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Civil, Geotechnical and Materials Engineers

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117 Industrial Road
Whitehorse, Yukon
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EBA file: 0201-10401

ATTENTION: Mr. Greg Jillson

SUBJECT: Preliminary Report
Dy Deposit Hydrology Assessment
Faro, Yukon

Dear Sir:

As has been requested by your field representative, Mr. John Zbeetnoff, a preliminary report encompassing the results and findings from Hydrological Testing of the Faro Dy Deposit during October and November of 1990 has been attached.

The purpose of this report is to provide yourself with sufficient information to plan further hydrological testing programs as may be required to develop the Dy Deposit. A comprehensive report of the Dy deposit Hydrology will be provided at a latter date.

The attached preliminary report consists of two sections. The first section contains an overview of field activities performed by EBA personnel. This section includes the presentation and interpretation of field testing data as well as a discussion of its limitations. The second section is a preliminary analysis of the estimated seepage inflows for envisioned workings required to develop the Dy deposit.

We hope this preliminary assessment provides sufficient information for assessing and planning further hydrogeologic investigation requirements. It is particularily important that a decision as to whether to conduct further testing is made as soon as possible in order to avoid unnecessary rental costs for the packer equipment. Should you have any questions or require additional detail please contact either Mr. Cord Hamilton at 668-3068 or Mr. Scott Sylte at 604-685-0275.

Respectfully Submitted,

Cord R. Hamilton, E.I.T.

Scott Sylte, P. Eng.

PRELIMINARY REPORT
DY DEPOSIT HYDROGEOLOGY TESTING
FARO, YUKON

1.0 REVIEW OF PERMEABILITY TESTING

1.1 Introduction

Permeability testing of the Dy deposit in Faro, Yukon was conducted over the period of October 26 to November 11, 1990 at the request of Mr. Greg Jillson of Curragh Resources Limited. Authorization to proceed with the testing program was issued by Mr. John Zbeetnoff, Project Geologist for the Dy deposit development.

The scope of the project consisted of performing permeability testing in drillholes 90DY01, 02, 03, 04, and 05. Testing in drillholes 90DY01, 02, and 03 would be falling head permeability tests conducted in piezometers previously installed by EBA in August, 1990. Permeability testing in drillholes 90DY04 and 90DY05 would be constant head permeability tests conducted by utilizing packer testing equipment. The number and location of the packer tests were to be determined by Mr. John Zbeetnoff during the field operation.

EBA personnel utilized for the project consisted of Mr. Scott Sytle from Vancouver, B.C. and Mr. Cord Hamilton of Whitehorse, Yukon.

The equipment required to perform the testing was obtained on monthly rental agreements between various suppliers and EBA. Longyear wireline triple packer assembles for both NQ and HQ drillholes were obtained from Slope Indicator Canada Ltd. of Richmond, B.C. Nitrogen gas inflation bottles were obtained from Northern Metallic Sales Ltd. of Whitehorse, Yukon. And a 200 metre electric water level indicator was obtained from Mow-Tech Ltd. of Edmonton, Alberta.

During the testing period, a total of three falling head permeability tests and eight constant head permeability tests were completed. For the constant head tests, which were completed using the down hole packer equipment, there were three double packer tests and five triple packer tests. The resulting permeability values from these tests are shown in Table 1, attached.

It should be noted that the relatively low number of tests completed during the testing period is a reflection of the logistical difficulties of testing in the relatively deep drillholes and the difficulties of working in cold winter conditions.

1.2 Permeabilities

Of the 5 triple packer tests carried out, the water flow in two of the cases were so low that we can only estimate an upper bound for the permeability. In another case, there appears to have been leakage past the packer (at least at higher test pressures).

The 3 double packer tests, having relatively long test lengths, are believed to have provided more reliable results. Although, these results represent the average permeability over the entire test length. The results of tests #7 and #8 have been averaged, since the excess head (artesian pressure) is so low that it may be difficult to measure it accurately.

Overall, it appears the majority of permeability (hydraulic conductivity) values fall in the range of $5x10^{-8}$ m/s to $2x10^{-9}$ m/s. This represents average conditions. Locally, values could be higher or lower.

1.3 Water Levels (Pressure Distribution)

From Hole 90DY04 we clearly see a downward flow gradient. At Hole 90DY05, the greatest artesian pressure at a test zone was ≈11.0 m at 1920' to 1930'. A higher pressure of 12.7 m (18 psi) was measured for the entire hole, although the pressure could actually be greater since the drill rods may not have been well sealed a the drill hole collar. In any event, it would appear some zones in hole 90DY05 are artesian, but at the depths considered, the artesian pressures represent only a small amount of the total hydrostatic head.

1.4 Triple Packer Tests

For the most part, the rock permeability is no low that it cannot be reliably measured over the short 3.0 m triple packer test length, unless one happens to precisely bracket a permeable zone. Although the test interval could be extended, the length of the packer assemble then becomes very awkward to handle. More importantly, the longer assemble would increase the risk of caving into the test interval, with subsequent damage to the test equipment.

Therefore, although the triple packer may be useful in some instances, it would be better to use the double packer setup with a longer test interval in most cases.

1.5 Double Packer Tests

The recommended approach for further testing would be to drill ahead using water, then retract the drill rods while doing double packer tests with zone lengths of say 20 m, 40 m, and 60 m (65', 130', and 200'). If a particular zone showed anomalous flow, then perhaps some triple packer tests might be considered.

After testing a 200' drill hole section, the hole should be mudded up, then drilling resumed for a further 200' using water.

1.6 Piezometers

Piezometers should be installed in a number of the drill holes. These will allow flow tests (artesian holes) or falling head tests to be carried out, and help to define the hydrostatic pressure distribution. Furthermore, the piezometers could be monitored, for example as the decline is advanced.

1.7 General Conditions

In hydrogeology, accuracy of an estimate is measured in orders of magnitude rather than percent. Although $3x10^{-8}$ m/s to $2x10^{-9}$ m/s is probably a good representation of average conditions, we suspect there are more permeable zones. If for example, in Hole 90DY05. The entire artesian flow of say 2.5 usgpm at 11 psi cam from a single 10' thick zone, that zone would have a hydraulic conductivity of about $8x10^{-6}$ m/s.

Although that would likely represent fairly fractured rock and poor ground conditions, it would only be for a short length , which could be identified by probe holes ahead of the decline face, and perhaps drained or grouted ahead of the face.

1.8 Possible Course of Action

- Avoid further HQ size packer tests (return this equipment)
- Unless there is strong evidence of a specific zone of interest which warrants triple packer testing, the best approach will be to do double packer tests, preferable while advancing through the zone of interest.
- I would recommend installing some piezometers, particularly in the holes less than 1000' deep (since deeper installations are very difficult). Perhaps by incorporating piezometer installation and packer testing, standby can be minimized.
- It is obvious that the cool weather greatly hinders the work. It seems to me that unless drill shacks can be kept warm enough to minimize the cold weather problems, we run the risk of expending a lot of time and effort with little to show for it. The cool weather difficulties must be carefully considered in any decision to continue the work.

2.0 PRELIMINARY ESTIMATE OF SEEPAGE INFLOWS TO PROPOSED SHAFT AND ACCESS DECLINE EXCAVATIONS

2.1 Introduction

In groundwater analysis, it should be appreciated that an error in the range of plus or minus one order of magnitude is not uncommon. The cause for this becomes apparent when you consider that rock permeabilities can typically range from 10^{-5} m/s to 10^{-12} m/s. Furthermore, we utilize the results of borehole permeability tests involving the displacement of perhaps 2 gallons of water, to estimate the possible inflow into an excavation of perhaps 100 times the diameter of the borehole. Whereas the water storativity of the rock and recharge to the rock mass will have no effect on the borehole permeability test these parameters may have a profound effect on the rate and duration of inflow to a much larger excavation where inflow may amount to thousands of gallons.

Although complex numerical analysis methods (finite element) may be used to estimate seepage inflows, from the above discussion it should be evident that such methods are not necessary in this situation where only very preliminary data is available. Accordingly, the methods used at this stage involve only basic analytical equations with appropriate simplifications. Some parameters such as excavation diameters, lengths, orientations, etc. have therefore also been estimated at this stage.

2.2 Excavation Geometry

2.2.1 Shaft

The shaft is assumed to be 800 m (2625’) deep, and 10 m (33’) in diameter.

2.2.2 Decline

The decline is assumed to be 6 m (20') wide and 5 m (16') high. For present purposes, we have assumed an effective size of 6 m diameter.

The length and grads of the decline are not known. We have assumed a grade of 15% and a length of 2000 m (6500').
The depth of cover is expected to increase uniformly from zero at the portal, to 800 m in the vicinity of the shaft base.

2.3 Hydrogeologic Conditions

Although the pressure head has been shown to vary with depth, the pressure head generally appears to be within about 20 m of the ground surface, either above (90DY05) or below (90DY04), hence we have assumed a hydrostatic distribution for preliminary analysis purposes.

Only limited permeability information is available at present. Based on that, we have assumed...

CASE I: Average Low Permeability = 2×10^{-9} m/s EST.
CASE II: Average High Permeability = 5×10^{-8} m/s EST.
CASE III: Localized High Permeability = 2×10^{-6} m/s EST.

Note that the localized high permeability zones are suspected but have not yet been identified during packer testing.

2.4 Analysis Methods - Seepage

Two methods have been considered....

2.4.1 Instantaneous (Initial Flow)

Assumes an undrained pressure head with instantaneous completion of the excavation. In reality, there will be drainage ahead of the face as the excavation advances. With time, the initial inflow will of course diminish, and hence the initial inflow estimate is only valid at the face, and even then just as a maximum flow boundary since advance is obviously not instantaneous.

2.4.2 Ultimate (Steady State) Inflow

The equation used provides an estimate of steady state seepage into the excavation after all transient drainage has ended. Of course, this equation is an approximation only since the steady inflow depends on both the recharge and storage coefficient, and the time since the start of excavation; these parameters are estimated where required.

2.5 Preliminary Estimate of Inflows to Shaft

2.5.1 Maximum Inflow at Face (at 800 m Depth)

CASE I: **k=2x10^{-9} m/s, low average inflow = 0.002 l/s per metre**
COMMENT: Inflow insignificant to excavation operations. Depressurization not practical or necessary.

CASE II: **k=5x10^{-8} m/s, high average inflow = 0.04 l/s per metre**
COMMENT: This is expected to be about the upper limit of inflow which could be tolerated without having significant impact on tunnelling/sinking operations. However, since depressurization/dewatering will occur as the face advances, groundwater inflow is unlikely to pose a problem.

CASE III: **k=2x10^{-6} m/s, localized high inflow - 1.5 l/s per metre**

(This could amount to about 50 Igpm total at the face/excavation round).

COMMENT: This sort of inflow might locally be expected in severely fractured zones, but not those containing a lot of gouge or other joint healing. Probe holes should be extended ahead of the face to give warning of such zones if known or suspected. If probe holes indicate that such permeable zones are present, drainage holes and/or grouting should be considered in advance of the face before continuing excavation with shortened rounds. Heavier support will likely be required.

NOTE: These estimates are based on the full depth of the shaft. Inflows would be proportionately less at shallower depths.

2.5.2 Utimate (Long Term) Total Inflow

CASE A: **750 m of $2x10^{-9}$ m/s and 50 m of $2x10^{-6}$ m/s**

TOTAL INFLOW = 5 l/s (65 Igpm)

COMMENT: Most of the inflow will be from the permeable ($2x10^{-6}$ m/s) zones, and since these are expected to be localized, they could probably be grouted to reduce long term pumping costs

CASE B: **750 m of $5x10^{-8}$ m/s and 50 m of $2x10^{-6}$ m/s**

TOTAL INFLOW = 12 l/s (160 Igpm)

COMMENT: Since most of the inflow will generally occur throughout the shaft, any efforts to reduce it will probably be most effective nearer the bottom where flows are likely to be greatest.

2.6 Preliminary Essimate of Inflows to Decline

2.6.1 Maximum Inflow at Face (800 m depth)

CASE I: **$k=2x10^{-9}$ m/s, low average inflow = 0.002 l/s per metre**
COMMENT: Inflow generally insignificant.

CASE II: **$k=5x10^{-8}$ m/s, high average inflow = 0.04 l/s per metre**
COMMENT: See Case II for shaft.

CASE III: **$k=2x10^{-6}$ m/s, localized high inflow - 1.4 l/s per metre**
COMMENT: See Case III for shaft. Probe holes ahead of face are warranted to identify such zones before they are encountered.

2.6.2 Ultimate (Long Term) Total Inflow

CASE A: **1900 m of $2x10^{-9}$ m/s and 100 m of $2x10^{-6}$ m/s**
TOTAL INFLOW = 10 l/s (130 Igpm)
COMMENT: See Case A for shaft

CASE B: **1900 m of $5x10^{-8}$ m/s and 100 m of $2x10^{-6}$ m/s**
TOTAL INFLOW = 28 l/s (370 Igpm)
COMMENT: See Case B for shaft

2.7 Summary Comments

At present, the available data seems to indicate that groundwater inflows will not pose a major problem, however, at present the data is limited to only two areas hence has not addressed geologic variations along the proposed decline alignment.

Since it is likely that the highest and most troublesome inflows will be associated with more highly fractured or faulted zones, further investigation should be directed towards locating these zones and more accurately identifying their characteristics (permeability and rock support parameters). Where such zones of poorer rock are known or expected to be present, it may be prudent to maintain probe holes advanced ahead of the face.

Following a brief examination of the drill core from the lower 400' of the shaft pilot hole, the phyllite may be prone to deterioration and may require initial shotcrete support to prevent this.

Rock exposures near the shaft collar are much more broken, although the depth of this apparent surficial weathering is not known.

Based on examination of core from the lower fault zone in the shaft pilot hole, the gouge may result in a lower than expected permeability, although the very weak nature of the rock will undoubtable require a concrete (reinforced) lining for permanent support.

It should be recognized that the effect of groundwater on tunnelling operations depends very much on the rock characteristics. For example, inflows of 1000 gpm in Karst limestone where tunnel support is not required versus inflows of 0.1 gpm in fault gouge where heavy steel support is required. At this preliminary stage, the groundwater conditions have been considered independent of the rock characteristics and support requirements.

TABLE 1

HOLE	TEST NO.	ZONE	LENGTH	S.W.L.[1]	K(m.s)	COMMENTS
FALLING HEAD TESTS:						
90 DY 01		330'-457'	38.7 m	-9.95	$3x10^{-8}$	
90 DY 02		188'-360'	52.4 m	-11.47	$2x10^{-10}$	
90 DY 03		386'-497'	33.8	-16.31	$1x10^{-8}$	
PACKER TESTS:						
90 DY 04	1	1940'-2173'	71 m	-19.7	$2x10^{-9}$	
	2	1900'-1910'	3 m	-13.2	$4x10^{-8}$	
	3	1850'-1860'	3 m	-6.4	$5x10^{-8}$	
	4	1810'-1820'	3 m	(-4.0)	$(3x10^{-7}$	Possible leakage past packer
90 DY 05	5	1890'-1900'	3 m	+8.4	$\leq 3x10^{-9}$	Below detection limit
	6	1920'-1930'	3 m	+11.0	$\leq 6x10^{-9}$	Below detection limit
	7	1750'-2158'	124 m	+4.9	$6x10^{-9}$	Average = $1x10^{-8}$m/s
	8	1810'-2158'	106 m	+2.8	$2x10^{-8}$	Average = $1x10^{-8}$m/s

NOTES: (1) Static Water Level (SWL). "+" indicates above ground. For angle hole 90 DY 01, the vertical distance is given. Levels measured from top of piezometer.