to study flow in anisotropic rock masses. (G-2)

b. The conductive liquid analogue, in which an electrolyte such as water is used. This method can examine three-dimensional seepage, or anisotropy in two dimensions. In the latter case, zones of different permeability may be represented by varying the depth of the fluid between conducting barriers, or having fluids of different conductivities between conducting barriers.

c. Other conductive sheet analogues, which are used in a precisely similar manner to the conducting paper analogue described below. The sheet material may be woven metal wire, conductive rubber, metallized paper or thin metal strips. The latter is useful for anisotropic studies in which different thicknesses of sheet can be used for different permeabilities.

3. Types (a) and (b) are relatively costly and time-consuming to construct compared with the conducting paper or conducting ink types, in which a physical model of the slope being studied is constructed from conducting paper or drawn in conducting ink, and the positions of lines of equal electrical potential are determined by inspection, after applying voltages to the boundaries of the model. Current flows through the paper from the upstream boundary strip at 100% potential to the lower boundary strip at zero potential.

4. These techniques, together with flow nets, (Appendix F), to which they are complementary, are alternatives to numerical techniques (Supplement 4-1). They are less flexible than numerical techniques and, except for complex electrical resistance analogues (G-2), can be used only for relatively straightforward studies. The great advantages of paper or ink techniques are low cost, simplicity and the fact that, once the model has been constructed, the operator has close contact and direct control as the solution of the problem proceeds.

EQUIPMENT

5. Materials required to produce an electrical analogue are:

a. drawing board,
b. conducting paper or ink (Teledeltos paper or Aquadag Colloidal Graphite),
c. heavy brass strips and conducting paper,
d. scale drawing of the study zone,
e. high impedance voltmeter - it is most important that the voltmeter draws negligible current to avoid affecting the potential distribution,
f. probe, with a point sufficiently sharp to measure spot values accurately without piercing the paper,
g. potentiometer (variable resistor),
h. 12 volt electrical supply (battery or transformer/rectifier).

6. Teledeltos paper is manufactured by the

Western Union Telegraph Company.

TEST PROCEDURE: CONDUCTING PAPER

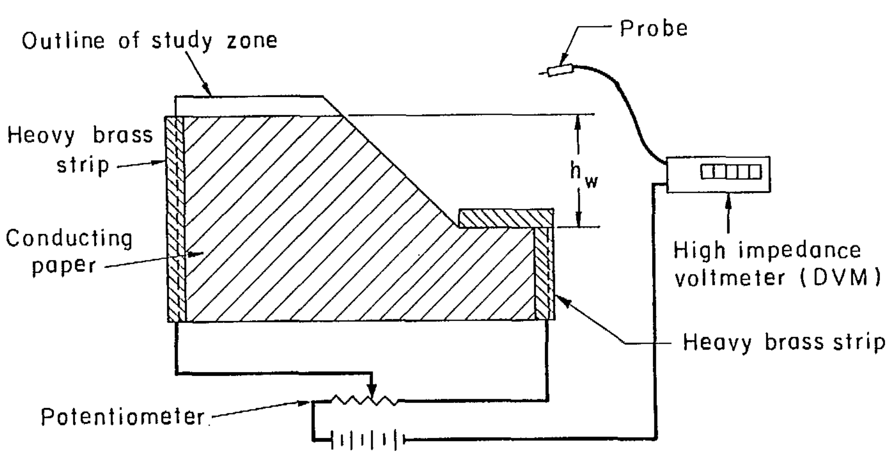
7. The test procedure for the conducting paper method is as follows:

a. Draw a cross section of the slope being studied to equal horizontal and vertical scales on Teledeltos paper. Select the scale to suit the size of the drawing board. Cut out the cross section which must extend from inside the toe of the slope to a point at a distance back from the crest where the phreatic surface is undisturbed, ie, at the point on the phreatic surface where drawdown commences. This point will have been determined from piezometer observations in the field.

b. Place the cross section on the drawing board, and secure it by laying brass strips at the boundaries. These define the boundary equipotentials, ie, zones where the hydraulic potentials are equal at all points - 100% and 0% at the upstream and downstream boundaries respectively). Place the upstream brass strip at the height of the undisturbed phreatic surface. For boundaries of irregular shape, conducting paint (eg, Dupont No. 4817 silver paint) may be used.

c. Connect the positive terminal of the power source, through the potentiometer, to the upstream boundary of the model. Connect the negative terminal to the downstream boundary and, through the voltmeter, to the probe. By adjusting the potentiometer, apply the minimum selected input voltage, V, which gives sufficient sensitivity in the voltmeter to enable the potential to be measured accurately. The set-up at this stage is shown in Fig G-1.

d. Determine the phreatic surface. At the phreatic surface, the water pressure is zero, or the hydraulic potential expressed as a head is equal to the elevation. At any given elevation, say 0.6 h_w, the voltage within the true flow zone must be equal to or greater than 0.6 times the voltage used for the upstream boundary of the system, V. Measure the voltage V_1 along this horizontal line using the probe. It will be found that voltages decrease from V on the upstream boundary, through a point where V_1 = 0.6 V, to some lower value at the slope outline. Wherever the value is less than the value corresponding to the elevation potential, ie, 0.6 V, this represents an unreal flow condition and is termed the "no flow" zone. The point where V_1 = 0.6 V lies on the lower



h_w is the height of the undisturbed phreatic surface

Fig G-1 - Conducting paper analogue at beginning of simulation.

boundary of the no flow zone, ie, at the phreatic surface.

e. Delineate the boundary of the no flow zone at several elevations, say, 0.8 h_w, 0.4 h_w and 0.2 h_w, and cut it off the cross section. The true phreatic surface must be determined by trimming off the no flow zone in a number of stages. Each time paper is removed, the voltage distribution changes and the procedure must be repeated. It is recommended that only part of the no flow zone delineated be removed each time and that several progressive steps be taken to produce the phreatic surface, as illustrated in Fig G-2.

f. Divide the applied voltage into a suitable number of intervals - 10 is generally satisfactory for most problems, ie, 0.1 V, 0.2 V, 0.3 V, etc. Using the probe, determine by inspection a number of points within the cross section where the potential is 0.9 V. Join these points to determine the trace of one equipotential. Repeat for the other values (Fig G-3).

g. When the plotting of equipotentials is complete, plot the flow lines by hand, conforming to the principles of producing curvilinear squares with flow lines intersecting equipotential lines orthogonally (Appendix F). This stage of the method is subjective but, because the pressure distribution is already defined, the operator is able to construct flow lines with a fair degree of confidence. In any case, for stability assessment the prediction of pressure is of primary importance.

8. <u>Anisotropic permeability</u> in vertical and horizontal directions can be modelled by cutting out a transformed cross section of the slope, determining the equipotentials as described above,

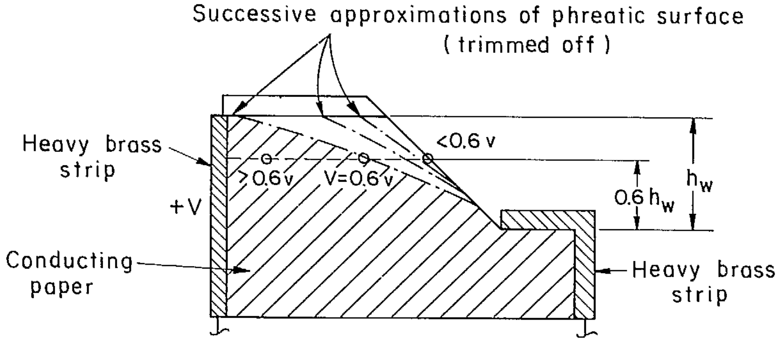


Fig G-2 - Conducting paper analogue during simulation.

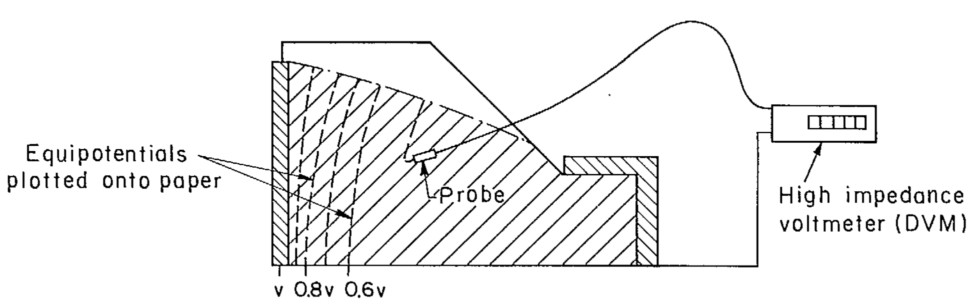


Fig G-3 - Plotting equipotentials on conducting paper analogue.

and replotting the flow net to the natural scale. The method of transformation is described fully in Appendix F.

TEST PROCEDURE: CONDUCTING INK

9. The equipment required and the procedure for determining the phreatic surface and equipotentials is as described above. The cross section in this case is cut out from paper which has lines ruled in conducting ink containing a colloidal solution of graphite or finely divided metal. The outline of the cross section must not be drawn in conducting ink. By using different grid patterns, materials of different permeabilities or a material with anisotropic permeability can be represented, as shown in Fig G-4.

EXAMPLES OF USE

10. In addition to determining the pressure distribution for a simple slope, as described above, the method may be used, for example, to evaluate the effect of drainage or to determine the flow pattern towards an adit (Fig G-5, also Appendix E).

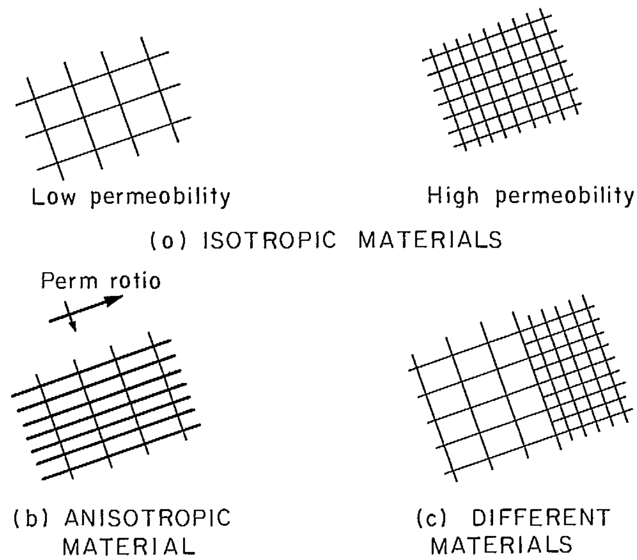


Fig G-4 - Use of conducting ink to model permeability characteristics.

CALCULATION

11. Once the drawing of the flow net has been completed, calculations for quantity of flow etc, are carried out as described in Appendix F.

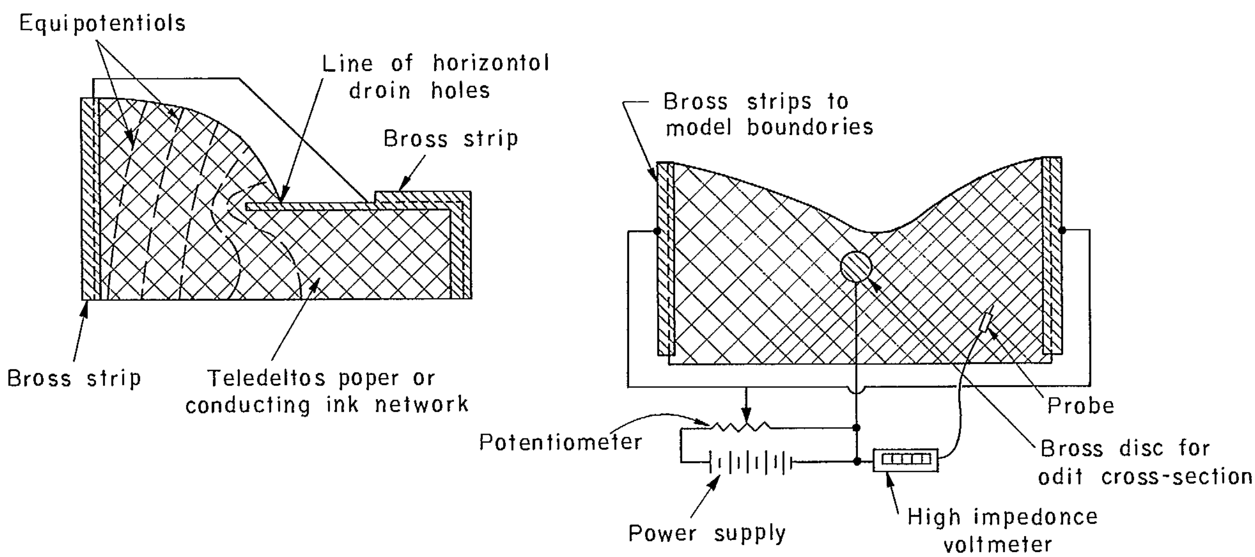


Fig G-5 - Analogues for drain holes (left) and an adit.

REFERENCES

G-1. Karplus, W.J. "Analog simulation"; McGraw - Hill Book Co., New York, 427 pp; 1958; (Extensive reference text to analogues of conducting paper and other types.)

G-2. Sharp, J.C. "Fluid flow through fissured media"; Ph.D. Thesis, University of London (Imperial College); 1970.

APPENDIX H

REGIONAL GROUNDWATER EVALUATION

SCOPE

1. A study of the regional hydrology and and hydrogeology should be made during the feasibility stage of a groundwater evaluation program with the aim of defining at least qualitatively the following:

- the distribution of rock and soil types, and their comparative hydrological properties,
- the regional geological structure and its affect on groundwater movement,
- the presence and distribution of aquifers and aquicludes,
- the influence of surface topography on drainage patterns, zones of discharge and recharge and the movement of surface water.

2. The purpose of the study is to build up a series of plans and cross sections, collectively called a regional groundwater map. Substantial progress can be made towards its preparation in a desk study by using available data; at later stages, some fieldwork is necessary.

PURPOSE

3. The regional groundwater map can be used to predict:

a. the effect of the regional groundwater regime on the proposed mine, including:
  - surface water run-off problems which may arise around the mine, particularly if it is in a basin;
  - the effect of any large scale surface water features, eg, lakes or rivers close enough to recharge the mining zone directly;
  - the likely relative permeabilities of the main rock types, eg, low permeability shales and slates, high permeability sandstones and limestones, etc;
  - the influence of overburden deposits and degree of weathering of the rock on groundwater behaviour;
  - the effect of major structural features, eg, faults and shatter zones, on the continuity of groundwater movement;
  - which rock units are likely to be aquifers, and whether the geological structure is such that they will form sources of recharge to the excavation;
  - whether the combination of geological structure, lithology and topography is likely to result in artesian conditions within the mining zone;
  - the constraints imposed on the siting of stability dumps and tailings dams by surface water systems or areas of discharge;
  - the availability of surface or groundwater sources for development as service water to the mine, mill, township etc.

b. the effect of the proposed mine on the groundwater system, including:
  - precautions necessary against pollution of surface and groundwater systems by surface run-off from dumps and mine or mill effluents;
  - possible interference with existing domestic water supplies due to mine dewatering or recharge of groundwater by polluted effluents;
  - long-term effects of prolonged mine dewatering, surface water course diversions to permit mining or tipping and new sources of recharge caused by tailings dams, etc.

EVALUATION TECHNIQUES

4. Several useful techniques in building up the regional groundwater map are described below. The interpretation of groundwater information often requires an experienced hydro-geologist, preferably with some knowledge of the region under investigation. Similarly, experience and skill is important in interpreting aerial photographs. It is therefore not possible to give definitive instructions for the use of such techniques.

GEOLOGICAL DATA

5. Available geological maps can be obtained from the Geological Survey of Canada, Ottawa. The regional structure and the definition of the major rock types present are of prime significance in determining regional groundwater behaviour.

6. Any existing reports and information resulting from previous geological and hydrogeological research, from mineral exploration activities and from producing mines should be obtained. If the area has been explored for water supply, a qualitative assessment of groundwater conditions

can often be made.

7. Transgressive features, such as faults, sills and dykes, and regional jointing should be noted from the geological maps. Fault patterns are often significant in forming either barriers or preferential flowpaths to groundwater movement. It may be possible to identify potential aquifers from these maps: sands and gravels are usually permeable: sandstones are often sufficiently friable or jointed to be permeable; igneous intrusions are relatively impervious. The structures ie, folds and dips, indicate whether potential aquifers will be present at depth.

8. It is often necessary to re-map the geology according to lithology and permeability variations. For example, clayey alluvia should be distinguished from sandy/gravelly alluvia; well bedded sandstones will have a predominantly horizontal permeability while massive sandstones may have a dominantly vertical permeability through fissures; marly limestone can give the highest yields of all rocks; weathered igneous and metamorphic rocks should be distinguished from fresh rocks etc. Aerial photography can be valuable in the preliminary stages of such an evaluation as described below.

TOPOGRAPHICAL MAPS

9. Where traditionally surveyed topographical maps exist, they should be used. Otherwise, graphical maps can be produced from aerial photographs by photogrammetric techniques. Most air photo specialist contractors offer this service.

10. Topographical maps can give valuable information on the near-surface hydrogeology of an area, including:

a. surface expressions of the geology

'Soft' rocks such as chalk weather to a smooth, undulating relief, whereas 'hard' rocks, such as igneous rocks or metamorphosed sediments, usually result in rugged relief. Alternating series such as sandstones and shales produce a typical 'stepped' profile in valley sides. Glacial moraines can often be distinguished by a hummocky surface.

b. topographic controls of surface drainage

Steep gradients result in high surface run-off while shallow slopes are more conducive to recharge of an aquifer. A widely spaced surface drainage network indicates underlying permeable rocks while a dense drainage network develops on relatively impermeable surfaces such as clays.

Precipitation is usually greatest over high land, which forms a recharge zone where it is permeable. In a permeable catchment, groundwater flows from topographic highs to lower ground where it discharges to rivers; thus the water table takes the shape of the surface but in a much more subdued form.

c. surface expressions of hydrology

Local features such as springs, sinks and marshes are also of importance. Springs mark the junction of permeable and impermeable rocks where the ground surface intersects the water table. Such spring-lines usually indicate the presence of a perched aquifer, above the regional water table. Solitary springs mark the discharge of groundwater from fissures in, for example, sandstones and limestones. Disappearing streams and sinkholes are often associated with karstic limestone and indicate the presence of permeable rocks above the regional water table. Areas of marsh result from water-logged conditions due either to impermeable soils which hinder recharge, or to the water table being close to the surface.

AERIAL PHOTOGRAPHS

11. A considerable range of methods, including panchromatic and multispectral photography and satellite imagery, has been developed to enable remote interpretation of geology, topography and hydrology; these are described fully elsewhere (H-1, H-2).

12. Black and white photographs are usually adequate for reconnaissance work. They provide much more detail about surface features than topographical maps and give indication of subsurface conditions.

13. Overlapping photographs give an exaggerated three-dimensional view of the surface when looked at through a stereoscope; these 'stereo-pairs' are used extensively for

interpretation and mapping. The scale of each photograph is usually accurate at the centre, but is distorted towards the edges.

14. If accurate topographical maps are to be produced, ground control, ie, the determination of the precise co-ordinates and elevations of a series of points on the ground which can readily be identified on the photographs, is required prior to the production of the survey. For approximate work and the interpretation of regional geology, vegetation etc, a composite uncontrolled mosaic of photographs is adequate.

15. Such photographs are often used for geological and geomorphological mapping (H-3, H-4). Photogeological interpretation is simplest where there is little vegetation and only a thin cover of soil. Fault and fracture systems are often more easily discerned from photographs than on the ground, even in vegetated areas. The soil type ie, clayey or granular, gives a general idea of the relative rates of infiltration during rainfall.

16. All surface indications of the hydrogeology described above can be distinguished on aerial photographs. Areas of groundwater seepage and saturated soils appear much darker than unsaturated ground: thus in addition to spring lines, zones of seepage from more homegeneous aquifers may be apparent. Techniques are described elsewhere (H-1, H-5).

17. The areal coverage of different plants or assemblages of plants can give an estimate of the amount of water lost by evapotranspiration and by interception and evaporation (H-6, H-7). The plant species may also indicate the underlying rock type or soil: ash trees are closely associated with carbonate rocks, while heath and peat assemblages most often develop on sandstone.

18. Aerial photographs are often more recent than published topographical maps, and may show a surface drainage pattern that has changed in detail. Such changes may indicate long-term trends - dried up springs would result from periods of relatively low precipitation and recharge. However, increased abstraction of groundwater or changes in land use could result in the same phenomenon. Consideration must also be taken of the time of year in which the photographs were flown, to distinguish seasonal from long-term changes.

19. Conventional black and white aerial photographs for all of Canada are available from the Department of Energy, Mines and Resources in Ottawa (National Air Photo Library, 615 Booth St, Ottawa, K1A 0E9).

20. Photographs using special films, filters or scales, which are valuable in some investigations, must be specially ordered and flown to the required specifications. Multi-spectral photography, using various colour filters to produce photographs which greatly increase the facility with which vegetation, surficial geology and groundwater variations can be recognized, is being used increasingly (H-1, H-2). Satellite imagery, which incorporates special emulsion and spectral presentations, is also useful; details are available from the Canada Centre for Remote Sensing, 2404 Sheffield Road, Ottawa, K1A 0E4 (H-1, H-2).

21. A certain amount of ground checking is always required to confirm the interpretation and to establish key indices. These are discussed in detail in most manuals of interpretation.

HYDROLOGICAL DATA

22. The regional groundwater pattern can be evaluated by considering:

- records of precipitation and surface run-off
- temperature records and freeze/thaw characteristics of the region
- groundwater levels
- evidence from topographical maps or air photos of infiltration or recharge, and seepage or discharge.

23. Precipitation and stream discharge records are generally obtainable from government regional measuring stations and local industrial, forestry and agricultural organizations. Interpolation between stations can be carried out with reasonable reliability. In temperate climates, 10 years' data are generally sufficient to establish the seasonal pattern of fluctuation, although 50 years' data are necessary for reliable prediction of future trends.

24. Stream discharge hydrographs are compiled

and used to estimate the amount of infiltration and surface run-off over a catchment area. Alternatively, the potential evapotranspiration can be calculated and a water balance obtained from these and the precipitation data. Techniques are described in standard texts, such as Chow(H-8), but generally require specialist involvement.

25. When direct measurement is possible, a short period of data collection will indicate the order of seasonal variation in the water table, and the time-lag response of streams and the groundwater level to precipitation. The volume of flow in streams can be measured by a simple weir or notch flow meter. The volume passing the weir is calculated from the level of water flowing over the weir. An automated recorder giving a continuous record of flow rate is the best method; such recorders can also be set in wells or boreholes to record water table variations. If automatic instruments are not available, a record of daily measurements should be kept.

FIELD TECHNIQUES

26. Regional groundwater evaluation in the context of feasibility studies or initial exploratory investigations mainly involves desk studies. These can produce an overall impression for a single site or comparisons between two or more alternative sites. To firm up the evaluation, field studies are necessary, some of which have already been referred to. Others are described below.

General Field Observations

27. Field observations are required to supplement and verify the interpretations of the topographical and geological maps and aerial photographs. Rock outcrops, areas of groundwater flow, springs etc, and vegetation types can be verified and plotted rapidly in the field once the initial interpretation of maps and photographs is complete.

Borehole Observations

28. Contours of standing water levels measured in wells or boreholes tapping the same aquifer, together with seepage or spring lines, give the shape of the piezometric surface.

Tracer Tests

29. Tracers are used to study the continuity of flow between the injection and sampling points. Tracers are chemicals or naturally occurring substances in the groundwater that can be identified in extremely low concentrations. Ideally they should remain in solution and not be absorbed by the porous media through which they are transmitted. Common tracers consist of dyes, eg, chromates and fluorescein, electrolytes, eg, sodium chloride and copper sulphate, and radio-active substances, eg, cobalt and tritium. Their presence is detected respectively by visual, electrical and radiometric observation. In practice, tracers have limited application to groundwater engineering. They are most commonly used to check whether or not a flow path exists, rather than to determine actual flow velocities.

Geophysical Tests

30. Various geophysical methods may be used to determine geological structure. As previously indicated, such information is extremely valuable in evaluating regional hydrology. Resistivity and seismic methods have been applied successfully to the detection of the water table and to variations in aquifer thickness. Such methods are only suitable where the geology is simple and straightforward and require specialist interpretation.

31. Subsurface data, eg, borehole resistivity or potential logging and geological interpretation of boreholes are required for calibration and interpretation.

REFERENCES

H-1. Estes, J.E. and Senger, L.W. (Ed) "Remote sensing techniques for environmental analysis"; John Wiley & Sons, Inc.; 1974.

H-2. American Society of Photogrammetry "The manual of remote sensing"; (2 vol); Washington; 1976.

H-3. Allum, J.A.E. "Photogeology and regional mapping"; Pergamon Press; 1966.

H-4. U. S. Dept. of the Interior "Aerial photographs in geologic interpretation and mapping"; Geological Survey Professional Paper 373; Washington; 1960.

H-5. Nefedov, K.E. and Pepova, T.A. "Deciphering of groundwater from aerial photographs"; National Aeronautical and Space Administration Technical Translation F-681; Amerind Pub. Co.; New Delhi; 1972.

H-6. Meyboom, P. "Groundwater studies in the Assiniboine River drainage basin, Saskatchewan, Part II"; Geological Survey of Canada Bulletin 139, Dept. of Energy, Mines and Resources, Ottawa; 1967.

H-7. Freeze, R.A. "Regional groundwater flow - Old Wives Lake drainage basin, Saskatchewan"; Scientific Series No. 5, Inland Waters Branch, Dept. of Energy, Mines and Resources, Ottawa; 1969.

H-8. Chow, V.T. "Handbook of applied hydrology"; McGraw-Hill Book Co., New York; 1964.

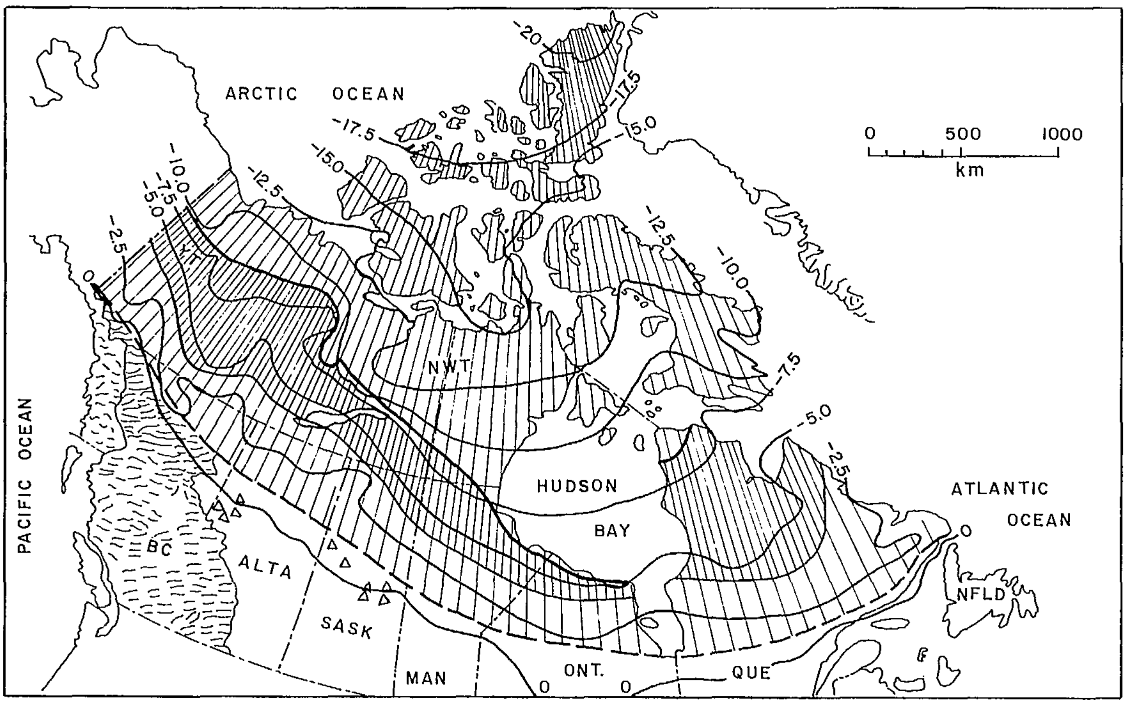
APPENDIX I

PERMAFROST

1. Permafrost - permanently frozen ground - occurs extensively in Canada. Figure I-1 shows the regions of continuous and discontinuous or intermittent permafrost. In the discontinuous region, zones of both frozen and unfrozen ground are found.

2. Permafrost zones include an active surface layer of ground that thaws in summer and overlies permanently frozen ground whose thickness ranges up to several hundred feet. Below the permafrost free groundwater may exist. The active layer may be several inches thick, or may extend many feet. Mining in permafrost may therefore be over, in, or below actual frozen ground.

3. Mining in permafrost has advantages and disadvantages. Frozen ground is inherently strong and permits steep, stable slopes. On the other hand, excavation in permafrost can be difficult and expensive. Slopes in frozen, stable ground may become weak and slough continually if surface thawing occurs. Surface water, free groundwater and groundwater resulting from thawing interact complexly, and the effect on mining operations may be difficult to forecast.



LEGEND

- Continuous permafrost zone
- Southern limit of continuous permafrost zone
- Widespread permafrost
- Southern fringe of permafrost region
- Southern limit of permafrost
- Patches of permafrost observed in peat bogs south of permafrost limit
- Permafrost areas at high altitude in Cordillera south of permafrost limit
- Mean annual air temperature, °C

Fig I-1 - Permafrost distribution in Canada.

4. Much of Canadian experience to date in permafrost mining has been gained by the Iron Ore Company of Canada Ltd (IOC) in its operations around Schefferville, Quebec, and by Cassiar Asbestos Corporation Ltd at their Clinton mine, in the Yukon. Descriptions of their experiences now follow.

IOC

5. IOC operate numerous pits in intermittent permafrost on the Quebec-Labrador border (Fig I-2). Permafrost problems have arisen chiefly in drilling and blasting, in drainage of ground and surface water, and in slope stability. Difficulties have also been encountered in handling ore, both in transportation from the face, in crushing, and in shipping by rail. These latter difficulties, however, are beyond the scope of this appendix.

Drilling and Blasting

6. Production blasting is more expensive in permafrost. Ice tends to absorb energy produced by the explosives. This results in large blocks which may require secondary blasting (Fig I-3). To overcome these problems, IOC has found that hole spacing must be reduced from the normal 26 ft by 26 ft (8 m by 8 m) to as little as 16 ft by 16 ft (5 m by 5 m). Increased powder factors and the use of expensive high energy explosives like aluminized slurries are required. Heat from the drill may melt the ice around the hole, causing serious caving conditions. To overcome this, sleeves may be required, or the hole might require redrilling. As a result the total cost of blasting in permafrost is three to four times that in unfrozen ground. In addition, blasting in permafrost often results in an uneven pit floor, which adversely affects operations and therefore production. Depending on the amount of ice and degree of weathering, unexpected caving conditions in holes may occur during development drilling. Also in some cases, the hard nature of frozen rock slows down the penetration rate, adding to cost. Reaming and casing, where necessary in frozen ground, increase the overall cost substantially. Alternatives to casing, such as the use of antifreeze, are even more expensive.

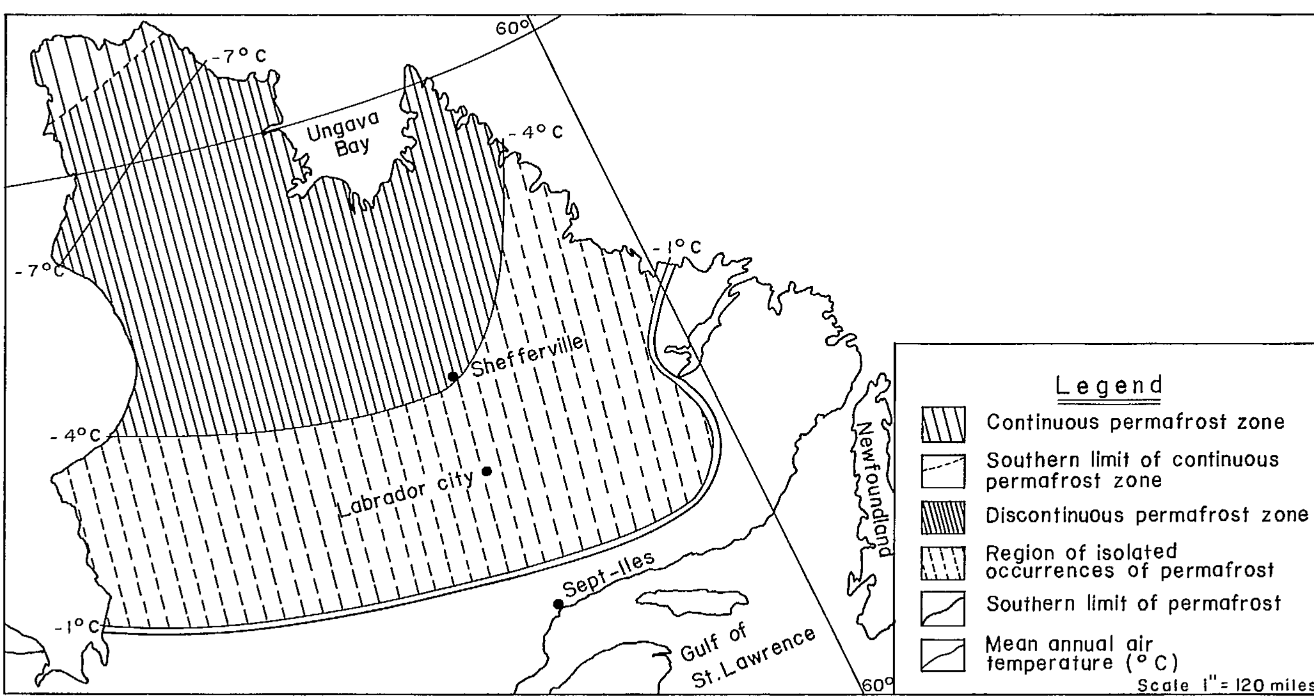


Fig I-2 - Operations of Iron Ore Company of Canada in Quebec-Labrador.

Fig I-3 - Poor blasting fragmentation attributed to permafrost.

Drainage

7. The difficulties that may arise in draining permafrost regions are illustrated at an IOC mine. Figure I-4(a) shows schematically how surface water flowed over frozen ground and into the pit. A dike was constructed from frozen material to dam the surface water (Fig I-4(b)). However, as the impounded water rose, the permafrost beneath thawed, allowing seepage beneath the dike (Fig I-4(c)). Eventually the top of the permafrost was substantially lowered (Fig I-4(d)), creating worse conditions than originally existed.

8. The difficulties described above are now avoided by diverting surface water before it

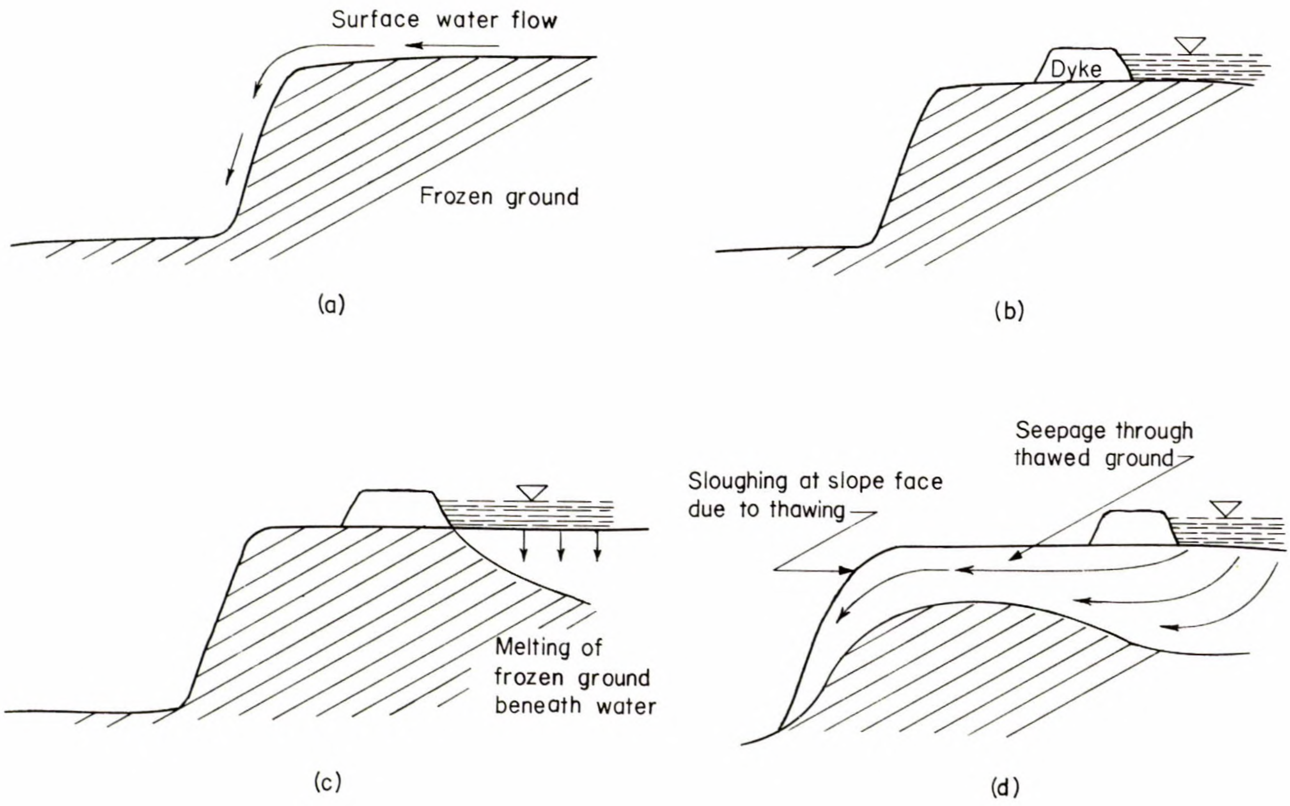


Fig I-4 - Effect of impounding groundwater above permafrost. (a) surface water flowing over crest disrupts pit operations. (b) water impounded by dyke. (c) impounded water forms a heat source and permafrost beneath thaws. (d) thawed zone spreads and weakens slope material, causing operational difficulties.

reaches the pit, using ditches and sumps. Care is taken to make sure the surface water is channelled well away from the pit.

9. Dewatering of groundwater as well as drainage of surface water is essential in most IOC pits, a) to reduce the moisture content of ore, b) to achieve dry operating conditions and, c) to improve slope stability. Deep wells both within and around the pits are used for this; a typical well layout is shown schematically in Fig I-5. It is essential for wells to penetrate below the base of the permafrost and also well below the true water table.

10. Wells drilled apparently successfully below the permafrost are sometimes not satisfactory. It is suspected this is due to thawing and settlement of ground around the wells, damaging the pump drive shaft and preventing satisfactory operation of the well.

Stability

11. Permafrost affects slope stability in two ways. First, freezing usually increases the strength of the material of the slope, and as a result the slope may be more stable. Second, frozen zones may influence the distribution of surface and groundwater, and therefore the effect of water on stability.

12. The experience with surface drainage diversion at IOC, described above, illustrates both these aspects of stability. The thawing of surface material extended eventually to a depth of about 100 ft (30 m) (Fig I-4(d)). At one pit face, severe sloughing resulted, due both to the reduced strength of the thawed material and to the adverse effects of seepage in the thawed zone. As a result, the designed 45° overall slope was to be reduced to around 35° overall to ensure adequate stability. This would be done by introducing a safety berm which would also act as a catchbench for loose material. It was felt that a 45° slope in undisturbed frozen ground would have been completely stable.

13. Face freezing, although not strictly related to permafrost, can cause stability problems by allowing water to build up behind the slope face. IOC have observed this effect, for example by finding steady seepage from a newly excavated toe of a frozen slope, although the face above was dry (Fig I-6). Slides may result unless the water impounded by the frozen face is drained.

14. Impounding water behind a frozen slope face and below a pit floor may create the problem shown in Fig I-7. Here artesian pressures may interrupt production by affecting blast holes in addition to affecting slope stability.

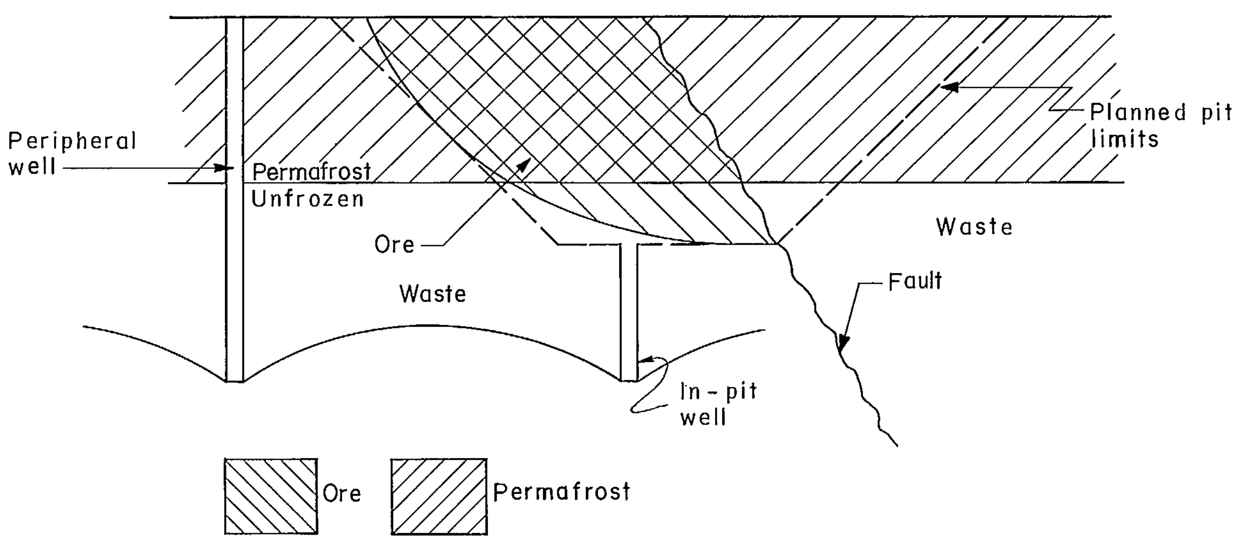
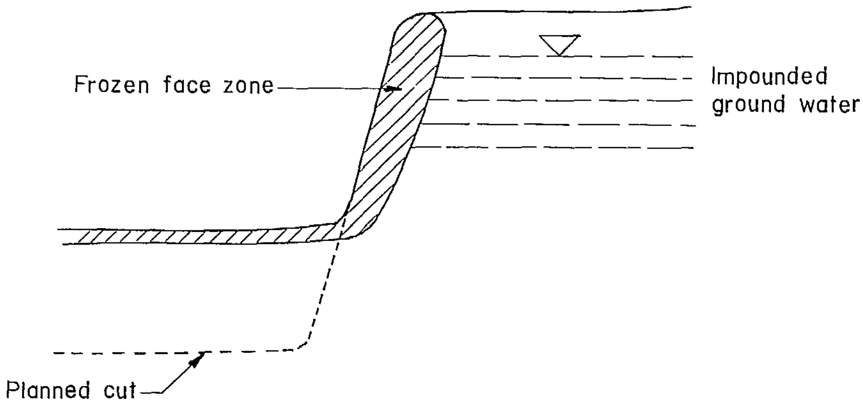
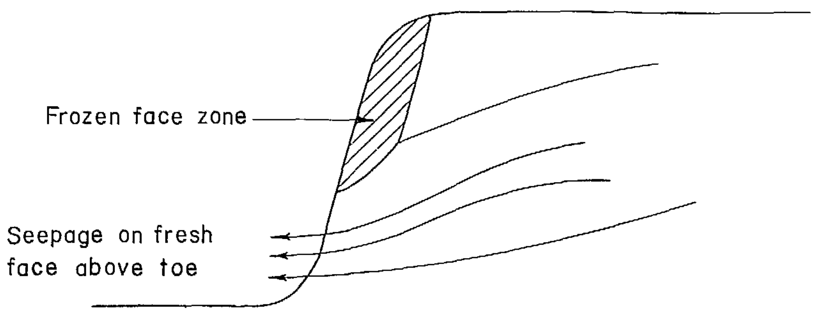


Fig I-5 - Typical well layout at IOC operations.



(a)



(b)

Fig I-6 - Excavating below a dry face (a) can result in toe seepage.

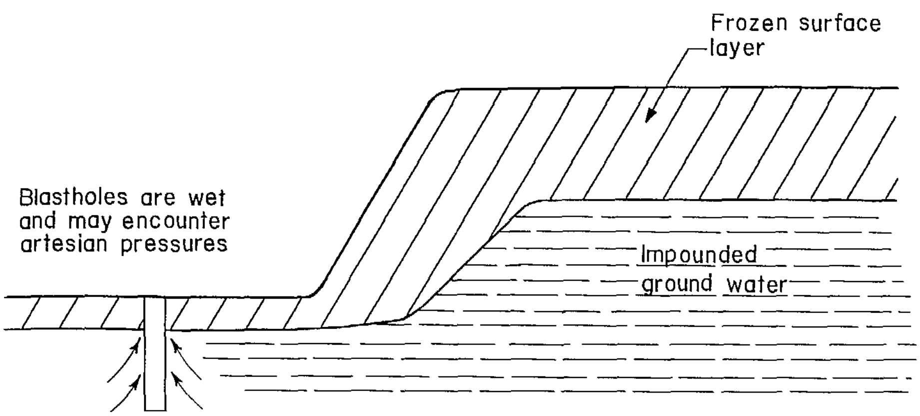


Fig I-7 - A frozen surface layer can result in artesian conditions and wet blast holes.

Mapping Permafrost

15. Permafrost at IOC is sufficiently significant to operations that detailed maps of permafrost zones are required. Mapping and classifying permafrost is now carried out routinely using a variety of techniques. Measuring ice content and distribution and determining shear strength and modulus are done for three reasons: a) to provide data in solving day-to-day operating problems such as described above; b) because the influence of permafrost on operations is found to vary with both the actual ice content and its nature; and c) for determining long range mining schedules.

16. The initial mapping of permafrost zones is done by analyzing topography and vegetation (Fig I-8). In the discontinuous zone, permafrost is generally found in elevated areas such as ridges, especially with sparse, tundra vegetation. Permafrost is also found in low-lying areas where snow cover in winter is shallow. Low-lying areas with a dense growth of vegetation or deep winter snow cover are usually not frozen.

17. Surface water from rivers, streams and lakes results in unfrozen zones, because more heat is conducted into the ground in summer than is lost in winter. Frost heaves or boils indicate permafrost within and beneath a shallow active layer. Cause of this is freezing of the water overlying the permafrost; expansion through freezing results in ground heave. The boils are conical in shape and rarely exceed 20 ft in lateral extent and 5 ft in height. Figure I-9 show a typical frost boil at IOC.

18. Table I-1 lists some of the topographical and vegetative characteristics associated with permafrost or unfrozen ground. Talik conditions, in the middle column, are pockets of unfrozen ground existing permanently in otherwise frozen regions. They differ from unfrozen ground in their relatively small extent, and in usually being underlain by permafrost.

19. The initial mapping by analysis of topography and vegetation is expanded by seismic and resistivity surveys, and to a lesser extent by geophysical logging in boreholes. Detailed confirmation of permafrost is then made where required by temperature measurements, using thermistors.

Seismic Mapping

20. Seismic refraction surveys are used to establish depths to overburden, and to the top of

Fig I-8 - Changes in vegetation can reveal the presence of permafrost. Here a tongue of permafrost between talik zones is shown by the lack of vegetation.

Fig I-9 - A typical frost boil.

the permafrost. The detectors, or geophones, are laid out along the geological strike to avoid the influence of dip on readings. Seismic surveys are undertaken in late fall when surface frost has effectively disappeared.

Table I-1: Topographical and vegetative characteristics of permafrost

	Permafrost	Talik	Unfrozen
Topography	ridges	valleys	low-lying ground
Snow depth	shallow	medium to deep	medium to deep
Vegetation	moss/lichen	dense growth of betula and birch	thick vegetation including trees
Drainage	impeded drainage	few but prominent channels	natural drainage

21. Depth to the various layers is calculated using standard relationships between velocities and critical distances. The following ranges of velocities broadly cover the types of material found at IOC.

a. unfrozen overburden - less than 3500 ft/sec (1000 m/sec);
b. frozen overburden and leached unfrozen rock - 3500 to 6000 ft/sec (1000 - 1800 m/sec);
c. unfrozen bedrock - greater than 6000 ft/sec (1800 m/sec);
d. frozen bedrock - much greater than 6000 ft/sec (1800 m/sec), up to 20,000 ft/sec (6000 m/sec).

22. The results of a typical seismic survey are shown in Fig I-10.

23. Seismic surveys can only be used to detect the top of permafrost, and not the base. This is because seismic methods can only detect a transition where a region of low sonic velocity overlies a region of high sonic velocity and not the reverse; frozen ground has a higher sonic velocity than unfrozen.

24. Routine seismic maps are produced at a scale of 1 in. = 40 ft and 1 in. = 100 ft (1:480 and 1:1200).

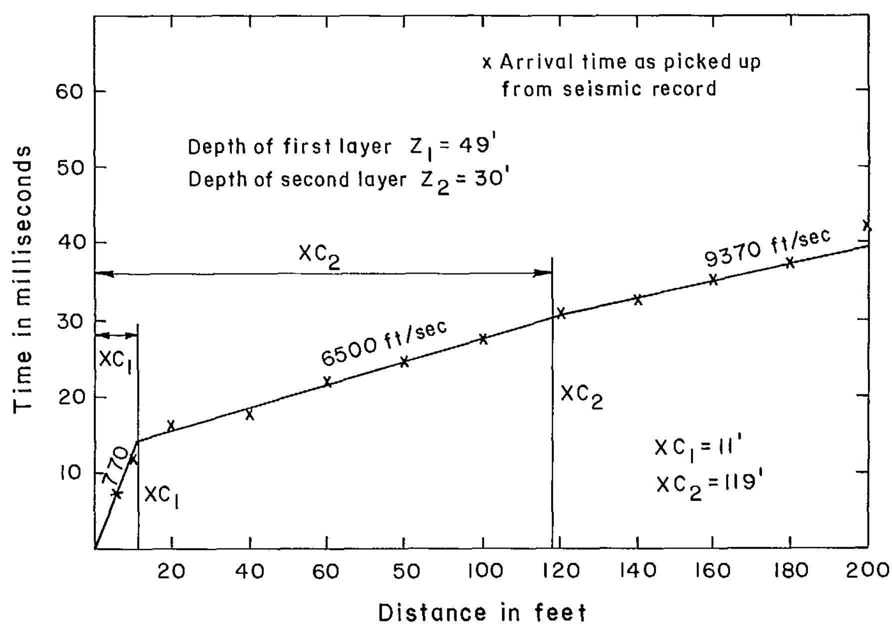


Fig I-10 - Typical seismic survey results. Arrival times of the seismic signals allow the depths to the various layers to be determined.

Resistivity Survey

25. Resistivity surveys are used to delineate the base of the permafrost. Survey lines are oriented along the strike of the geological formations. The principle of the survey is to apply a voltage between two fixed electrodes in the ground and to measure the potential drop at various points between them. Standard resistivity curves based on theoretical computations are used to determine the depths to layers with different electrical conductivities; frozen ground has a higher conductivity than nonfrozen ground.

26. The equipment used by IOC is the Soiltest R-60 DC system. The maximum depth of permafrost that can be measured with this equipment is about 150 ft (50 m). Some work has been done, however, with high powered AC voltage sources; depths of about 250 ft (75 m) have been measured with this equipment.

27. Results from resistivity surveys have been found to correlate well with those from direct temperature measurements taken at depth in drill holes.

28. Routine resistivity maps are produced at a scale of 1 in. = 40 ft and 1 in. = 100 ft (1:480 and 1:1200).

Borehole Logging

29. Initial experiments at IOC indicate that geophysical logging, such as by natural gamma and dry hole resistivity methods, has potential for delineating permafrost zones. Similarly, uphole seismic techniques are also believed to have a potential application at IOC. However, these methods have not been used there routinely.

Temperature Measurements

30. Temperature measurements at IOC are made with thermistors mounted on multi-conductor cables. Thermistor sensors have also been used as part of the portable logging unit for measuring ground temperatures in drill holes. Thermocouples have been used in the past, but were found unsatisfactory because of limitations, in terms of EMF fluctuation, of the measuring system in cold weather. Thermistor cables are made up to required lengths, and left permanently in bore holes. A simple commercially available readout unit is easy to use and works satisfactorily even in extreme cold.

31. Results of the topographical and vegetation surveys plus physical measurements are used to produce a large-scale permafrost prediction map at a scale of 1 in. to 1000 ft (1:12000). Experience has shown that the accuracy of such a map, produced before mining commences, is about 500 ft (150 m), ie, measured permafrost boundaries agree within 500 ft (150 m) with predicted boundaries.

32. The data collected on the properties and the behaviour of frozen ground are also useful for site investigations for screening plants, mine service garages, and layout of access roads and rail routes.

CLINTON MINE

33. Clinton Mine is the most northern open pit operation in Canada. It is located approximately 65 miles northwest of Dawson City, Yukon, and 8 miles east of the Canada-Alaska border (Fig I-11). The mine site is on Porcupine and Snowshoe Hills on the west side of Clinton Creek. Locations of the various orebodies are shown in Fig I-11. The plant site is on Trace Hill on the east side of the creek. The townsite is located on the north bank of the Fortymile River near Clinton Creek. Access to the mine is by road from Dawson City, Yukon or by air from Whitehorse.

34. The area lies within the zone of discontinuous permafrost and is subject to severe climatic conditions during winter months. The thickness and nature of permafrost appear to vary depending on the soil or rock type, direction of the slope faces and water content. Precipitation is approximately 10 in. (25 cm) per year, most of which occurs as rain in June, July and August. Annual temperatures vary between -57°C and +27°C.

Groundwater

35. The Porcupine pit shows little visual evidence of groundwater. However, blastholes are wet enough to require Hydromex in the following areas:

a. near sumps and at the pit base;

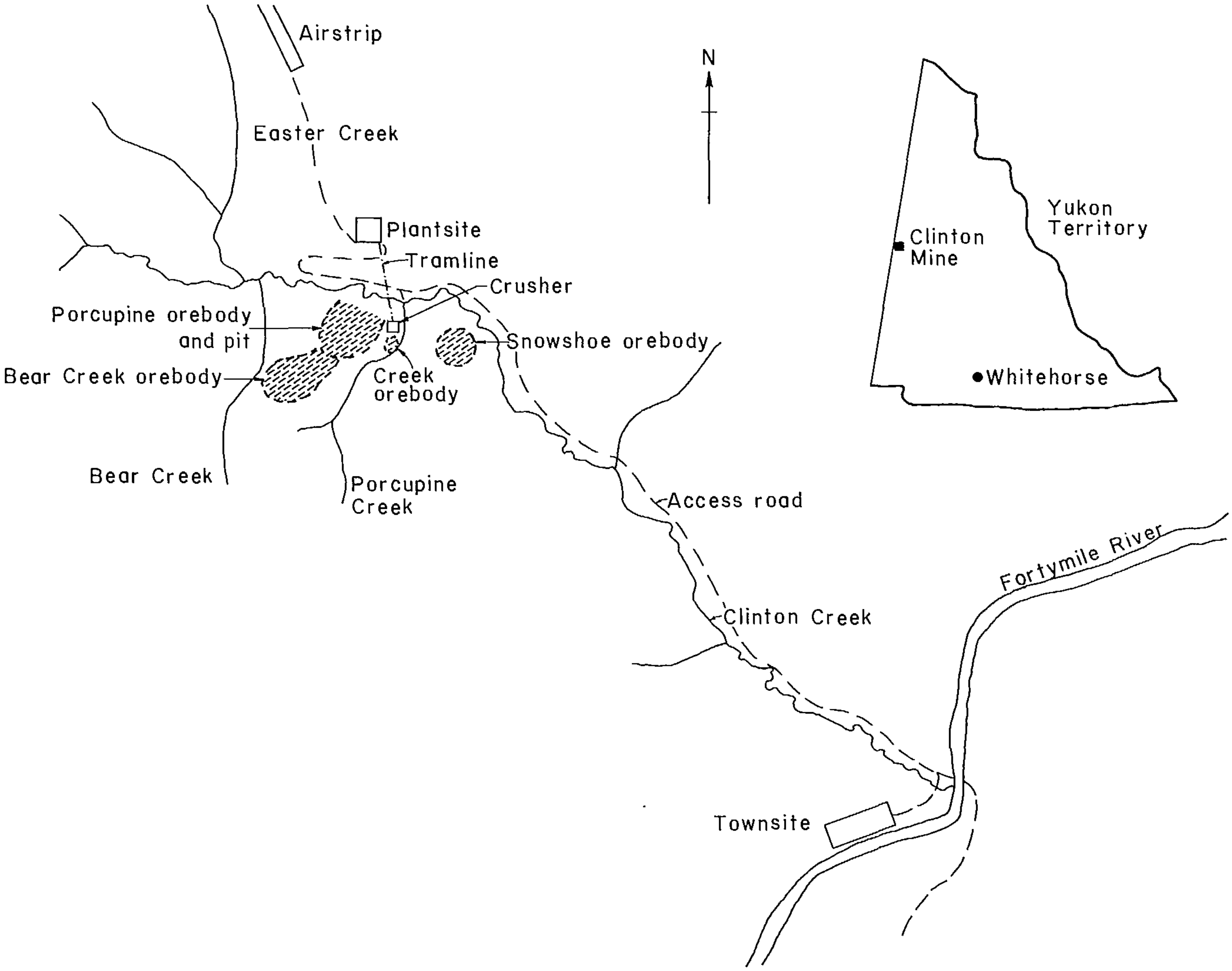


Fig I-11 - Location of Clinton Mine, Yukon Territory.

b. near original ground where surface permafrost thaw, snowmelt and runoff occur;

c. the argillite/serpentine contact in the hanging wall where water collects above the relatively impervious quartz-carbonate alteration zone (ground ice has been noted in this area);

d. two broadly defined southeast-trending zones in the argillite on the hanging wall, probably related to transverse faults or highly faulted zones.

36. Water flow was observed near the transverse faults, particularly the Water Fault (Fig I-12), and isolated damp pockets were observed in some parts of the pit. Several minor seepages appear during mid to late summer and large ice buildups form in the winter months in these same areas.

37. The groundwater table appears to be extremely deep near the pit. Recharge from thaw of surface ice and drill water, which had been allowed to run over the crest of the pit or which had seeped laterally from the drill holes, has probably contributed significantly to slope instability. Water and ice, for example, were evident on the surfaces of several wedge slides observed in the pit.

38. Groundwater is significant in Creek Pit; the water comes through waste dumps which have impounded Porcupine Creek upstream of the pit. In the Snowshoe Pit, which is planned as a side-hill

Fig I-12 - Accumulation of ice resulting from seepage of groundwater along the Water Fault at Clinton Mine.

cut, no groundwater has been encountered in the first few feet of stripping, although hydromex has been required in some blastholes due to thawing of permafrost.

Investigation of Permafrost

Ground Temperature

39. Ground temperature was measured with thermistors. Thermistor measurements in vertical boreholes near the pit crest indicate that the bedrock temperature to a depth of approximately 250 ft (75 m) is from -1°C to +1°C. Actual permafrost depth is from 3 to 13 ft (1 to 4 m) with an active layer 3 ft (1 m) deep.

Moisture Content

40. The moisture content of all samples tested was less than 8%. No ground ice or ice lenses were observed in the bedrock in the Porcupine Pit, except in the upper parts near undisturbed ground. Ground ice has been seen at the contact between the argillite and the quartz-carbonate alteration zone, as well as in the fault zone.

Mining in Permafrost

Slope Stability

41. Pit walls in weak rock generally have much greater strength and cohesion when frozen, especially in those rock masses with moderate to high water content (I-1). The mining operation exposes these walls to seasonal climatic variations, thus severely reducing stability. Water flow may occur in the surface zone of freeze-thaw as well as along faults and through unfrozen or marginally frozen zones.

42. At Clinton mine the effects of the freeze-thaw cycles on the exposed cut slopes cause severe ravelling of the relatively soft argillitic rocks. The result of this action is continuous small rockfalls, with the result that some benches completely disintegrate. The maximum depth of seasonal freeze-thaw at Clinton mine appears to be about 20 ft (6 m).

43. South-facing slopes, which are subject to more intense freeze-thaw action, are generally more susceptible to ravelling and rock falls than north-facing slopes.

Drilling and Blasting

44. Drilling and blasting in the wetter permafrost zones cause some operating problems. During drilling the permafrost melts, producing mud and wet holes which require special waterproof blasting compounds such as Hydromex to ensure detonation. Mud clogs bits and drilling is slowed considerably. Spare bits must be kept on hand and frequent bit changes are required to clean off the mud. To remove mud and water, blastholes are blown out with air, but permafrost continues to melt and water often continues to infiltrate the blasting compound. Creep sometimes occurs under these conditions as well, causing blastholes to close up, cutting off part or all of the blasting compound and resulting in incomplete detonation.

45. Due to the extra strength imparted to the rock by ice, blasthole spacing and burden are

generally only half that in unfrozen rock. This is necessary to obtain adequate fragmentation and breakage.

46. In zones of wet permafrost, or even in unfrozen ground, blasted material and muck piles will often refreeze with the result that the material can be excavated only with difficulty.

47. The general approach in wet permafrost ground at Clinton mine is to drill, load and blast every two or three days. Holes are drilled, blown out, loaded and blasted and the muck cleaned out as quickly as possible to avoid problems. In dry permafrost, which comprises most of the pit, no specific problems related to drilling and blasting are encountered.

48. In areas such as the Creek pit where groundwater hampers mining operations and degrades ore quality, it is common practice to mine during the winter months when water flow is restricted by freezing.

Excavation

49. Overburden in permafrost must generally be blasted before being removed. If not blasted, the material is invariably too hard to dig. If permafrost near surface is stripped and allowed to melt, it turns to mud and water and excavation becomes difficult.

50. At Clinton mine blasted waste is excavated with belly dump scrapers and shovels and blasted ore is excavated with front end loaders or shovels. Muck removal schedules are kept close to blasting schedules so that refreezing of muck does not occur.

Road Construction and Maintenance

51. Roadbeds in permafrost are cut on a slight angle so that water and mud will be diverted from the road and the permafrost will melt evenly. Most mine area roads are on bedrock; few permafrost-related maintenance problems are apparent.

REFERENCE

I-1 Glozman, G.R., Yermakov, I.I. and Bodov, N.G. "Stability of the sides of open pit mines in permafrost"; Groundwater in the Cryolithosphere; U.S. Army Corps of Engineers CRREL; Draft Translation 437; pp 291-295; 1973.

GLOSSARY

ANISOTROPIC

Having physical properties that vary in different directions. Contrasts with isotropic.

AQUICLUDE

A formation which although porous and capable of absorbing water slowly, will not transmit it fast enough to furnish an appreciable supply for a well or spring. Aquicludes thus include intact rocks and gouge filled faults.

AQUIFER

Stratum or zone below the surface of the earth capable of transmitting and producing water as from a well. Aquifers are thus relatively permeable rocks or heavily fractured rock masses.

AQUITARD

A slightly permeable stratum which permits some passage of water.

ARTESIAN WATER

Artesian conditions exist where water is confined under pressure beneath overlying impermeable strata. Such water rises towards the surface when tapped by a well. Where the pressure is sufficient, the water flows out at the surface and constitutes a flowing well.

ASPERITY

The peak or ridge on a rock discontinuity resulting from the natural roughness or undulations.

BULK PERMEABILITY

The total flow per unit of the rock mass in a given direction. This takes into account the contribution of all discontinuities for that flow direction. Also known as mass permeability.

CONDUCTING PAPER ANALOGUE

Conducting paper (eg, Teledeltos paper) may be used to form the basis of a simple electrical analogue for studying groundwater problems. The paper is cut to the desired shape and equipotentials traced on the sheet by means of electrical probes. The method is best suited to the solution of isotropic seepage problems.

CONE OF DEPRESSION

When water is pumped from an aquifer at a steady rate, the surface of the water table around the well is lowered, and it assumes roughly the form of an inverted cone, known as the cone of depression. When a cone is developed, the pressure gradient thus established produces the necessary flow towards the borehole to balance the amount of water which is being extracted.

CONSTANT HEAD TEST

The principle of the constant head test is to pressurize a known section of the borehole above the static pressure level and to measure the resultant flow into the formation. From the observed relationship between stable flow rate and excess pressure, the bulk permeability of the rock mass or the hydraulic conductivity of individual fissures can be calculated.

CUMEC

Cubic metre per second.

CUSEC

Cubic foot per second.

DARCY'S LAW

Darcy's Law states that the velocity of flow through a saturated soil is proportional to the hydraulic gradient. This can be expressed mathematically as:

$v = ki$ where v is the seepage velocity
i is the hydraulic gradient
k is the coefficient of permeability.

It is more commonly written:

$Q = Aki$ where Q is the flow rate
A is the cross-sectional area through which flow takes place.

DILATION

An increase in the volume of a rock mass or in the opening between two faces of a discontinuity during shear movement, caused by adjacent portions of rock "riding up" over asperities.

DISCHARGE

Rate of flow at a given instant in terms of volume per unit time.

DISCHARGE AREA

Any area where water is flowing out of the rock mass.

DISCONTINUITY

Any natural parting in a rock mass separating adjacent solid pieces of rock, eg, joint, bedding plane, fracture, etc.

DISCRETE RESISTANCE ANALOGUE

A grid of fixed resistances which represent the permeability or conductivity conditions of an area of groundwater study. The analogue may be used to simulate the bulk permeability of rock mass or the conductivity of fissures.

DRAWDOWN

The lowering of the water table or piezometric surface when pumping, discharge or natural inflow is in progress. The difference between the static water level and the drawdown level produces the pressure head difference that causes water to flow through the rock towards a well or excavation.

EFFECTIVE STRESS

The effective normal stress across a joint at any point within a rock mass is the total stress due to external and gravity loading minus the groundwater pressure at that point. The effective stress will govern the shearing resistance that can be developed across the joint.

ELECTRICAL RESISTANCE ANALOGUES

The law governing electrical flow (Ohm's Law) is analogous to that governing the flow of water in rock and soil (Darcy's Law). Groundwater problems may thus be solved by the use of suitable electrical analogues such as conducting paper, resistive ink or discrete resistances.

ELEVATION HEAD

The height of a given point with respect to a particular elevation datum.

EQUIPOTENTIAL

Equipotentials are lines drawn through points of equal hydraulic potential.

EVAPOTRANSPIRATION

The total of evaporation and transpiration.

FALLING HEAD TEST

The falling head test consists of subjecting a section of a borehole to a pressure head above the static groundwater pressure. This head is then allowed to fall to the static value while measurements of the loss in head with time are taken. From this, a relationship between flow and pressure is determined from which the permeability is calculated.

FLOW LINE

The path that a molecule of water will follow when flow is taking place.

FLOW NETS

Flow nets are a graphical method of representing the overall flow conditions within a slope. They are plots showing flow lines (the path that a molecule of water will follow) and equipotentials (contours of equal hydraulic potential) arranged on a regular grid. From a flow net, three useful data items can be determined: rate of flow, head and gradient.

HYDRAULIC CONDUCTIVITY

Analogous to permeability. Hydraulic conductivity is a property of the medium (ie, the rock, rock mass or soil). Often used in reference to joint characteristics.

HYDRAULIC GRADIENT

The hydraulic gradient 'i' between two points in a rock mass within which water is flowing is defined as the difference in hydraulic potential between those two points divided by the length of the flow path.

HYDRAULIC HEAD (HYDRAULIC POTENTIAL)

The sum of the elevation head referred to a specific datum, plus the groundwater pressure, expressed in terms of a head of water, at that point.

HYDROGEOLOGY

The study of the geological factors relating to the behaviour of groundwater.

HYDROGRAPH

A graph showing the change of flow rate against time for a river or stream at a particular cross section.

HYDROLOGICAL CYCLE

The complete cycle through which water passes - commencing as atmospheric water vapour, passing into liquid and solid form as precipitation, then along or into the ground surface, and finally again returning to the form of atmospheric water vapour by means of evaporation and transpiration.

HYDROLOGY

The study of the behaviour of surface water and, in some cases, groundwater.

HYDROSTATIC PRESSURE

The pressure exerted by the water at any given point in a body of water at rest. The magnitude of this pressure is given by the product of the fluid density and the fluid height.

INFILTRATION

The flow or movement of water through the soil surface into the ground.

INFILTRATION CAPACITY

The maximum volume of falling rain which soil in a given condition can absorb.

INFILTRATION RATE

Maximum rate at which soil can absorb rain or shallow impounded water.

ISOTROPIC

Having the same properties in all directions. Contrasts with anisotropic.

JOINT CONDUCTIVITY

Joint conductivity refers to the flow characteristics of a single specific fissure. It is analogous to permeability. Conductivity can be defined for a given fissure as:

$$q = k_j \cdot i$$

where q is the flow per unit width of the fissure
k_j is the conductivity of the fissure
i is the hydraulic gradient in the direction of flow.

LEAKY ARTESIAN CONDITIONS

An aquifer bounded above and below by slightly permeable layers (aquitards).

MASS PERMEABILITY

see BULK PERMEABILITY.

NON-LEAKY ARTESIAN CONDITIONS

An aquifer bounded above and below by impermeable layers (aquicludes).

PACKER COEFFICIENT

The use of the packer coefficient is a simplified method of relating permeability to flow rate and excess head in a pressure test over a section of borehole. The relationship is given by:

$$k = C_p \ (Q/h_e)_o$$

where K is the coefficient of permeability. C_p is the packer coefficient, values of which may be found in various texts. $(Q/h_e)_o$ is the slope of the flow v excess head graph, measured at the origin (see Appendix C).

PERCHED WATER TABLE

A perched water table may be formed if a local impervious layer (aquiclude) such as a clay seam, occurs in a permeable deposit above the main water table. Under such conditions, a body of groundwater may be held up by the aquiclude to form a perched water table.

PERMEABILITY

The permeability of a material is its capacity for transmitting a fluid. The degree of permeability depends on the size and shape of the pores and their interconnections. The coefficient of permeability, k, is defined as the rate of flow per unit area of soil or rock under unit hydraulic gradient. Permeability has the dimensions of a velocity and is usually expressed in centimetres per second. It represents the velocity which would produce the same rate of discharge if the water flowed through the whole area instead of through the voids. The term permeability applied to a rock mass is usually implied to mean 'mass or bulk permeability' which expresses the total flow per unit area of the rock in a given direction and takes into account the contribution of all discontinuities for that flow direction. Generally the permeability of intact rock material is relatively insignificant compared with the mass or bulk permeability.

PHREATIC SURFACE

There are two different usages of the term phreatic surface in groundwater texts. Throughout the Groundwater Chapter, it has been used as synonymous with water table, which is the commoner usage. Elsewere, it may be used in a context synonymous with piezometric surface.

PIEZOMETER

A device for measuring water pressure. These may vary in complexity from simple standpipes to more complex instruments using electrical or pneumatic transducers. The devices are generally installed in boreholes and sealed into the holes in such a way as to measure the piezometric pressures at the sampling points only. Piezometers may broadly be classified on the basis of the volume of water required to operate the device. Those piezometers with a high volume demand (eg, standpipes) are unsuitable for measuring rapid changes in groundwater conditions in low permeability materials. Low volume demand piezometers require very small volume changes in the water and thus give a more rapid and accurate response to changing groundwater pressures.

PIEZOMETRIC HEAD

The hydraulic head measured by a piezometer.

PIEZOMETRIC SURFACE

A piezometric surface is a profile of water pressure on a given surface. Suppose true groundwater pressures are measured at a series of points on a given surface, A, say, in the zone of groundwater flow. Let these pressures be represented physically by vertical heights, proportional to the pressure, above the points of measurement, eg columns of water such that the water head corresponds to the pressure. A surface joining the tops of the columns of water is the piezometric surface for A. Thus, if sealed stand pipes are installed in a zone of flow so that each stand pipe measures pressure at a point on surface A, the water levels in the sealed stand pipes define the piezometric surface for A. It follows that there is in general a different piezometric surface for each surface through the zone of flow. For hydrostatic conditions only - no flow or uniform horizontal flow - the phreatic surfaces all coincide with the water table. In slope stability analysis, the water pressures on a surface of sliding can be correctly calculated if the piezometric surface for the surface of sliding is known. The use of any other piezometric surface, or the water table, to calculate pressures generally results in erroneous pressure values.

POTENTIAL

See hydraulic potential.

PRECIPITATION

The discharge of water from the atmosphere generally upon a land or water surface. The water may be in the form of rain, hail, snow or sleet. The quantity of precipitation is always measured as a liquid and expressed in units of height such as millimetres.

PRESSURE HEAD

Hydrostatic pressure expressed as the height of a column of water that can be supported by the pressure. It is given by:

$$h_w = p/\gamma_w$$

where h_w is the pressure head (usually given in feet)

p is the pressure

γ_w is the density of water.

PUMPING LEVEL

The level at which water stands in a well when pumping is in progress. In the case of a flowing well, it is the level at which water is flowing from the well.

RECHARGE

The process by which water is absorbed and added to the groundwater storage areas.

RECHARGE AREA

Any area of a rock mass where groundwater is being replenished.

RECOVERY LEVEL

The water level in a well at a given time during the recovery period.

RECOVERY PERIOD

The period of time, after pump shut down, which is required for the water level in a well to return to the original (prepumping) level.

RECOVERY TIME

The period of time which has elapsed since the pump was shut down.

RESIDUAL DRAWDOWN

The original prepumping water level minus the water level at a given time after the pump was shut down.

RESISTIVE INK ANALOGUE

An electrical analogue of groundwater flow may be made by the use of ink containing a colloidal suspension of graphite. The method is suitable for the study of flow through jointed rock.

RQD

Rock quality designation.

RUNOFF

The proportion of precipitation discharged through surface water systems. The quantity of water discharged in streams etc, is usually expressed in units of volume, such as cusecs (cubic feet per second).

SPECIFIC YIELD

The specific yield of a rock or soil is the ratio of the volume of water that the saturated material will yield by gravity, to its own volume.

STATIC WATER LEVEL

The level at which water stands in a borehole when no water is being taken from the aquifer either by pumping or free flow. The depth to static water level is generally expressed as the distance from the ground surface (or from a measuring point near the ground surface) to the static water level.

STORAGE COEFFICIENT

The volume of water released from storage in each vertical column of the aquifer having a base 1 ft square when the water table or other piezometric surface declines 1 ft. This is approximately equal to the specific yield for non-artesian aquifers.

TIME LAG

Time lag exists in water pressure measurements because the pressure in the measuring system initially differs from that in the soil thus requiring flow into or out of the piezometer measuring element before equilibrium is reached. The time lag for a piezometer to reach equilibrium with the in situ groundwater pressures depends on the permeability of the material and the piezometer type.

TRANSPIRATION

The process by which water vapour escapes from a living plant and enters the atmosphere.

WATER TABLE

The water table defines the upper limit of the saturated zone within a rock mass, as indicated by a series of shallow observation wells in free communication with the rock. The groundwater pressures at such a surface are zero. (see PHREATIC SURFACE).