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CONTROL OF GROUNDWATER IN SURFACE MINING

C. O. Brawner

Consulting Geotechnical Engineer, and
Professor, Mining Department, University of B.C.,
Vancouver, Canada.

ABSTRACT

The presence of groundwater in surface mining operations often creates serious problems. The most important is generally a reduction in stability of the pit slopes. This is caused by pore water pressures and hydrodynamic shock due to blasting which reduce the shear strength and seepage pressures, water in tension cracks and increased unit weight which increase the shear stress. Groundwater and seepage also increase the cost of pit drainage, shipping, drilling and blasting, tyre wear and equipment maintenance. Surface erosion may also be increased and, in northern climates, ice flows on the slopes may occur. Procedures have been developed in the field of soil mechanics and engineering of dams to obtain quantitative data on pore water pressures and rock permeability, to evaluate the influence of pore water and seepage pressures on stability and to estimate the magnitude of groundwater flow. Based on field investigations, a design can be prepared for the control of groundwater in the slope and in the pit. Methods of control include the use of horizontal drains, blasted toe drains, construction of adits or drainage tunnels and pumping from wells in or outside of the pit. Recent research indicates that subsurface drainage can be augmented by applying a vacuum or by selective blasting. Instrumentation should be installed to monitor the groundwater changes created by drainage. Typical case histories are described that indicate the approach used to evaluate groundwater conditions.

THE INFLUENCE OF GROUNDWATER

In the majority of surface mines, groundwater will generally be encountered below 50-150 meters. The amount of groundwater present, the rate at which it will flow through the rock, the effect it may have on stability and the influence it will have on the economical development of the pit, depends on many factors. The most important of these are the topography of the area, precipitation, temperature variation, the permeability of the rock mass and overburden soil, and fragmentation and orientation of structural discontinuities in the rock.

The most important effect of groundwater on open pit mine stability is the effect it has upon the stability of the pit slopes. This effect is developed in several ways.

(a) Reduction in shear strength

Shear strength is normally expressed by the Coulomb Equation.

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

where σ = the weight normal to the slip surface
u = the neutral or pore water pressure
ϕ = angle of internal friction
c = cohesion

Where the groundwater table is above the potential failure surface the weight of the rock above that surface and below the groundwater table which develops friction along the failure surface is reduced by the buoyant uplift of the groundwater. Below the water table the frictional resistance developed by the rock is reduced by about 37 per cent for average rock. If the groundwater table were to extend to the ground surface, the overall reduction in stability as compared to a zero water pressure condition would approximate this value. In mountainous regions where the surface mine may be well below the level of the surrounding mountains, it is possible that pore water pressures in excess of the total height of the slopes can be encountered. In this case the reduction in stability will exceed 37 per cent. In intact rock the cohesion is significantly influenced by changes in moisture content. Colbach and Wild found that the angle of shearing resistant remains essentially unchanged with the change in moisture but that the cohesion is reduced as the moisture content of the rock increases[1]. The greatest reductions occur in the fine grained clayey rocks. Note that water does not generally act as lubricant to reduce strength. The reduction in strength is due to the reduction in effective stress and/or reduction in cohesion. The greatest reduction in stability can be expected to occur during and shortly after spring snow melt, following heavy rainfall, and in northern climates, during the late stages of winter.

It has frequently been assumed that winter conditions should be conducive to stability since the surface of the rock slope is frozen. However, the water which normally is free to flow out through the cracks in the rock during the summer period becomes frozen during the winter and greatly reduces the permeability of the surface of the rock slope. This can result in a build-up of very high pore water pressures behind the face. As a result, the shear strength of the rock diminishes rapidly and sudden failure of the rock may take place. During the winter months, the rock slopes are frequently covered with snow so that tension cracks, which normally warn of impending movement, may not be noticeable. It is very important that piezometers be installed and maintained to monitor pore water pressure changes.

(b) Development of seepage forces

When water flows through the rock, seepage forces develop as a result of the frictional resistance offered to the flow of the water. The force acts in the direction of flow of the water, i.e. into the open pit. Seepage forces can become very high. Coates and Brown quote an example where seepage forces were computed to increase the shear stress by 24 per cent[2]. Figure 1 illustrates seepage forces acting in a slide area.

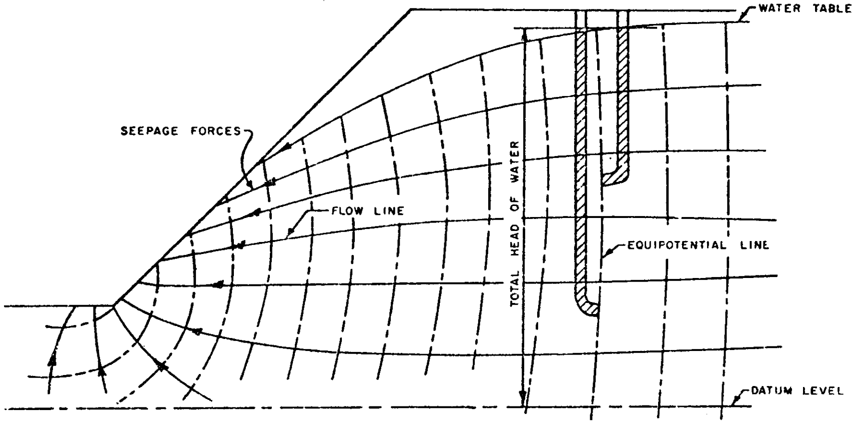


Figure 1. Two dimensional flow net illustrating seepage forces in a slope (Hoek and Pentz, 1968)[3].

Seepage forces and pore water pressures can occur in slopes where the rock surface appears to be dry. If the permeability of the rock is moderately low, it is possible that the rate of evaporation from the surface of the rock may exceed the rate of seepage. As a result, rock slopes which appear dry on the surface may experience high pore water pressures and high shear stresses at shallow depth. It is important to recognise that high pore water pressures can be developed by small amounts of water. The statement is frequently observed in the literature that the influence of pore water and seepage pressures on stability can be ignored where massive rock exists. This statement is theoretically valid. However, from a practical standpoint, there is usually insufficient evidence obtained on most projects to guarantee that unfavourably oriented discontinuities do not exist. Therefore, the consequences of failure would usually be sufficiently serious that extreme caution with this assumption is advised.

(c) Water in tension cracks

Tension cracks frequently develop at and around the top of slides. These cracks are normally treated as indicators of very low stability (i.e. a safety factor near 1.0). Should rain occur while these cracks are open, these openings may fill with water. This water creates a horizontal hydrostatic pressure on the sliding mass which further reduces stability.

In open pit slopes, tension cracks may extend to depths in excess of 1 to 20 meters so that hydrostatic pressures in these cracks can significantly influence stability. It is recommended when tension cracks are first observed, that movement measuring hubs be installed on either side of the cracks (Figure 2) and the cracks be filled immediately with low permeability material to prevent water from

Figure 2. Monitoring movement across tension cracks. The cracks should be filled with impervious material to prevent high water pressure buildup behind the slide.

entering and reducing the already borderline stability. A typical example of hydrostatic pressure in a tension crack is shown in Figure 3.

(d) Hydrodynamic shock due to blasting

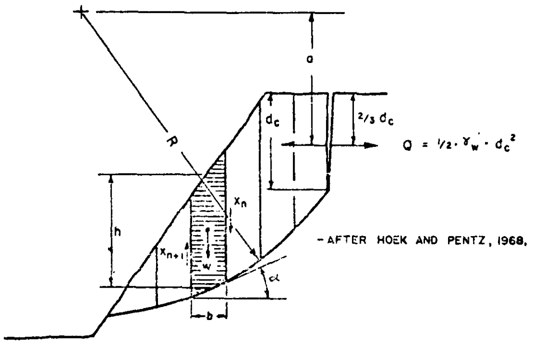
High short term pore water pressures may develop over a broad area in rock slopes during blasting if the rock is fully saturated. This occurrence reduces effective pressures and rock slope stability. The magnitude of this force can be reduced by using delays and limiting the amount of explosive per delay[4].

(e) Increase in weight

The weight of a rock mass increases as the elevation of the water table increases due to the weight of the water in the joints, discontinuities and voids in the rock. This additional weight slightly increases the shearing stress in the slope and therefore acts to reduce stability.

(f) Liquefaction

Fault zones in rock are occasionally filled with a rock flour-like gouge. If this gouge is in a loose density, is uniformly graded in silt or sand size range, and is completely saturated, there is danger that vibration due to earthquakes, blasting, or vibrating mining equipment may cause the gouge to liquefy. Failures due to this mechanism have been more commonly associated with failures of spoil dumps or tailings dams such as the El Cobre slides in Chile in 1966[5].



CIRCULAR ANALYSIS

$$F = \frac{R}{\sum (W \cdot R \cdot \sin\alpha) + Q \cdot a} \sum \left[\frac{c' \cdot b + \left[W(1 - u/\gamma h) + X_n - X_{n+1} \right] \tan\phi'}{\cos\alpha \left[1 + \frac{\tan\phi' \cdot \tan\alpha}{F} \right]} \right]$$

F = FACTOR OF SAFETY

c' = EFFECTIVE COHESION

u = NEUTRAL PRESSURE AT BASE OF SLICE

γ = UNIT WEIGHT OF SOIL

ϕ' = EFFECTIVE ANGLE OF INTERNAL FRICTION

Q for 20' deep tension crack = $1/2 \times 62.4 \times (20)^2$ = 12400 lb/lineal ft.

Q for 40' deep tension crack = $1/2 \times 62.4 \times (40)^2$ = 49920 lb/lineal ft.

Figure 3. Influence of water filled tension cracks on stability[3].

In addition to the adverse influence on stability, groundwater may have other detrimental effects. These include :

(i) Pit Drainage

Precipitation and seepage from the slopes and pit bottom may require expensive drainage control and pumping from the bottom of the pit. Pit drainage control may cost thousands of dollars annually.

(ii) Increase in Shipping Costs

On some projects the ore excavated from the pit is transported considerable distances. If the ore is moved in the saturated state, considerable water is transported and costs for hauling this water may be significant. Stubbins and Munro state that an extra 2 per cent moisture in the iron ore at Knob Lake, Quebec, increased transportation costs 12 cents per ton[6].

(iii) Drilling and Blasting in Wet Holes

Where excavation is required below the groundwater table and the bedrock is moderately pervious, blast holes may fill up very rapidly with groundwater. This requires more expensive explosives and blasting techniques and may result in caving prior to loading the hole. Blasting costs for wet holes may double those which are normally incurred for dry holes.

(iv) Increased Equipment Costs

During spring thaw or heavy rainfall, excess runoff and seepage water may collect on the floor of the pit and on haul roads causing them to become very muddy. This reduces the life of tires, tracks and brakes. In addition, hazards with electric cables are increased.

(v) Surface Erosion

Occasionally relatively large amounts of water will occur as springs from the rock slopes. If the rocks are badly fractured, soft or weathered, the flow of water may cause severe erosion on the slopes.

(vi) Ice Glaciers

In northern climates water issuing from the slopes may continue to freeze and build up ice glaciers. In the spring, during the thaw period, these glaciers can create a hazard and require special maintenance.

MEASUREMENT OF GROUNDWATER PRESSURES AND PERMEABILITY

Pore water pressures can have a great influence on stability and the volume of seepage water can greatly influence the economics of pit and mine drainage. It is therefore advisable to determine the general groundwater conditions prior to final feasibility studies. The hydrology engineer can provide the mining engineer with considerable assistance in this regard. Extensive experience has been gained in the field of soils engineering and groundwater studies performed in the investigation, design and construction of major dams around the world. Determination of pore water pressures in the general pit area is recommended. The simplest procedure is to install porous pot or equivalent piezometers. These can be installed in boreholes that are drilled to determine the location and the grade of ore. One piezometer can be installed in an AX size hole and up to three can be installed in an NX size borehole. Since it is not uncommon to find perched water tables in bedrock, two or three piezometers are recommended per hole. In order to install instrumentation in the best located boreholes, it is recommended that all exploration holes that are drilled be cased to bedrock and protected so they can be used at a later date if necessary. The depth to the water level in the piezometers is usually determined by lowering an electrical resistance probe. Where water pressure data is required at a great depth, it may be necessary to install special piezometers. The preferred type is the air piezometer such as that produced by SINCO of Seattle[7] (Figure 4). Increasing air pressure is applied from the surface against the pressure diaphragm until the air pressure reaches the pore water pressure and a reading is taken. An alternative is the electrical Maihak vibrating wire piezometer[8]. This instrument is suitable for short-term installation. On a long-term basis difficulties often develop with calibration and the instrument is easily damaged by nearby blasting. The disadvantages of the Maihak instrument appear to have been overcome in a prototype Solartron quartz-strut piezometer developed in England[3]. The vibrating wire has been replaced by a quartz-strut loaded in compression through a diaphragm upon which the water pressure acts.

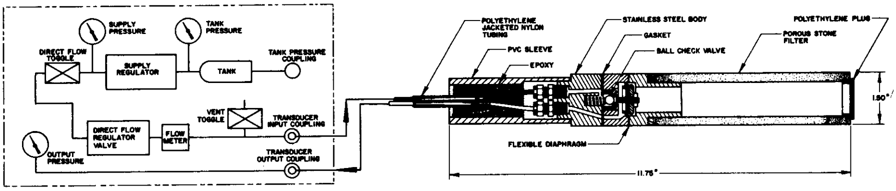


Figure 4. Slope Indicator Company air type pore pressure system with Model S1401 transducer.

Piezometers should be tested periodically to determine that they are operating accurately. Water levels should be read at least once per month, or following rain or thaw conditions, and the data compared with precipitation and temperature information. All piezometers installed outside the final pit should be established as permanent installations. To evaluate stability and potential drainage problems, it is also necessary to know the general permeability of the bedrock. In most rocks, more water flows through discontinuities than through the intact rock. In this case, laboratory tests on rock core will not provide useful permeability data. Pumping tests can be performed to determine average permeability. In these tests groundwater is pumped from a central hole or well and water level readings are measured continuously in observation holes located in several directions from the centre hole (Figure 5). This procedure is practical and reasonably economical for depths up to about 30 m. With the great depths encountered in most open pit mines this type of test has limited application. At the present time, the borehole water packer permeability test is the most suitable and economical technique to measure

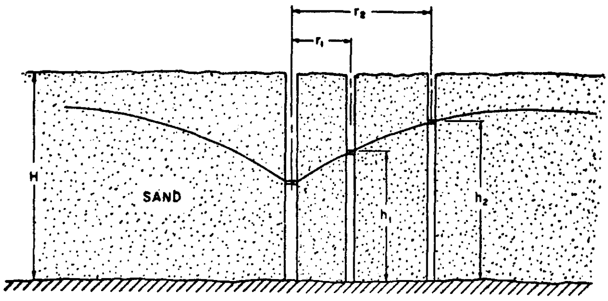


Figure 5. Pumping test layout to determine permeability in a uniform material.

the range or rock permeability. Usually a packer with a 3 to 6 meter spacing is installed in the borehole at any desired depth (Figure 6). Water is pumped into the hole between the packers under a moderate pressure and the volume of water forced into the rock is determined. The permeability profile for the entire borehole is determined. Using standard permeability formulae[9], the average permeability of the rock in the zone tested can be estimated. Rock permeability has been tested to a depth of 300 m in BX boreholes under the author's direction. Providing the exploration boreholes have been maintained open and can be used, 6 to 10 boreholes around the pit area can usually be tested to depths of 250 m in a one to two month period.

To obtain a rapid estimate of general permeability, falling head tests in boreholes can be performed[10]. This involves filling the borehole with water to the surface and measuring the rate of drop of the water level with time. Standard formulae for this type of analysis are shown in Figure 7. At the end of each drilling shift the casing should be filled to the top with water. The depth to the water level at the start of the next shift should be noted and the approximate permeability determined. On many mining projects adits are driven to check ore grades. By installing piezometers adjacent to the adit and monitoring the groundwater levels, the permeability can be evaluated.

Based on the permeability test results and the flow net, seepage volumes can be computed[12,9]. By analysing the field pore water pressure data and geometry of the flow net, cleft water pressures can be predicted for any slope geometry. The influence on stability can be estimated using the standard stability analysis employed in the field of soil mechanics[13]. Once a mining project develops, the plan which shows all blast holes should also show the depth of water in each blast hole. By developing a plan such as shown in Figure 8 the major

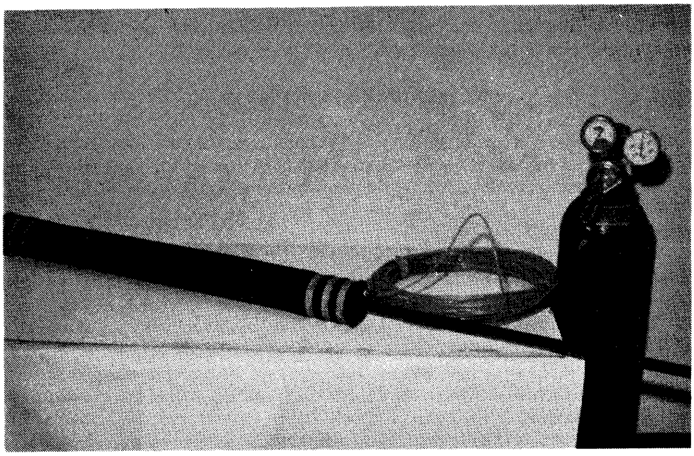
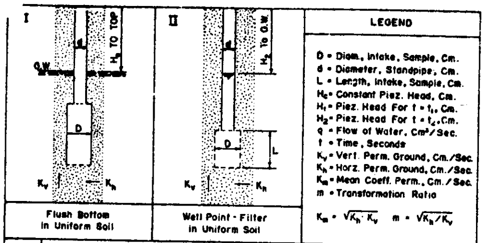


Figure 6. Single water packer with nitrogen bottle for packer expansion.



Case	Constant Head	Variable Head
I	$K_m = \frac{q}{2.75 \cdot D \cdot H_c}$	$K_m = \frac{\pi d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $K_m = \frac{\pi \cdot D}{11 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ For $d = D$
II	$K_h = \frac{q \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_c}$	$K_h = \frac{d^2 \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $K_h = \frac{d^2 \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ For $\frac{mL}{D} > 4$

Figure 7. Permeability determination in boreholes[10].

seepage zones where drainage will be beneficial will be evident. Drainage in dry areas is a waste of time and money.

CONTROL OF GROUNDWATER

If the evaluation of groundwater conditions indicates excessive inflow into the pit or adverse effects on stability, several procedures to improve conditions are available to the mining engineer.

(a) Horizontal Drains

One technique which may be utilised to improve the stability of the rock slopes is to install horizontal drains, a technique which is commonly used to stabilise earth landslides[14,15]. Holes 5 to 8 cm in diameter are drilled near the toe of the slope on about a 5 per cent grade for a distance of 50 to 100 meters into the slope. If the holes cave, a perforated drain must be installed. Typical installations are shown in Figure 9. To reduce drilling time it is common to fan 3 to 5 holes from one drill location. Groundwater flows into the drain holes, lowers the groundwater level and improves stability. During the cold winter weather in northern climates it may be necessary to protect the outlets of the drains from freezing and to collect the water with a frost free collector system. In the winter months in northern climates it is common for the pit slopes to freeze so that seepage does not exit from the slopes. As a result high pore water pressures frequently develop. This factor appears to account for the fact that many failures occur in the February to April period in Canada. An alternative to horizontal drainage to minimise the buildup of pore-water pressures in the slopes is to blast the entire lower bench 10 to 13 meters wide around the toe of the slope in the open pit and not to excavate this blasted toe during the winter months. This area will have high permeability and will act as a large drain in allowing water to seep from the slope. Water from this area must be collected in one or more sump areas and pumped from the pit.

(b) Drainage Adits

In some instances it may be practicable to construct an adit under the ore body and use it as a drainage gallery from which water is pumped or drained by gravity. For large volumes of water or for deep pits, drainage galleries at more than one elevation may be required. To increase the effectiveness of the drainage gallery, drill holes can be drilled on a fan pattern outward from the adit to increase the effective drainage diameter. Drainage adits have been used at Marcopper and Atlas in the Philippines, Similkamene Mining in Canada, Anamax Twin Buttes in the U.S.A. and the Deye Mine in China. It is recommended that the drains or adit be placed under a partial or complete vacuum. Recent research at Gibralter Mine, Canada showed a dramatic reduction in pore water pressure when the vacuum was applied. Drainage galleries may be particularly adaptable where open pits are located on steep mountain side slopes so that the adit may be drained by gravity.

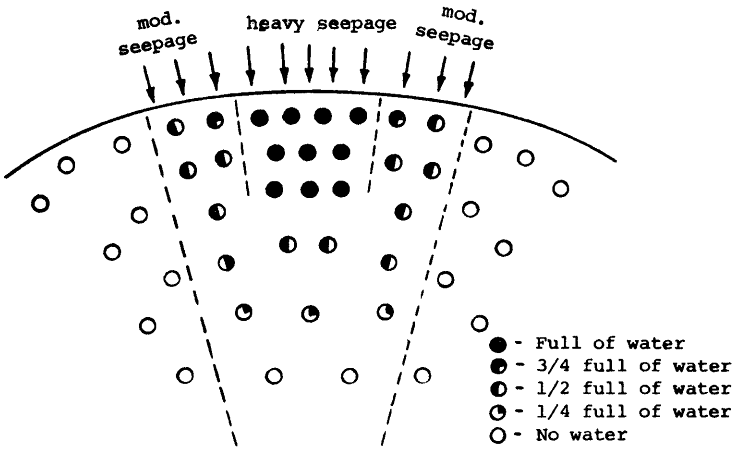


Figure 8. Blasting plan showing water in blastholes. Such a plan clearly indicates where slope drainage is required.

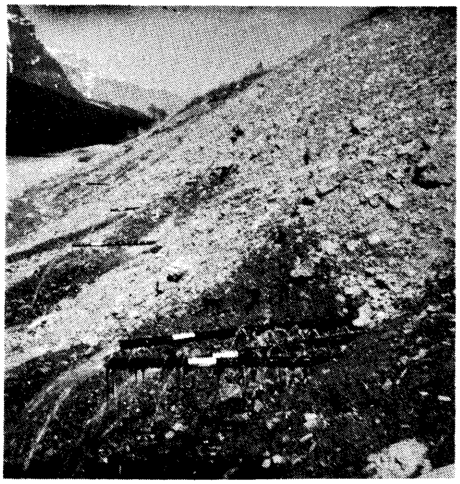


Figure 9. Typical horizontal drain installations.

(c) Wells

Where very heavy seepage is expected, pumping from deep wells located around the periphery of the pit may prove practical and economical. Facilities of this type have been installed in excess of 125 meters with success[6,16]. Where the groundwater flow is large and if the influence on stability of pore water pressures and seepage pressures is significant, the pumping system must be designed with reasonable over capacity so that if one pumping unit becomes inoperative there is sufficient excess of pumping capacity to prevent the development of local areas of high water pressure which might cause instability. In addition to drainage control within the pit itself, the control of surface drainage outside the pit boundaries is necessary to ensure that surface water does not flow into the pit. Besides the extra pumping capacity required, water flowing into the pit percolates into surface fractures and openings, many of which have been created by blasting, develops cleft water pressures and enhances rock breakup by freezing and thawing action. This aggravates rock falls and the occurrence of local slides between benches. Not only is it desirable to determine the influence of groundwater on stability but it will be desirable to determine whether drainage of the pit slopes will alow an increase in the overall slope angle. For the same safety factor, reducing pore pressures by 6 - 10 meters will usually allow an increase in slope angle approximating 3 to 6 degrees. An evaluation can be made of the cost of drainage versus the economical benefit to be gained by the increase of the slope angle that the drainage will allow. In order to evaluate the effectiveness of drainage it is necessary to install piezometers at key points in and around the pit to measure cleft water pressures. Normally it will be adequate to read instrumentation on a monthly basis, with more frequent readings during the spring runoff period, following heavy rains and during the late winter period.

As the open pit deepens, the probability of high pore water pressures developing in the base area of the pit increases and they could conceivably become sufficiently large to cause a blow up of the base of the pit. This probability increases where the bedrock structure is horizontal or if significant horizontal tectonic stress exists in the rock. To reduce the water pressures in the base of the pit, pressure relief wells should be considered[17]. The design of drainage control in open pit mines should always be preceded by a moderately detailed field permeability testing program unless extensive previous experience at the site is available.

(d) Blasting

Thin weak layers oriented to dip out of the slope frequently contribute to planer or block failures, particularly in sedimentary sequences. Selective controlled blasting may be used to reduce the continuity of the layers and reduce the pore water pressures due to the volume increase caused by the blast. The blast hole spacing should be about 3 to 5 meters to disrupt the weak layers. To develop pore pressure reduction only, the spacing can be increased to about 6 - 10 meters, depending on the rock type.

On the Syncrude tar sand project in Canada, blasting was used to improve stability for draglines to operate on highwall benches. Field instrumentation revealed sufficient drop in pore water pressure to increase the safety factor by 30%.

TYPICAL CASE HISTORIES OF GROUNDWATER EVALUATION

Three typical examples of groundwater evaluation are given.

Example No. 1

The present depth of the open pit is approximately 200 meters. Increasing amounts of seepage were observed with pit depth. The frequency of water filled blast holes was also increasing with depth. As a result it was desired to evaluate the amount of seepage and influence on stability of increasing cleft water pressure which would be expected as the pit depth gradually increased to 325 meters. Several holes which had been drilled two years previously were found to be open to depths up to 275 meters around the periphery of the pit. Water packer permeability tests with packers spaced 4 meters apart were performed at 15 meter intervals in each of the four boreholes. A typical summary of the data from one of the boreholes is shown on Figure 10. Rock permeability results ranged from 1×10^{-4} cm/sec to 1×10^{-9} cm/sec. The average value computed for all the tests was 1×10^{-6} cm/sec. For design purposes an average permeability of 1×10^{-5} was used. Flow nets for several different sections of the proposed pit at ultimate depth were drawn and the flow into the pit was estimated. It was computed that total seepage for a depth of pit of 325 meters would approximate 31 liters/sec. Based on the evaluation of rainfall records for the area, it was estimated that rainfall and snow melt could account for an additional 31 liters/sec. As a result a continuous pumping capacity of 65 liters/sec at a 325 meter head was recommended for drainage capacity of the ultimate pit. Piezometers were installed in the four boreholes following the permeability testing. These indicated that groundwater pressures would exist in the slope as the pit was deepened. The major concern for stability is during the winter months. For long term control it is proposed that the lowest bench be blasted and left unexcavated during the winter months to provide a major frost free toe drain to minimise tbe buildup of cleft water pressures. It is also proposed that a series of piezometers at 130 to 180 meter intervals be installed along the toe of the pit slope. These piezometers are installed in holes drilled with the standard rotary equipment.

Example No. 2

At a potential underground mine in eastern Canada there was concern that excess underground water may be encountered. A study was performed to estimate the rate of groundwater flow into the mine stopes. This required an estimate to be made of the permeability of the bedrock formations and the cleft water pressures in the area. The most practical type of test was found to be water packer permeability tests in diamond drill holes. An expanding packer was used, sealed off and water was pumped into the section under pressure. The rate of inflow of water into the hole was measured and the average value of permeability of the rock was computed. Water packer tests were performed at numerous depths in six boreholes and average permeability values were determined for these zones. The computed

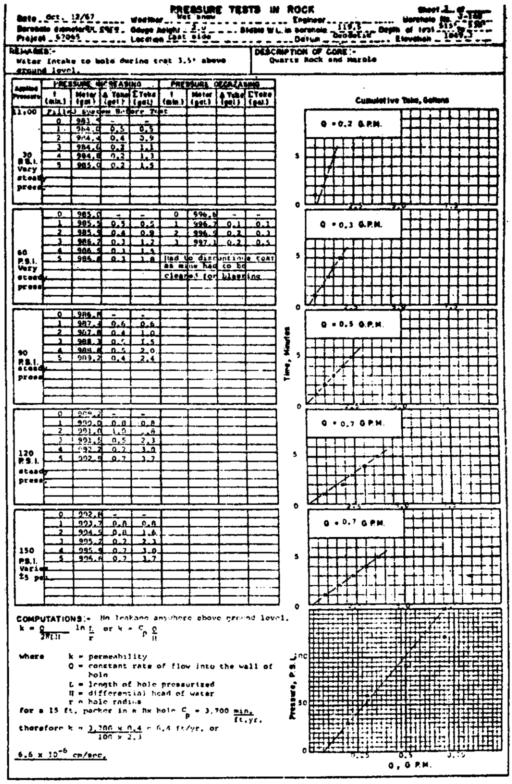


Figure 10. Typical record sheet for water packer permeability test.

coefficient of permeability varied from 3 x 10^{-4} cm/sec to 4 x 10^{-6} cm/sec. As a comparison, permeability tests were performed on samples of rock core in the laboratory. The rock core was, in general, 100 times less pervious than the field formations. This indicated that the majority of the groundwater flow would occur through fractures, fissures and joints in the rock. Using a flow net analysis and the permeability values obtained from the field testing program it was estimated that seepage into the stopes would range from about 3 to 45 litres/min. per meter width of stope. The volume of water encountered during mining was within 25 per cent of that predicted.

Example No. 3

At a major open pit coal mine, block slides have extended well beyond the crest of the slopes. One failure was developing where the slope was 150 meters high with an average face slope of 3 horizontal and 1 vertical. Several piezometers were installed and indicated a piezo-

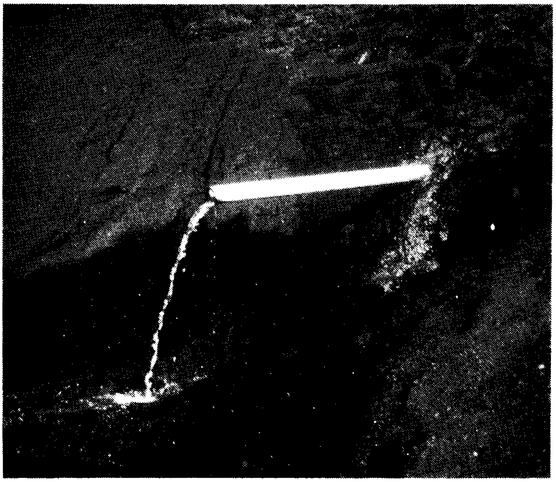


Figure 11. Horizontal drain 250 meters long to reduce pore water pressure in the slope at a major coal mine.

meter head approximately one half the height of the slope. Underlying the coal was an overconsolidated clay with an effective angle of friction of about 12^{o}. It was not practical to modify the slope angle or construct a beam to control the slide. Stability analysis indicated that by lowering the pore water pressures with horizontal drains, stability could be obtained. The drains were installed to depths up to 250 meters. The drains have also been effective in reducing water pressure in tension cracks that developed from stress relief due to coal removal. Figure 11 shows a typical horizontal drain that was installed.

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