OPEN-PIT MINING AND EQUIPMENT MAINTENANCE

Open-Pit Dewatering at Pine Point Mines

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ABSTRACT

As open-pit operations expanded at Pine Point, hydraulic aquifer evaluation was undertaken to establish the criteria necessary for the design of a feasible dewatering system.

The extreme climate of this northern location dictated that the ore zone under extraction be rendered as dry as possible to minimize freezing effects. The design and implementation of such a system necessitated detailed hydraulic evaluation, and the problems that occurred are typical of those inherent in open-pit mining operations in northern areas.

The paper describes the problem solution as approached from a mine engineering point of view, with particular reference to the practical aspects of operation and installation. The hydrological analysis used is simple and readily applicable to most open-pit dewatering problems.

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THE PAPER WAS PRESENTED: at the Annual General Meeting of the Institute, Montreal, April, 1969.

KEYWORDS: Open-pit mining, Dewatering, Pumping, Pine Point Mines, Aquifer evaluation, Drawdown tests, Water flow, Hydrology.

C.I.M. TRANSACTIONS: Vol. LXXII, pp. 341-347, 1969.

INTRODUCTION

THIS REPORT is not intended to be a definitive text on hydrologic procedures, but rather a description of the dewatering operations at the Pine Point mine. It may aid the mining engineer in solving a similar problem, particularly with regard to the practical aspects involved. There are numerous analytical and test approaches that could solve a similar problem; this is a description of one such method successfully applied.

The Pine Point mine, owned by Pine Point Mines Limited and operated by Cominco Ltd., is located on the southern shore of Great Slave Lake, about 750 road miles north of Edmonton, Alberta. The area is flat and swampy, covered with scrub-like and stunted spruce and pine vegetation. The climate is classed as semi-arid, with a prolonged winter season of sub-zero temperatures and a short, dry summer season.

GENERAL

As the mining sequence at Pine Point progresses, a large percentage of the production is scheduled to come from below the water table. To facilitate mining below the water table, it was decided to conduct a detailed hydraulic evaluation of the ground-water conditions to establish the design parameters requisite for a dewatering system. An aquifer drawdown testing program was set up in 1967 to evaluate these characteristics.

A brief résumé of the geologic and ground-water conditions may help in appreciating the problem presented. The aquifer consists of a sequence of coarsely re-crystallized, vuggy to cavernous, middle Devonian dolomite and limestone, randomly interspersed, with a series of discontinuous argillaceous layers. The favourable ore horizon comprises a series of lenticular deposits of varying thickness, at depths of up to 300 feet below surface, spread over an area of roughly 25 by 10 miles. The pre-pumping water level varies from 20 to 120 feet below surface, with suspected eventual discharge into Great Slave Lake some 6 miles away. (H_2O Level) Ground-water movement is mainly influenced by localized fractures and solution openings, with recharge primarily from snowmelt and rainfall. The average annual precipitation is 13 inches, representing a possible recharge of about 620,000 (U.S.) gallons per day per square mile. This is a low recharge in terms of gallons per minute, and is therefore considered to be favourable for dewatering operations. (recharge) An estimate has been made that only 25 per cent of the total precipita-

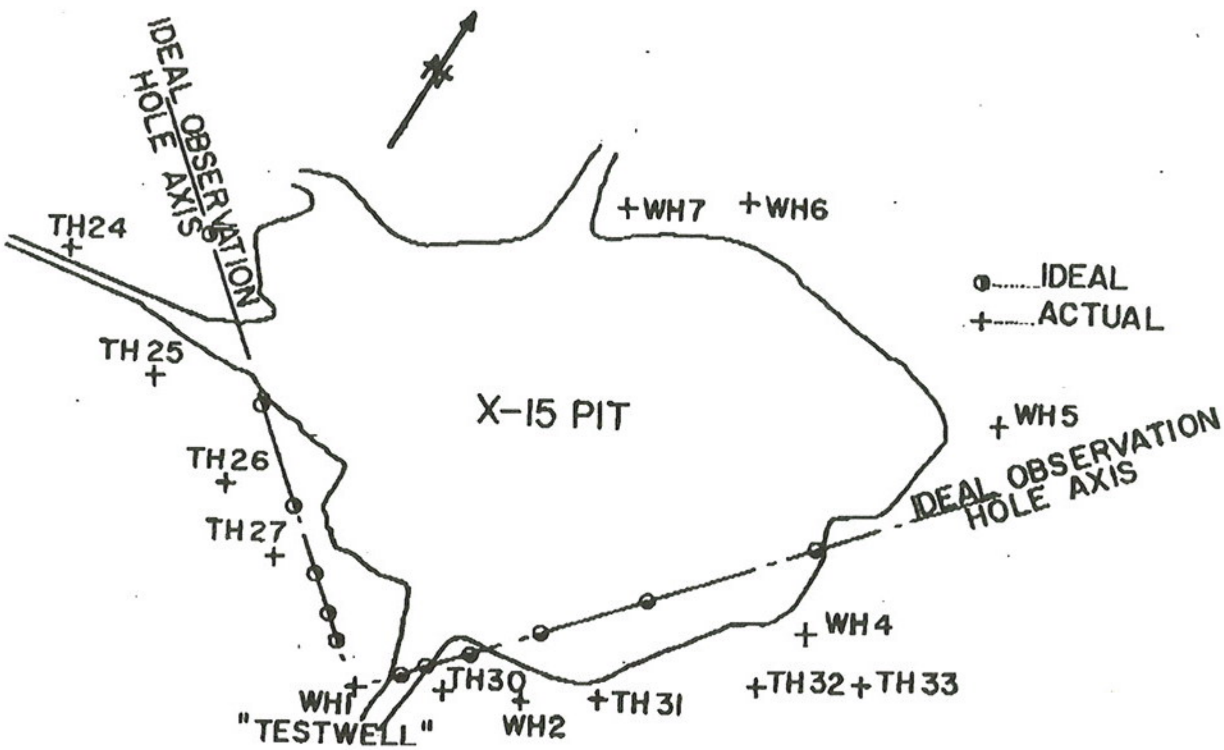

Figure 1.—Idealized and Actual Test Patterns.
Actual pattern utilizes both diamond-drill and well holes (i.e. TH and WH holes).

tion actually recharges the aquifer; the remainder being lost through run-off, evaporation and transpiration. This will be established more accurately after several years' data have been analysed.

The decision to run an aquifer drawdown test was based on the fact that all previous ground-water data had indicated greatly variable conditions, with no obvious uniformity of results. Basic to a quantitative analysis of an aquifer, a number of assumptions were made: the aquifer was considered to be ideal, i.e. isotropic, homogeneous, of infinite areal extent and uniform in thickness. Inevitably, such conditions do not exist; however, the Pine Point aquifer is considered to be sufficiently uniform to supply useful data from a controlled drawdown test.

TEST PATTERN

An aquifer drawdown test requires the establishment of a test well and observation-hole pattern designed to give the type and quantity of results suited to the analysis used. At Pine Point, the analytical method used was that evolved by Theis, Cooper and Jacob[1,2,3,4], based on the non-equilibrium equation, with check calculations using the drawdown-distance relationship suggested by Theim[6]. These methods require a test involving a centrally placed test well and an observation-hole pattern covering all four quadrants and spaced on logarithmic centres away from the well (see *Figure 1*). Ideally, the test well should be at the pit centre to simulate the prevalent hydraulic conditions as closely as possible. However, mining operations and practical limitations generally require that well holes be on the pit periphery.

The high diamond-drill-hole density at Pine Point permitted the utilization of a number of the old core holes as observation points. It was accepted that the locations depart from the ideal layout. Perforated casing was sometimes required in incompetent ground, although its use is not advisable due to hydraulic distortion effects.

The well holes were drilled 17 inches in diameter through the overburden and 12¼ inches in diameter in rock to a depth of 100 to 150 feet below the ore footwall. This was necessary to guarantee sufficient aquifer penetration for adequate yield, and to minimize the possibilities of surging and cavitation. The well holes were drilled with a production blasthole rotary drill or with a rotary oil-well rig, using mud to about 300 feet and compressed air at depths greater than 300 feet to assist in cleaning the hole. Drilling problems were minimal in bedrock, but the unconsolidated section often proved troublesome due to sidewall caving. The well holes were cased through the overburden, with the casing well seated into bedrock. The decision not to install perforated casing in rock is a calculated risk; we derive minimum frictional effects on the aquifer without installing casing, but accept the possibilities of caving.

In practice, test patterns consist of a single well hole, located on the rim, with at least ten observation holes. The number of observation holes is variable, but, in our experience, ten are sufficient to provide the accuracy and quantity of results required. *Figure 1*, showing the test pattern employed at the X-15 pit, illustrates the deviation from the ideal two-axis, logarithmic spacing. However, although the holes are erratically spaced, the over-all area coverage was good and the results proved suprisingly uniform, allowing for easy analysis and engineering design. This particular pit, and the associated test and design, will be referred to throughout this paper as a typical Pine Point dewatering project.

TEST PROCEDURES

Once a suitable test pattern was established and the pump was operating satisfactorily, preliminary readings were taken for the commencement of the aquifer drawdown test. For hydraulic analysis, it was required to:

— measure point elevations and co-ordinates
— pre-test static water levels
— measure the test-well discharge rate
— measure dynamic water levels at known time intervals.

To obtain these readings, a test timetable, trained personnel and some relatively inexpensive instruments were necessary.

Test-well discharge rates were measured using an open-ended orifice plate on the pump discharge pipe in conjunction with a manometer tube and a gate valve. This method of flow-rate measurement is extremely sensitive, with a ½-in. change in manometer height representing 6 (US) gpm for a 5-in. orifice in a 6-in. pipe. It has proved far easier to maintain than an incline orifice or weir arrangement. The calculation formula is:

$$Q = \sqrt{kA}\ 2gh$$

where: Q = discharge, in (U.S.) gpm
A = area of orifice, in square inches
g = acceleration due to gravity, in feet per second^2
h = height of water in manometer tube, in inches
k = experimentally established constant dependent on the ratio of pipe diameter to orifice diameter (viz. Layne & Bowler[6]).

The over-all arrangement is illustrated in *Figure 2* and this arrangement is standard on any pit pumping installation. It is necessary because flow-rate regulation and observation are requisite for both mechanical and design reasons. This practice may be changed as the pumping procedures become more routine.

The measure-point elevations and co-ordinates are established by routine survey procedures, and closed traverses are advisable. Accuracy is necessary, as all test readings are taken to the nearest 1/100th of a foot.

The static water levels are established (on a twice-daily basis) over a period of at least one week prior to the pumping of the test well. It is advisable that, during the test period, no pumping, blasting or well-hole drilling should be carried on within a 2,000-ft radius of the test well. If this condition cannot be met, checks are maintained throughout to establish the time and severity of such operations, and corrections are made at a later date. For example, as the mill-site water supply wells are discharging continuously, all that can be done is to maintain their total discharge at a constant figure throughout the test period. These effects can create definite ground-water trends; e.g. drawdown was observed 1,500 ft away from a well hole being blown with compressed air. It is stressed that maximum control during an aquifer test is of prime importance.

Water-level elevation changes are monitored at the well hole and at all observation holes, with the density timed on a logarithmic basis[1]; i.e., a high frequency of measurements to begin with. To gather this information requires a large personnel commitment at the start of the test. This commitment is steadily decreased as the test advances, because lower observation frequencies will allow one man to cover several points.

The instruments used are either Fisher M-scopes or Stevens Type F automatic recorders. The M-scope is a simply constructed recorder based on the principle of a lead probe completing an electrical circuit at the water-table contact and causing an ammeter deflection. Although a fairly robust piece of equipment, it must be handled with care. Rough handling can cause erroneous readings, and it has been our experience that periodic overhauls during a test period are necessary. Slipping of the footage markers on the cable, short circuiting and probe damage are the most common causes of incorrect measurements. Accurate calibration and frequent maintenance are therefore imperative.

The Stevens Type F automatic recorder is superior in accuracy and reliability, although correspondingly more expensive. It is a mechanically driven instrument working on the principle of a counterbalanced float rotating a cylindrical drum in conjunction with a clock-drive pen trace. The obvious extra advantage is a saving in labour, as it may be left unattended. Unfortunately, hole size is limited in that a 5-in.-diameter hole is about the smallest size able to accommodate the the instrument. Nevertheless, a couple of these instruments somewhere in the test pattern are invaluable in helping to eliminate anomalous M-scope readings. The Stevens recorders also provide an excellent evaluation of seasonal trends. Both instruments are sketched in *Figure 3*.

An aquifer test yields several series of data for the valid aquifer characteristic interpretation required in the designing of a practical dewatering system. The Pine Point testing to date has resulted in the design and successful implementation of several dewatering systems. However, the test data and conclusions are continually re-examined so that new developments and interpretations can be incorporated into predictions.

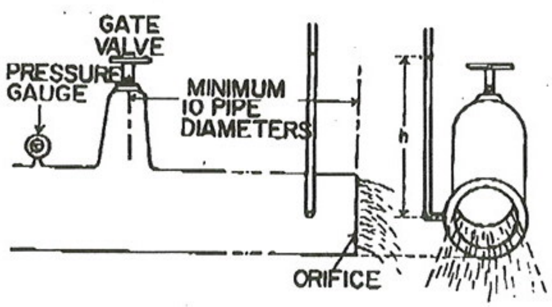

Figure 2.—Discharge Measurement.

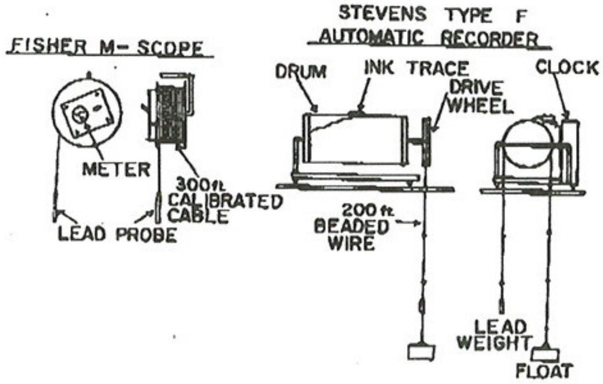

Figure 3.—Recorders.

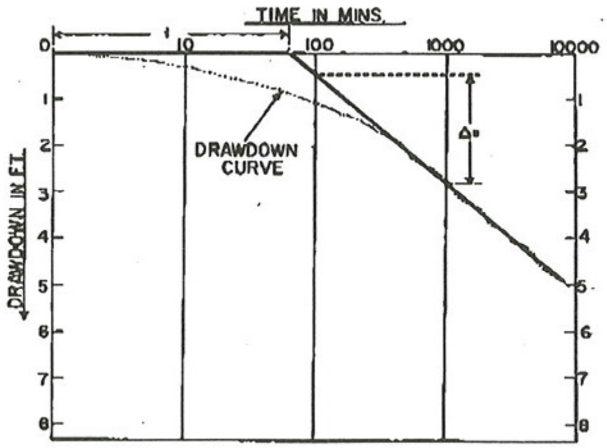

Figure 4.—Drawdown-Time Curve.

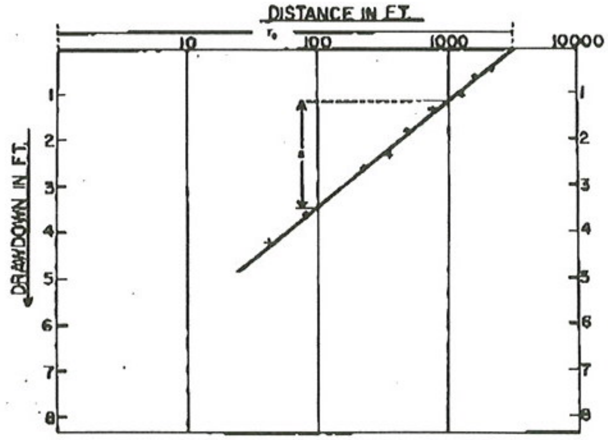

Figure 5.—Drawdown-Distance Curve.

As new pits are planned, additional testing will be done to assist in designing the system.

As in any kind of test or data-gathering operation, properly trained personnel and close supervision are essential.

DESIGN PARAMETER CALCULATION

The evaluation methods employed at Pine Point comprise well-established hydrologic procedures.[1]

From the test data, a list showing the drawdown achieved against time is tabulated for each observation point related to a given single well-hole discharge. A semi-logarithmic graphical plot is then made for drawdown versus time (one graph for each observation hole), similar to the graph in *Figure 4*. Drawing a tangent to the curve in its final straight-line approximation and substituting in the formula:

$$T = \frac{264 \times Q}{\Delta s} \quad (1)$$

gives a value of T, the aquifer transmissibility, in gallons per day per foot where:

Q = test well discharge in (U.S.) gpm
s = drawdown per log cycle in the final straight-line condition.

The inflection of the curve indicates a decrease in the effective value of T, due to the drawdown cone impinging on localized boundary conditions. From T, a value of S, the storage coefficient, is calculated using the formula:

$$S = \frac{2.08 \times 10^{-4} \times T \times t}{r^2} \quad (2)$$

where:
T = calculated transmissibility
t = intercept on the time axis, in minutes (*Figure 4*)
r = distance, in feet, from the pumping source.

Thus, a series of values for T, the transmissibility, defined as "the number of gallons of water at 60°F passing through each mile width of the water-bearing bed (measured at right angles to the flow) for each foot of the thickness of the bed and for each foot per mile of hydraulic gradient," and for S, the storage coefficient, defined as "the volume of water that an aquifer releases from or takes into storage per unit surface area of aquifer per unit change in the component head normal to the surface," are calculated for the test pit. These values, when averaged, provide the two basic hydraulic characteristics employed as design parameters.

It should be mentioned that, in order to know the necessary duration of any test, a rough plot of drawdown versus log time should be maintained in the field (on one hole only) to ensure that the final "equilibrium" condition has been established prior to stopping the test. By equilibrium, it is inferred that, for a given increment of time, the drawdown at any point within the radius of the cone of influence will be identical. This means that the boundary conditions pertinent to the area have become evident in the drawdown cycle (as shown by the straight-line condition on the graph). If further boundary conditions were reached, the curve would bend again, as in *Figure 4*.

As a check on the T and S coefficients, a graph is plotted of drawdown versus log distance *(Figure 5)*. The drawdowns are those achieved at each observation point at a time t_1, after the pump was started, an arbitrary time during the equilibrium stage of the drawdown cycle. The distances are those to the pumping source. There is only one graph for the whole system. Employing the following two formulae will give check values of T and S:

$$T = \frac{527.7 \times Q}{s} \quad (3)$$

$$S = \frac{0.3 \times t_1 \times T}{r_o^2} \quad (4)$$

where:
r_o = intercept on distance axis (*Figure 5*).

To illustrate peculiarities often evident on the graphs, *Figures 6* and *7* show a few unusual traits recurrent at Pine Point:

1. — Irregularity 'a' indicates a very rapid and strong aquifer recharge, possibly due to a severe rainstorm.
2. — Irregularity 'b' is probably due to the drawdown cone reaching out to a high-pressure water-bearing area, again giving strong recharge.
3. — Irregularity 'c' shows a similar area running dry, after its source has been cut off by the advancing drawdown cone.
4. — Irregularity 'd' indicates a boundary condition showing up as the drawdown cone expands. This could be due to a fault, a sinkhole or an intrusion.

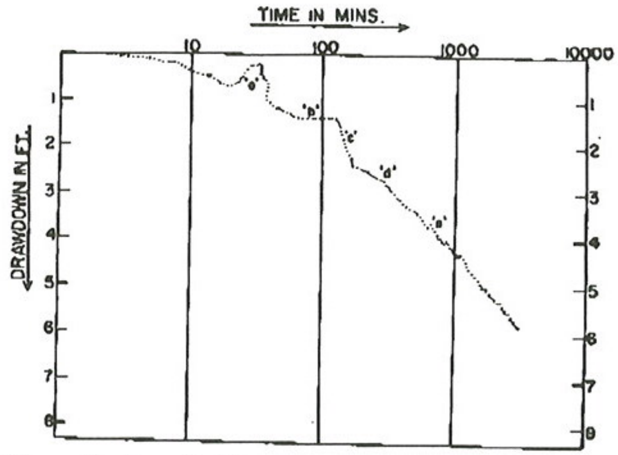

Figure 6.—Irregularities.

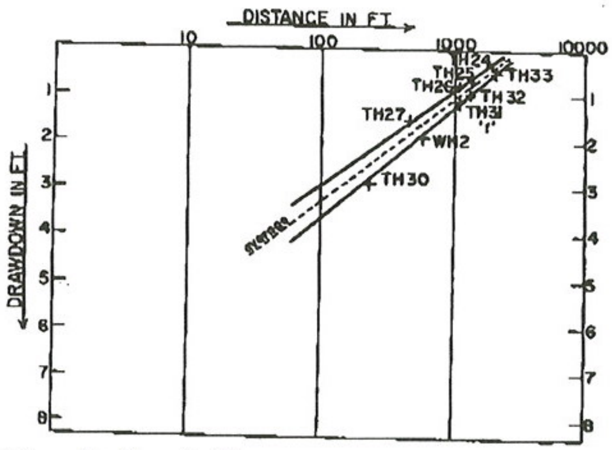

Figure 7.—Irregularities.

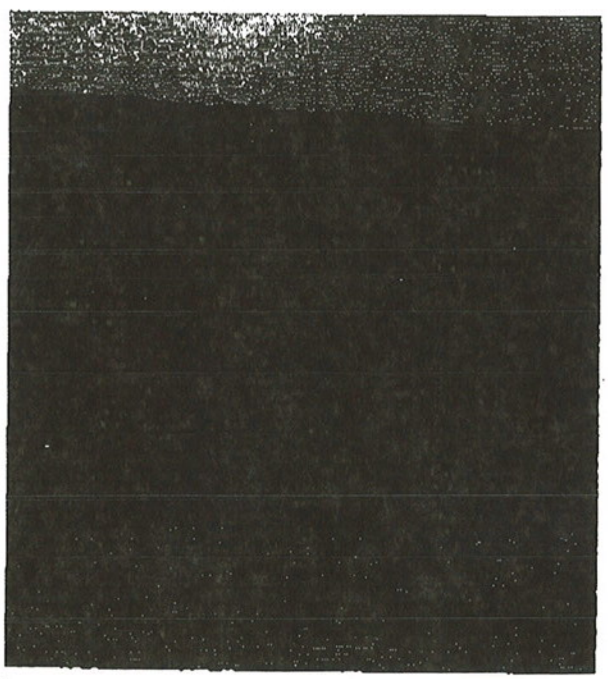

Figure 8.—Discharge line from N-42 test pump in pit bottom.

5. — Irregularity 'e' implies a regular seasonal recharge, probably due to steady overburden seepage from snowmelt.
6. — Irregularity 'f' *(Figure 7)* shows two straight lines which illustrate the different effective transmissibilities of the two axes divergent from the well hole (see plan in *Figure 1*). The average value is that pertaining to the total pit under study.

Numerous other curve variations can result, and a comparison with geological conditions often supplies useful information when finalizing well-hole locations relative to minimizing poor-yield wells and capitalizing on image effects.

DESIGN PROCEDURE

Before attempting theoretical design, it was found advisable to establish the design format. Pine Point's northern location inevitably brings up the question of the effects of the sub-zero winter climate. The immediate thought was to investigate the practicality of pumping from within the pit using long discharge lines, as shown in *Figure 8*. One power failure, and the resultant freeze-up, was enough reason to keep the discharge-line lengths to a minimum.

This, together with the danger of damage from flyrock and the possible shock-wave damage to well holes, dictated that pumping installations would be more practically situated beyond the pit rim. It was accepted that an increase in capacity would be required due to the increased distance from the pit node. Deep-well pumps with short discharge lines were chosen over other types. Operational requirements and climatic conditions precluded the use of sump pumps, except during clean-up phases or for dewatering isolated pockets in pit bottoms.

Having decided that the use of deep-well pumps on the pit periphery was the solution, it had to be established how far back from the pit rim they could be located. The considerations of overburden slope failure and instability during the spring thaw led to the establishment of a 90-foot minimum distance from the rim to the well site to guarantee permanency. This figure was based on experience rather than on detailed soil-failure analysis. The great variety in composition and thickness of the overburden would have necessitated considerable research for a more reliable figure.

The siting of drainage ditches and power lines relative to the well hole created considerable discussion relative to installation and maintenance advantages; and initially resulted in the power line being placed on the extreme outside to guarantee its permanency. The sketch in *Figure 9* illustrates an idealized pit-rim section, with approximate dimensions. However, it was

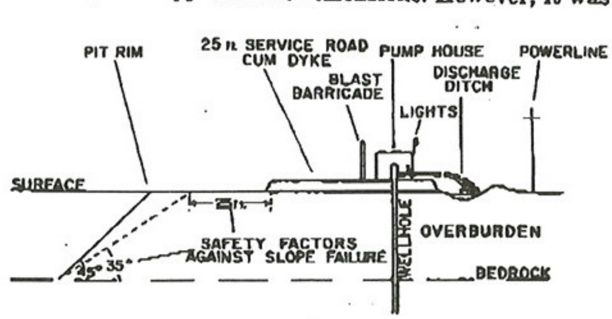

Figure 9.—Idealized Pit Rim Section.

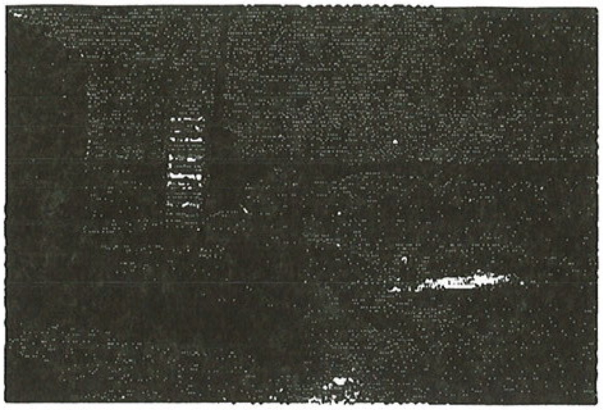

Figure 10.—Pump, ditch relationship.

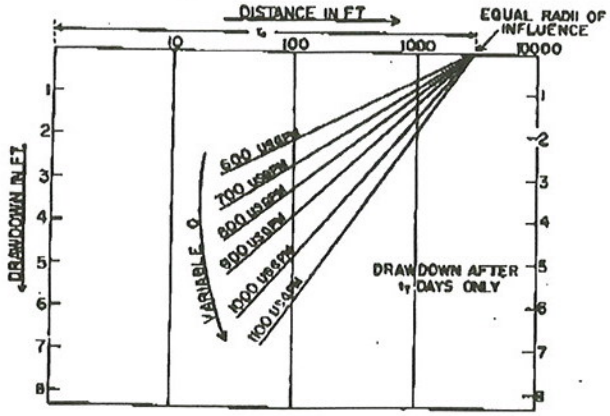

Figure 11.—Design Curves.

later decided to place the ditch to the outside of the power line to minimize access problems to the transformers and to avoid the stringing of cables over the ditch. *Figure 10* shows the present relationships of the pump, power line and ditch.

After referring to the established design parameters of T and S, and the graph of drawdown versus log distance (*Figure 5*), the design stage of the operation was relatively simple. In the formula:

$$T = \frac{527.7 \times Q}{s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

Q was varied to give a series of values of s, using the calculated average of T. A family of curves was then drawn (*Figure 11*), radiating from the same distance axis intercept, giving the expected drawdown achieved by a single well hole, discharging with a known flow rate, at any distance from the well hole within the radius of influence after time t_e (equilibrium stage), in days. Consideration was then given to the pit which was to be dewatered (the final rim and ore-depth design outlines being known). A reasonable estimate of the number of well holes required was made, and a plan was prepared to show them equally spaced around the pit circumference. A "reasonable estimate" is admittedly rather arbitrary; however, a poor guess becomes immediately obvious. It is valid to say that the drawdown at the centre of a pit, being dewatered at a discharge rate equal to the sum of the individual pump discharges, is equal to the sum of the individual drawdowns. However, this drawdown is only that gained after t_e days (equilibrium), whereas the drawdown achieved after n months is the required figure.

Therefore, re-drawing the graph of drawdown versus log time (days) for the whole system, where Q = total system discharge and the initial point on the graph is known from the summation of the original drawdowns, it was possible, by comparing forecast requirements with drawdowns achieved by the system within that time, to see if the number of pumps, i.e. the total discharge, was adequate. If not, the process was repeated by trial and error until the correct design solution was established. This is an ideal computer exercise.

This is the basic method of design employed at Pine Point and, apart from the incorporated safety factors, it is relatively simple. It has been the practice to dewater the 7-foot overdrill below the drill depth on any given bench to minimize the use of high-cost, water-resistant blasting agents and to minimize the effects of power outages. This safety factor is extremely valuable in the event of an unscheduled power outage, as the recharge in the first 24 hours is the mirror image of the first 24-hour drawdown.

After a design has been presented, the total discharge figure recommended is broken down into the number of wells considered to be able to supply this discharge. These wells are then located in practical positions around the pit perimeter. The expected individual discharge rates are based on the variation in the storage coefficient figures, using the well hole as the datum comparison. Three pump sizes are kept on hand, due to the quite considerable flow-rate variations encountered. The peripheral spacing is ideally constant; however, the high incidence of sinkholes has often dictated considerable reshuffling of the well-hole locations.

INSTALLATION PROBLEMS

Once established and agreed upon, the installation of the system proved to be a relatively simple matter; however, as with all new equipment and techniques, problems of various types inevitably arose.

Pump houses were found to be necessary to protect the pumps and prevent freezing in winter. The addition of a roof sling or hook to the pump houses facilitated their installation and movement. Periodic power bumps, blast damage or mechanical failure necessitated frequent checks on the pumps. To aid in a shift-by-shift check, coloured lights are mounted on the pump-house roof, designating the status of the pump at any given moment; i.e., red — okay, green — timing device, orange — maintenance, no light — problems. A recurrent problem in our initial installation period was that the pumps were mysteriously tripping out. It was eventually realized that, due to the lack of ventilation within the pump houses, the ambient temperature was becoming too high, causing the pump to trip out. The solution was to install ventilation panels in the shacks. It was found expedient to minimize the peak load factor, so the pumps were put on a delayed start-up basis at two-minute intervals.

The pump installation and well-hole preparation program have not provided any serious problems to date. Flyrock damage was minimized by the installation of 8- by 8-in. barricades around all the major pumping equipment. The drainage ditches have been a source of concern because of the possibility of recharge back into the pit and, although dye tests have proved negative, half-culvert, light-wall pipe or plastic piping is being used in some potentially troublesome areas. Bentonite was tried as a ditch sealant, but this was inconclusive.

The important factor in design estimation is the total discharge attained by the system. Our experience in pit X-15, for example, is that some 25 per cent of the well holes drilled are not too productive; some producing as little as 100 (US)gpm. Mechanical on/off pump surging has improved some of the wells, whereas others have required drilling with mud-dispersing agents. However, 100 per cent development success has not been achieved, and other solutions are under consideration. The basic causes of the low yield are not yet fully appreciated, although they probably involve a combination of a retaining mud seal, skin damage and the low inherent transmissibility. Consultation has begun in an attempt to improve the over-all average, and the indications so far are that pressurized acid injection may prove to be the cheapest solution. Either way, investigations should reveal that extensive development is economic or that the problem is one of having to drill more well holes.

As a result of experience with the variability of the discharge rates achieved, design practice tends toward a staged pumping solution. Seventy-five per cent of the estimated total design capacity is installed and then, when operational, re-evaluated based on actual discharges and not theoretical estimates. This could give a higher total discharge requirement, due to the time factor involved, but it allows advantage to be taken of discharges that are in excess of original estimation and could result in a smaller number of installations. As pumping requirements can result in large capital expenditures and high operating costs, it is common practice to expedite the work in a pit being dewatered so that the pit can be completed in as short a time as practical. This will release pumps and power for duty in other pits.

Table I—Pumping Costs

	Total	$/Ton Dewatered
(A) CAPITAL COSTS:		
Pumps and switchgear ..	$222,600	$0.113
Substations	75,200	0.038
Power lines	26,000	0.013
Pit ring lines	4,000	0.002
Pump houses	12,500	0.006
Casing, pipe	25,000	0.013
Misc.	15,930	0.008
TOTAL	$381,230	$0.193
(B) 1968 OPERATING COSTS:		
Operating — labour	$ 22,600	$0.011
— supplies ..	89,200	0.045
Repair labour	45,300	0.023
Power	40,200	0.020
Other charges, i.e. equipment rental	33,000	0.017
Sub-total	230,300	0.116
Well-hole drilling	132,200	0.067
Test-hole drilling, consultant fees, etc.	34,600	0.018
TOTAL	$397,100	$0.201

Note:

(i) Ditching cost of approximately $370 per 1,000 ft of ditch excavated is included in operating and other charges.

(ii) Access road costs, including pump pad construction, are also included in operating and other charges. Costs are $890 per 1,000 ft if access is built from ditch material, and $3,000 per 1,000 ft if access is built from pit waste material.

DIRECT PUMPING COSTS

The costs (Table I) are spread over 1,972,000 tons of ore which had to be dewatered in N-42, 0-42 and J-44 pits. These costs are higher than have been experienced in X-15 pit or anticipated in future pit dewatering systems. This is due to differences in pit size, well-hole productivity and the time available for dewatering to the required levels.

RESULTS

The testing, installation and maintenance functions are all standard procedures now, designed to increase the efficiency of the pumping set-up and to establish it as a routine operation.

Automatic recorder and M-scope monitoring of the four pits being dewatered has shown that drawdown is maintaining a very good approximation to the theoretical forecast rates. This is most satisfactory, and it is hoped that the accuracy of forecast can be maintained. Comprehensive drawdown rates and figures are continually updated, so that, in the event of a miscalculation, the necessary corrective measures can be readily evaluated and enacted upon.

As to the future problems — artesian water will be experienced in some areas. A detailed aquifer drawdown test should supply the necessary solution. The mill-site water supply poses a question mark in that excessive drawdown could cut off or at least decrease the supply available due to the decreased aquifer thickness. Monitoring and study are maintained to keep a check on any radical abnormalities.

Reviewing the over-all approach to the dewatering problem, probably the most obvious procedural error involved the time spent in feeling our own way into the problem without first consulting experts. It should be stressed that, as with most semi-technical and technical problems in the mining industry, someone somewhere has had a similar problem; and that people with knowledge of the problem do exist. In this context, Cominco would like to extend their appreciation to Messrs. Brashears, Leggette, and Graham, consulting groundwater hydrologists of New York, who provided invaluable guidance in helping us solve the Pine Point dewatering problem.

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