19/1/95

Dick

003170

PITEAU ASSOCIATES
GEOTECHNICAL AND
HYDROGEOLOGICAL CONSULTANTS

215 - 260 WEST ESPLANADE
NORTH VANCOUVER, B.C.
CANADA V7M 3G7
TELEPHONE (604) 986-8551
FAX (604) 985-7286

CURRAGH RESOURCES INC.

GRUM PIT

REVIEW AND ANALYSIS OF HYDROGEOLOGICAL DATA AND DESIGN OF PHASE I DEWATERING SYSTEM

Project 89-193

January 1991

CONTENTS

FIGURES

TABLES

1. INTRODUCTION

1.1 BACKGROUND

Curragh Resources Inc. (Curragh) is developing the Grum orebody at their mining operation near Faro, Y.T. Initial development on the east side of the pit will be in unconsolidated sediments, to a maximum depth of approximately 100m in a bedrock channel, or thalweg, which trends in a northerly direction through the pit area (Fig. 1). Initial geotechnical exploration behind the east wall of the proposed pit encountered permeable sediments in the thalweg beneath the overlying till. Piezometers installed in these sediments flowed continuously, indicating that the permeable sediments comprise a fairly extensive artesian aquifer. To prevent problems with pit wall stability and to improve operating conditions in the pit, it will be necessary to depressurize and dewater this aquifer in advance of mining.

1.2 TERMS OF REFERENCE

Piteau Associates Engineering Ltd. (PAEL) was retained by Curragh to review available data on the hydrogeology of the Grum Pit area, and to design a dewatering program for the Grum Pit. The program was to consist of two phases. Phase I was to include a nominal number of wells sited at various locations throughout the aquifer, around the pit perimeter. Ideally the Phase I wells would have sufficient capacity to depressurize the aquifer before the pit excavation reaches the 1260m elevation. Phase II wells, which would be installed after the Phase I wells had been in service for a short period, would be drilled at intermediate locations. They would be installed on an as-required basis, dependent upon the effectiveness of the Phase I dewatering system.

1.3 PREVIOUS STUDIES

Previous investigations at the Grum orebody have included extensive diamond drilling from both surface and an underground adit. Curragh personnel have used the surface drilling information to interpret a bedrock surface contour plan, which can be used to define the boundaries of the aquifer.

A preliminary groundwater investigation was conducted by Mr. John Huntley of Curragh during 1987/88. This study involved the installation and subsequent monitoring of ten piezometers, interpretation of groundwater flow directions and recommendations regarding a pump test to characterize the aquifer and to further define the dewatering requirements (J.F. Huntley, March 1988).

A test well program was conducted in early 1989. EBA Engineering Consultants Ltd. supervised construction of the test well and observation wells, and the subsequent pumping test. Hydrogeological Consultants Ltd. analyzed and reported the results of the pumping test (Hydrogeological Consultants Ltd., July 1989).

2. HYDROGEOLOGY

2.1 GENERAL DESCRIPTION

The majority of surficial sediments which overlie bedrock in the Grum Pit area are low permeability till, and are of little concern to the hydrogeological aspects of the proposed mining operation. The permeable sands and gravels which have infilled the bedrock channel, and generally lie within the elevation range of 1260m to 1190m, represent a major aquifer, which is the major water bearing hydrogeological unit in the Grum Pit area.

Piezometric data for the various piezometers in the Grum Pit area were contoured by J.F. Huntley. The contours indicated that groundwater flows in a southerly direction, with groundwater recharge occurring to the north and northeast of the proposed pit, and groundwater discharge occurring in the gentle valley to the south of the proposed pit (Fig. 1). Artesian heads measured in the aquifer under the area of Doal Lake indicate that groundwater flow to the discharge area is restricted, possibly due to low permeability zones within the aquifer.

2.2 AQUIFER PARAMETERS

Well GDW89-01 was pump tested in May 1989. The pump test analysis report (Hydrogeological Consultants Ltd., July 1989) was reviewed and some of the pump test data was reanalyzed (Appendix A). Aquifer parameters interpreted from the pump test data are summarized in Table I.

The pump test results indicate that the transmissivity of the aquifer is approximately $8x10^{-3}$ m^2/s, and that the storativity is approximately $8x10^{-4}$. Continual breaks in slope in the drawdown plots (Appendix A) indicate that many boundaries were encountered during the test. The absence of any positive breaks in slope on the semi-log plots (Plots A-1 and A-5 in Appendix A), and incomplete recovery at the end of the recovery period, indicate that recharge is quite limited.

It is assumed that the sand and gravel that infill the thalweg are the primary aquifer and that the bedrock sides to the thalweg form the aquifer boundaries. However, it is possible that the fractured zone at the bedrock surface has some transmissivity, and any dewatering wells should be completed in both the sand and gravel and the first few metres of bedrock.

The till which overlies the aquifer is estimated to have a hydraulic conductivity of 10^{-9} to 10^{-8} m/s in the horizontal plane, and a slightly lower hydraulic conductivity in the vertical direction. For the purposes of the modelling, horizontal and vertical conductivities of 10^{-7} m/s and $3x10^{-8}$ m/s, respectively, were assumed. These slightly higher conductivities would more readily distribute recharge applied at the finite-difference mesh surface, and allow more leakage from the till into the aquifer; hence, the modelling results should be slightly conservative.

3. SIMULATION OF AQUIFER HYDRAULICS

A three-dimensional finite-difference model (MODFLOW) was used to simulate the aquifer hydraulics, in order to extrapolate the short term pump test data to the long term dewatering situation. The finite-difference mesh was a 27x24 grid superimposed on the area shown on Fig. 1. The mesh consisted of four layers, with layers 1 and 2 being the till units, layer 3 the sand and gravel aquifer, and layer 4 the upper 50m of bedrock.

3.1 SIMULATION OF PUMP TEST

To simulate the pump test, a uniform head was applied to all nodes in the mesh, and boundary conditions were all set to no flow (i.e. zero recharge). Various parameters were applied to the four layers, and the program was run repeatedly until a reasonable match with the field data was obtained. As shown on Fig. 2, a reasonable match to the field data could be simulated with the zero recharge assumption. Best fit parameters for the four layers are shown on Fig. 2.

3.2 SIMULATION OF INITIAL DEWATERING TRIAL

A dewatering trial was conducted between October 28 and December 2, 1990. Dewatering well GDW89-01 was pumped at a rate of approximately 18 L/s (300 USgpm) over this period, until the pump malfunctioned on December 2. Drawdown measurements were taken in the wells three times during the pumping period, and twice following the pump failure.

The finite-difference program was used to simulate the trial dewatering test. Two general head boundaries were added to simulate recharge to the sand and gravel aquifer at the mesh boundaries. General head boundaries allow controlled recharge to the mesh as a function of the difference between the head simulated for the particular node at the edge of the mesh and the constant head maintained at the mesh boundary. In addition, a recharge flux was applied to the mesh surface to simulate recharge from precipitation.

A constant head was applied to the entire mesh at the start of the simulation. Ideally, the starting heads should have been a steady-state flow condition computed by the model. However, a steady-state condition similar to that observed in the field could not be generated without making many assumptions regarding lateral variations in the aquifer. Low permeability zones would have been necessary within layer 3 of the mesh to induce the artesian heads that are observed in the field.

A fairly close match between the simulated and observed field drawdown was obtained using the parameters tabulated on Fig. 2, plus surface recharge of about 4 L/s (65 USgpm) and a very small recharge component (<0.5 L/s or 10 USgpm) from the general head boundaries at the boundaries of the mesh in layer 3. Approximately 80% of the water pumped by the well was accounted for by removal of water from storage in the aquifer and overlying layers.

3.3 DEWATERING SIMULATION

The finite-difference model was used to assess the effectiveness of a four well and a five well dewatering system. The parameters and boundary conditions were left unchanged from the dewatering trial simulation, and wells were added at nodes in layer 3, at locations around the perimeter of the pit. Approximate well locations are shown on Fig. 1.

Results of the modelling indicate that four wells (including GDW89-01) will not be sufficient to depressurize the aquifer within a three to six month time frame, but five wells (including GDW89-01) pumping at an average rate of 12.5 L/s (200 USgpm), will depressurize the aquifer to the 1264m elevation after three months, and to 1260m elevation after six months (Fig. 3). The piezometric surface in the aquifer is expected to draw down fairly uniformly; hence, any groundwater mounds between wells should be very small, and were computed to be less than 3 to 4m.

Lowering the piezometric level in the aquifer below the 1260m elevation will be a slower process as the aquifer must be dewatered, as opposed to depressurized, below this elevation.

It should be noted that this simulation is based on limited data, and is not intended as a definitive analysis of aquifer hydraulics. The results of the analysis are sufficient for the design of a first phase dewatering system, but the early performance of the system should be monitored and a second phase dewatering system designed and implemented, in order to meet the required dewatering objective.

4. DEWATERING SYSTEM RECOMMENDATIONS

It is recommended that four wells be installed as a Phase I dewatering program. The first three wells should be installed at the locations indicated on Fig. 1 and Table II, and the fourth well should be at one of the alternative locations shown. Well 1 should be drilled first, so that the hydraulic connection between this portion of the aquifer and GDW89-01 can be assessed (by radical change in pumping rate at GDW89-01, and monitoring for a response in the new Well 1). If there is a poor hydraulic connection (i.e. little or no response to hydraulic upset induced by GDW89-01), Well 4 should be located near Well 1, unless Well 1 is expected to be a poor producer. If there is a good response, indicating a good hydraulic connection, or if Well 1 is expected to have a low yield, Well 4 should be drilled at a site north of the pit, as shown on Fig. 1.

4.1 WELL DESIGN

The proposed well design is shown on Fig. 4. Screen slot sizes and actual installed depth should not be determined until after the wells are drilled but, assuming that 3x1.5m long screens with 0.600 in slots are installed in each well, the screen transmitting capacity should be approximately 400 USgpm, roughly twice the maximum rate that the wells would be pumped at. Provided the screens are properly sized and developed, there should be adequate screen capacity to obtain the required well yields.

The specific capacity of well GDW89-01 was approximately 1.4 L/s/m at 10.5 L/s and 1.1 L/s/m at 18.9 L/s. This is not considered to be an efficient well for an aquifer of this transmissivity, and considerable effort should be expended in developing the new wells to obtain the most efficient wells possible. If there is too much head loss across the well screens, it will not be possible to pump the wells at a rate of 200 gpm, and additional wells would be required to achieve the desired total withdrawal rate of about 63 L/s (1000 USgpm).

4.2 WELL INTERFERENCE

The finite-difference model incorporates interference between wells into the computed drawdown. Interference between the Phase I wells will be quite severe, and will certainly reduce the available drawdown at each well to prevent pumping at the 18 L/s at which GDW89-01 can presently be pumped. Provided efficient wells are constructed, and head loss across the well screens is less than about 5m at the proposed 12.6 L/s (200 USgpm) pumping rate, interference between the wells will not be so severe as to prevent pumping of the wells at the design rates. However, as more wells are added as part of a Phase II dewatering program, interference effects will increase, and pumps will likely have to be valved back to allow for continuous operation.

4.3 PUMPS

Pumps for the Phase I dewatering system should all be nominal 150mm pumps with rated capacities of between about 10 and 13 L/s, against a total dynamic head varying from about 60 to 90m. Ideally, the pumps should be selected after an initial test of each well but, if they are to be purchased beforehand, three pumps with a 12.6 L/s (200 USgpm) capacity and one or two pumps with a 10 L/s (160 USgpm) capacity could be selected to provide some flexibility at the time of installation.

It should be noted that the initial pumps will eventually have to be replaced, possibly when additional wells are added during the Phase II dewatering program (and interference effects reduce well yields), and definitely after the aquifer has been partially dewatered and well yields fall off. At the conclusion of the dewatering program, it is anticipated that small pumps installed in three or four of the wells will be sufficient to maintain a near dewatered condition in the aquifer.

4.4 ESTIMATED PHASE I COSTS

Estimated costs for the Phase I dewatering program are summarized on Table III. The cost estimate assumes a total of four wells are drilled, and the existing pump is used for one of the wells. All discharge pipe and fittings required to convey water from the well head to the perimeter ditch or discharge pipe are not included in the estimate. The supervision costs are based on eleven days of on-site supervision, and preparation of a brief completion report. Costs do not include any assessment of the monitoring data, which would be performed as part of the Phase II program.

4.5 PHASE II DEWATERING PROGRAM

The Phase I dewatering program is intended to depressurize the aquifer to the 1260m elevation, so that mining can proceed to this depth with minimal problems due to seepage from the overburden. Dewatering below 1260m will require additional wells, but these should not be designed until the Phase I dewatering system is operational, and monitoring data is available to further assess aquifer hydraulics. It is anticipated that a number of smaller capacity wells will be necessary to obtain the groundwater withdrawal rate required to dewater the aquifer below 1260m, on a schedule compatible with the mining schedule.

The second phase dewatering system would incorporate the additional wells required to dewater the aquifer below the 1260m elevation, and these could possibly be located on final benches or at locations in the interim pit, to reduce the quantity of drilling involved.

The cost of the Phase II program cannot be estimated at this time, but would likely require a review of the Phase I dewatering program monitoring data and construction of an additional five wells, for a total cost similar to the Phase I program.

4.6 OTHER DEWATERING MEASURES

Wells will not be capable of completely dewatering an aquifer; hence, allowance must be made for some seepage into the pit at the bedrock/overburden contact, particularly near the centre of the thalweg. The pit plan should provide for a wide enough berm on the first bench in bedrock to construct a sump for seepage collection and conveyance, and also to buttress the toe of the overburden slope, or flatten the angle of the overburden slope toe, to prevent instability due to saturated toe conditions and to control erosion due to seepage.

5. SUMMARY AND CONCLUSIONS

The deep aquifer in the Grum Pit area is of fairly limited extent, with many boundaries evident in the pump test data. Recharge to the aquifer is also very limited, as indicated by the plots of pump test drawdown and recovery data, and also the slow rate of recovery following the pump failure during the one month long dewatering trial.

Although the extent of the aquifer appears to be limited and recharge quantities are indicated to be low, there is a large quantity of water stored in the aquifer and overlying sediments. An estimated withdrawal rate of 63 L/s (1000 USgpm) will be necessary to depressurize the aquifer in a three to six month period.

The rate at which groundwater can be pumped from the aquifer is limited by many practical constraints. As the aquifer is depressurized/dewatered, the capacities of the individual wells will fall off, necessitating construction of additional wells to maintain the required rate of drawdown. If the pit development is to proceed on a schedule which will require dewatering of the aquifer below the 1260m elevation at a fast rate, further wells will be necessary to increase the pumping capacity above 63 L/s (1000 USgpm). However, as more wells are added, interference effects will reduce the capacity of each individual well; hence, there is a practical limit to the number of wells that should be installed.

The proposed Phase I dewatering system should provide adequate dewatering to mine to the 1260m elevation; a Phase II dewatering system will be necessary to dewater below this depth. It is therefore imperative that the Phase I dewatering system be installed as early as possible, so that groundwater withdrawal can begin and the performance of the first five wells can be assessed. Following this assessment, the Phase II dewatering requirements can be determined, and the necessary additional dewatering measures implemented.

Respectfully submitted,

PITEAU ASSOCIATES ENGINEERING LTD.

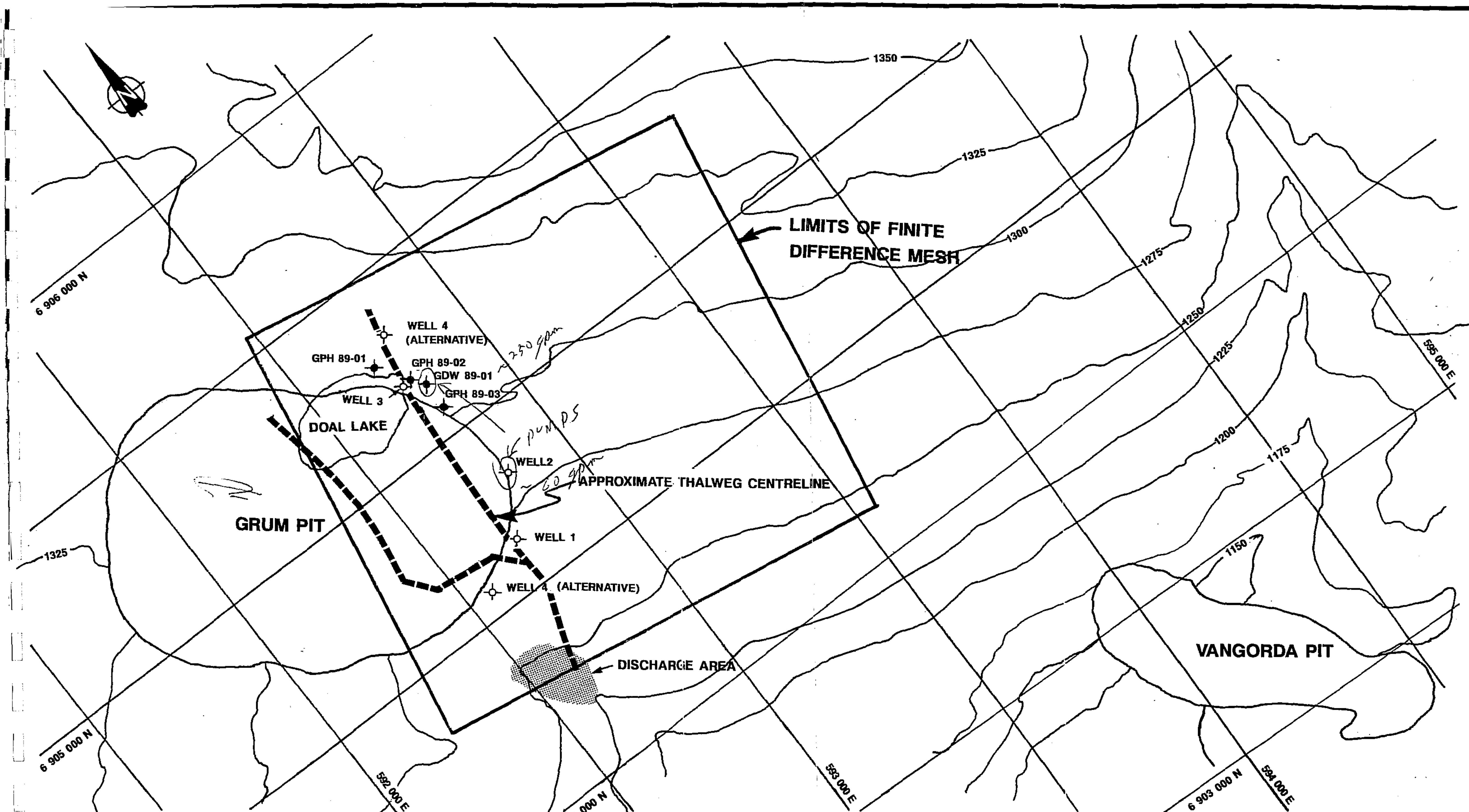
Andrew T. Holmes, P.Eng.

REFERENCES

Huntley, John F., March 1988. "Vangorda/Grum Groundwater Investigation (Interim Report)". In-house report for Curragh Resources Inc.

Hydrogeological Consultants Ltd., July 1989. "Grum Pit - 1989 Dewatering Assessment". Report prepared for Curragh Resources Inc.

FIGURES



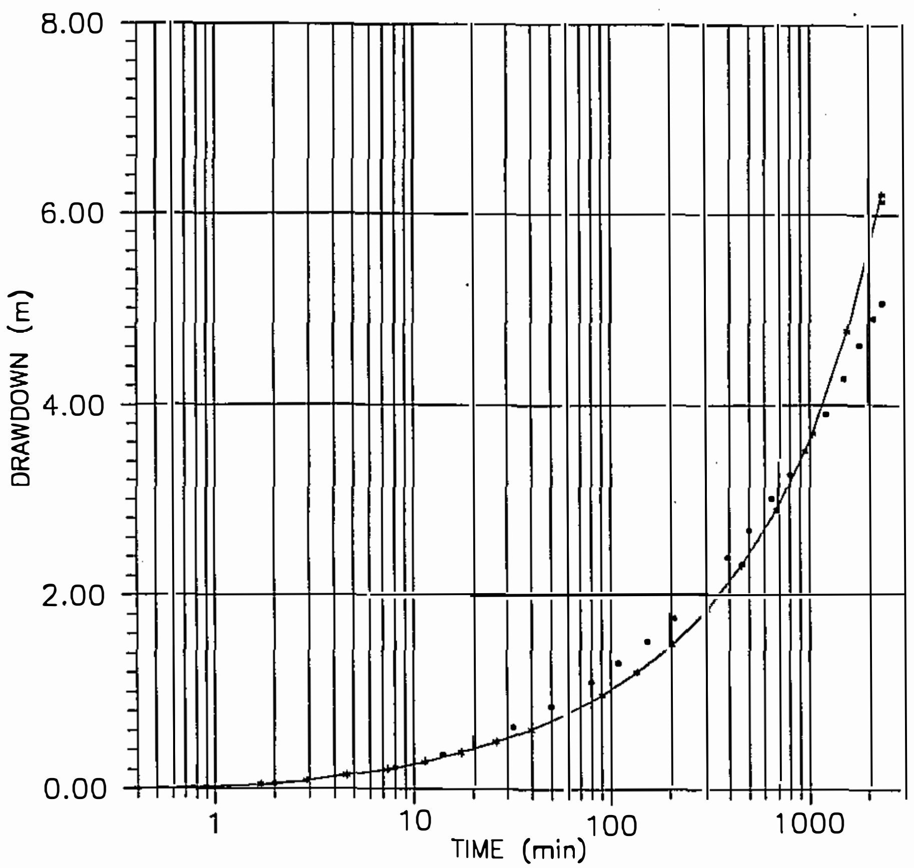
0 100 200 400 600 800 1000m

SCALE 1:10,000

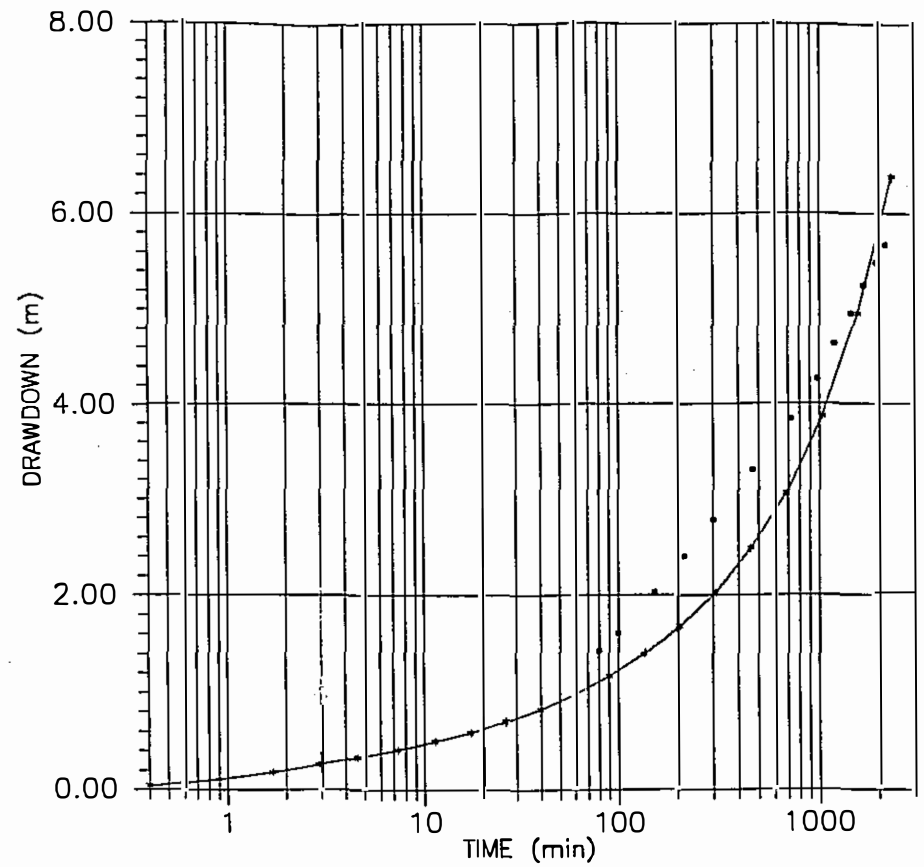
SYMBOLS

- EXISTING WELLS
- PROPOSED DEWATERING WELLS

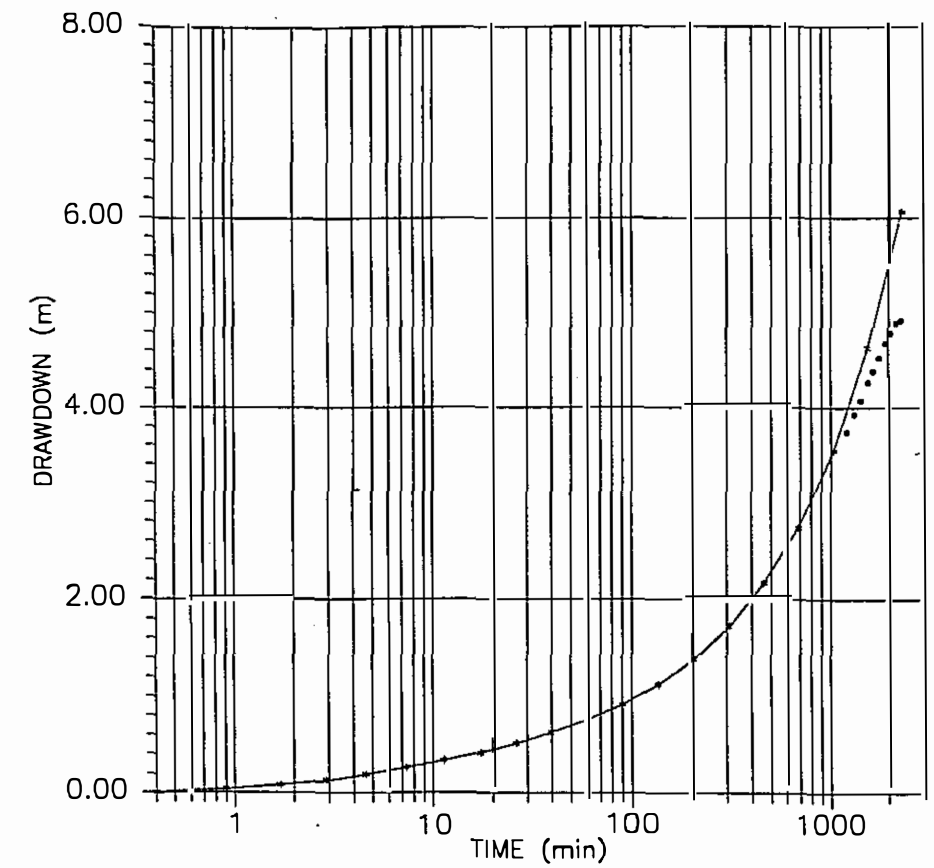
CURRAGH RESOURCES INC. GRUM PIT		PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY	
PLAN OF GRUM PIT AREA		BY: LAS	DATE: DEC.90
		APPROVED:	FIG. 1



GPH 89-01



GPH 89-02



GPH 89-03

SYMBOLS

****** MODEL GENERATED DRAWDOWN IN LAYER 3

...... ACTUAL DRAWDOWN IN LAYER 3

SUMMARY OF INPUT PARAMETERS FOR BEST FIT
SIMULATION OF PUMPING TEST OF GDW 89-01

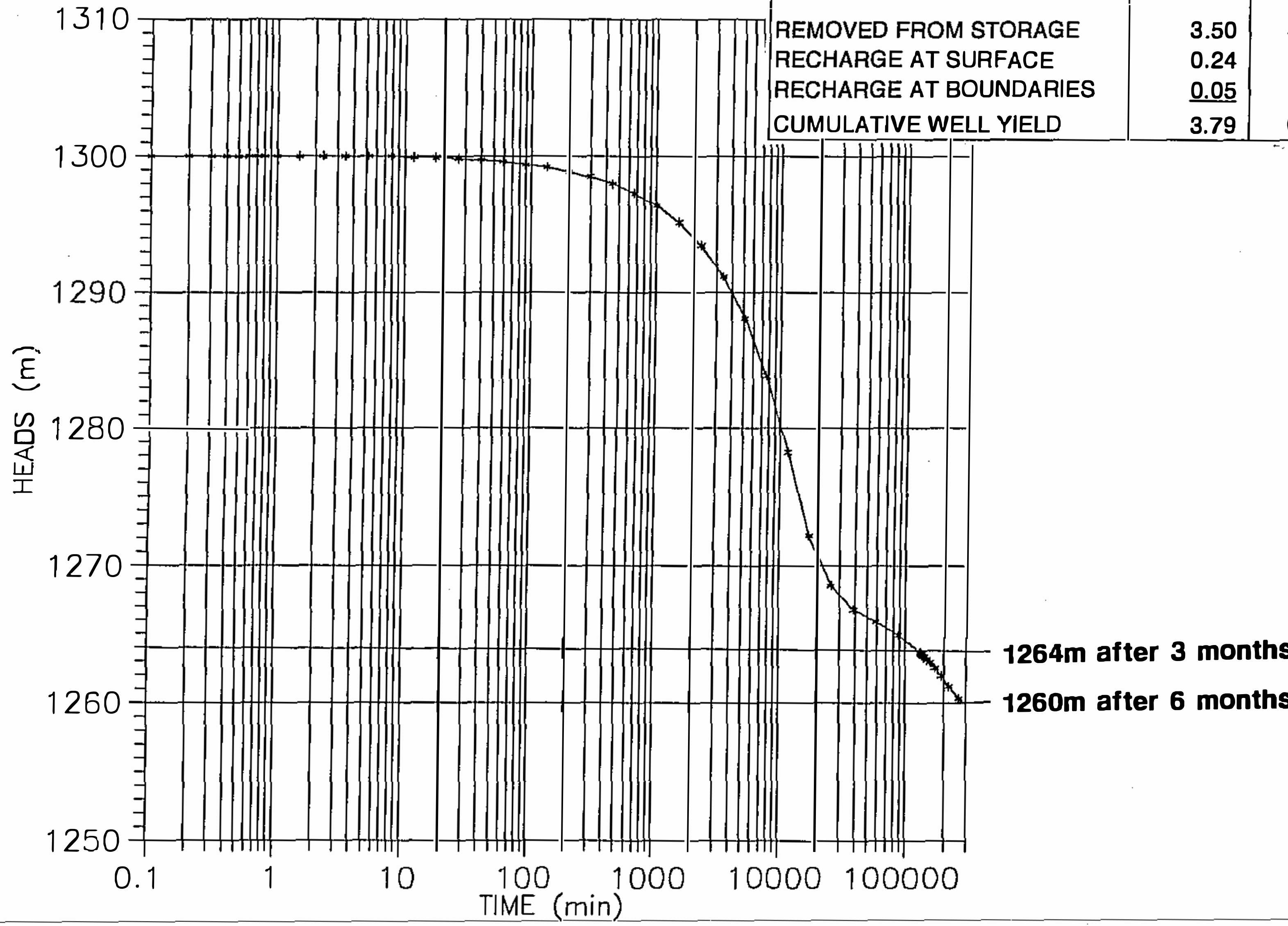
	K(m/s)	K(m/min)	T(m2/min)	S (1/m)	Sy	b (m)	Kv (m/s)	Vc (1/min)
LAYER 1	1.0E-07	6.0E-06	1.2E-04	1.0E-04	5.0E-02	20	3.33E-08	1.00E-07
LAYER 2	1.0E-07	6.0E-06	1.2E-04	1.0E-04	5.0E-02	20	3.33E-08	2.00E-07
LAYER 3	3.8E-04	2.3E-02	6.9E-01	1.0E-03	1.0E-01	30	1.28E-04	8.00E-08
LAYER 4	1.0E-07	6.0E-06	3.0E-04	1.0E-06		50	3.33E-08	

NOTES:
1) LAYER 1 IS UPPER TILL
2) LAYER 2 IS LOWER TILL
3) LAYER 3 IS SAND AND GRAVEL AQUIFER
4) LAYER 4 IS BEDROCK

CURRAGH RESOURCES INC. GRUM PIT	PITEAU ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY
PUMP TEST SIMULATION WITH FINITE - DIFFERENCE MODEL	BY: LAS DATE: DEC.90 APPROVED: GH FIG. 2

JOB NUMBER

WATER BALANCE AT TIME = 3 MONTHS		
FLUX	(M^3/min)	(L/s)
REMOVED FROM STORAGE	3.50	58.3
RECHARGE AT SURFACE	0.24	4.0
RECHARGE AT BOUNDARIES	0.05	0.8
CUMULATIVE WELL YIELD	3.79	63.2



NOTES: 1) DRAWDOWN IS AT POINT IN CENTRE OF THALWEG
2) FIVE WELLS PUMPING AT CONTINOUS RATE OF 12.6 L/S (200 USGPM) EACH

CURRAGH RESOURCES INC.
GRUM PIT

SIMULATED AQUIFER DRAWDOWN
FOR PHASE I DEWATERING SYSTEM

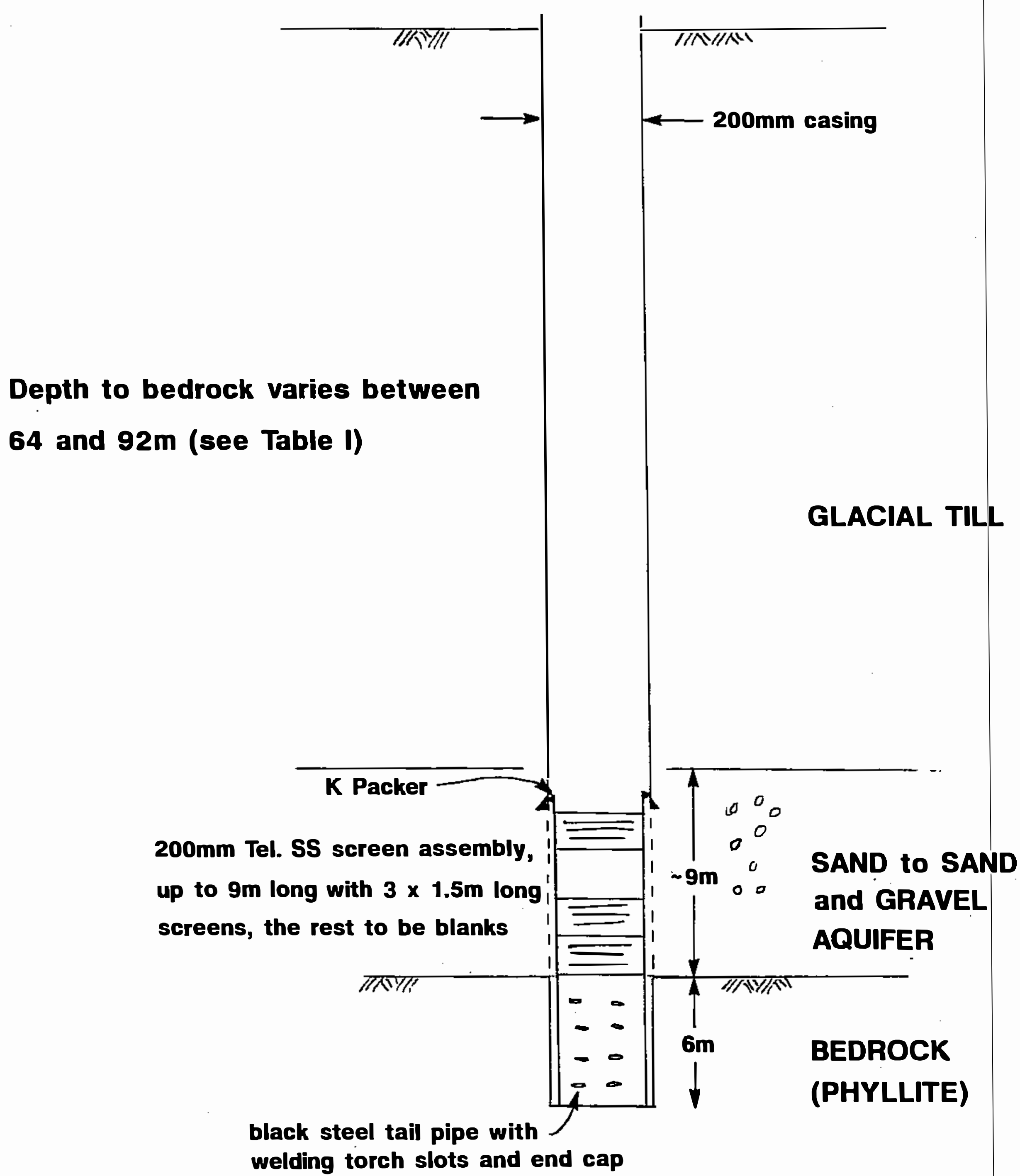
PITEAU & ASSOCIATES
GEOTECHNICAL CONSULTANTS
VANCOUVER CALGARY

BY: LAS
DATE: JAN.91
APPROVED:

FIG. 3

JOB NUMBER

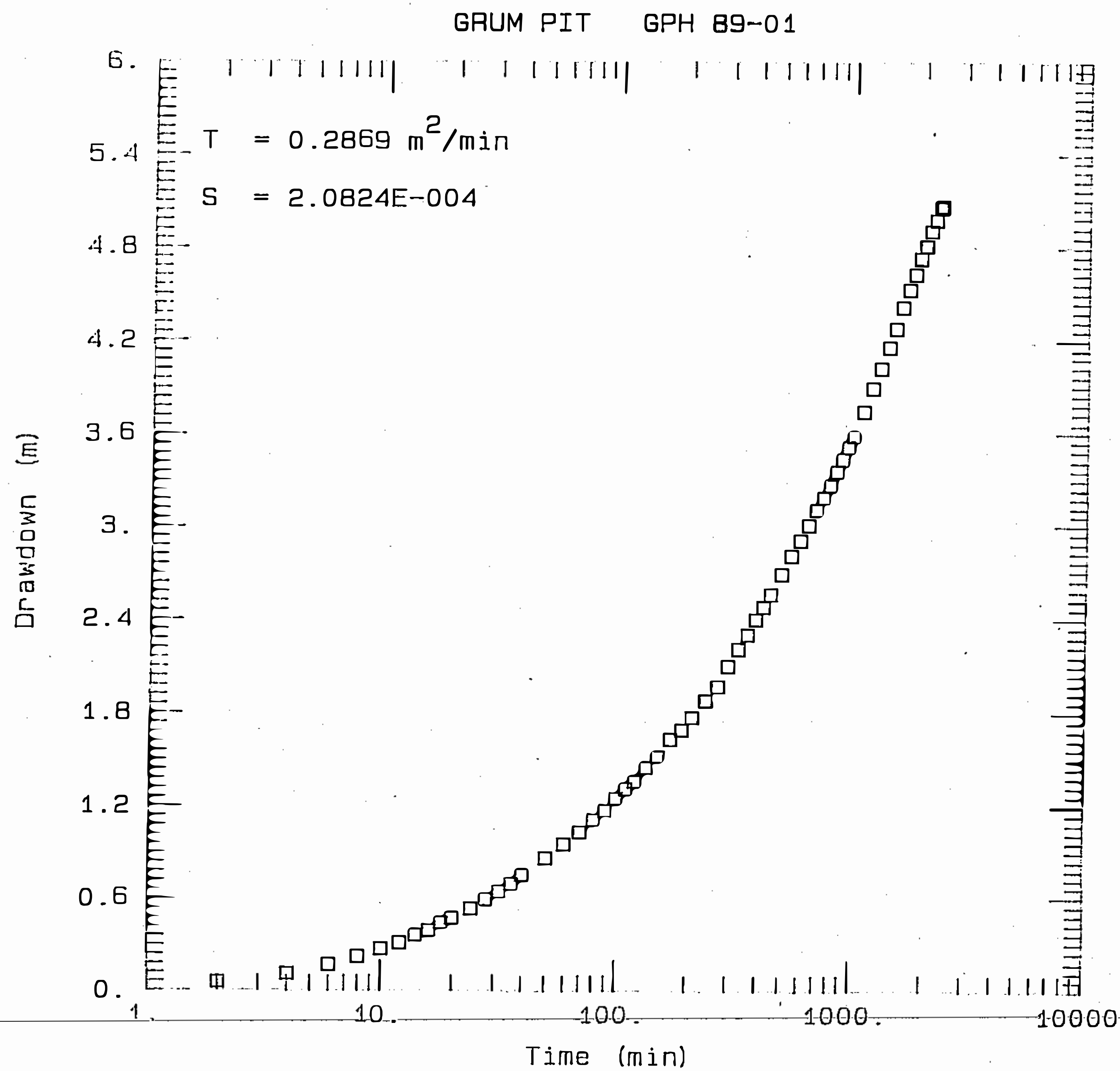
GRUM PIT WELLS



CURRAGH RESOURCES INC. GRUM PIT	PITEAU & ASSOCIATES GEOTECHNICAL CONSULTANTS VANCOUVER CALGARY		
PROPOSED DEWATERING WELL DESIGN		BY: ATH	DATE: DEC.90
		APPROVED: AH	FIG. 4

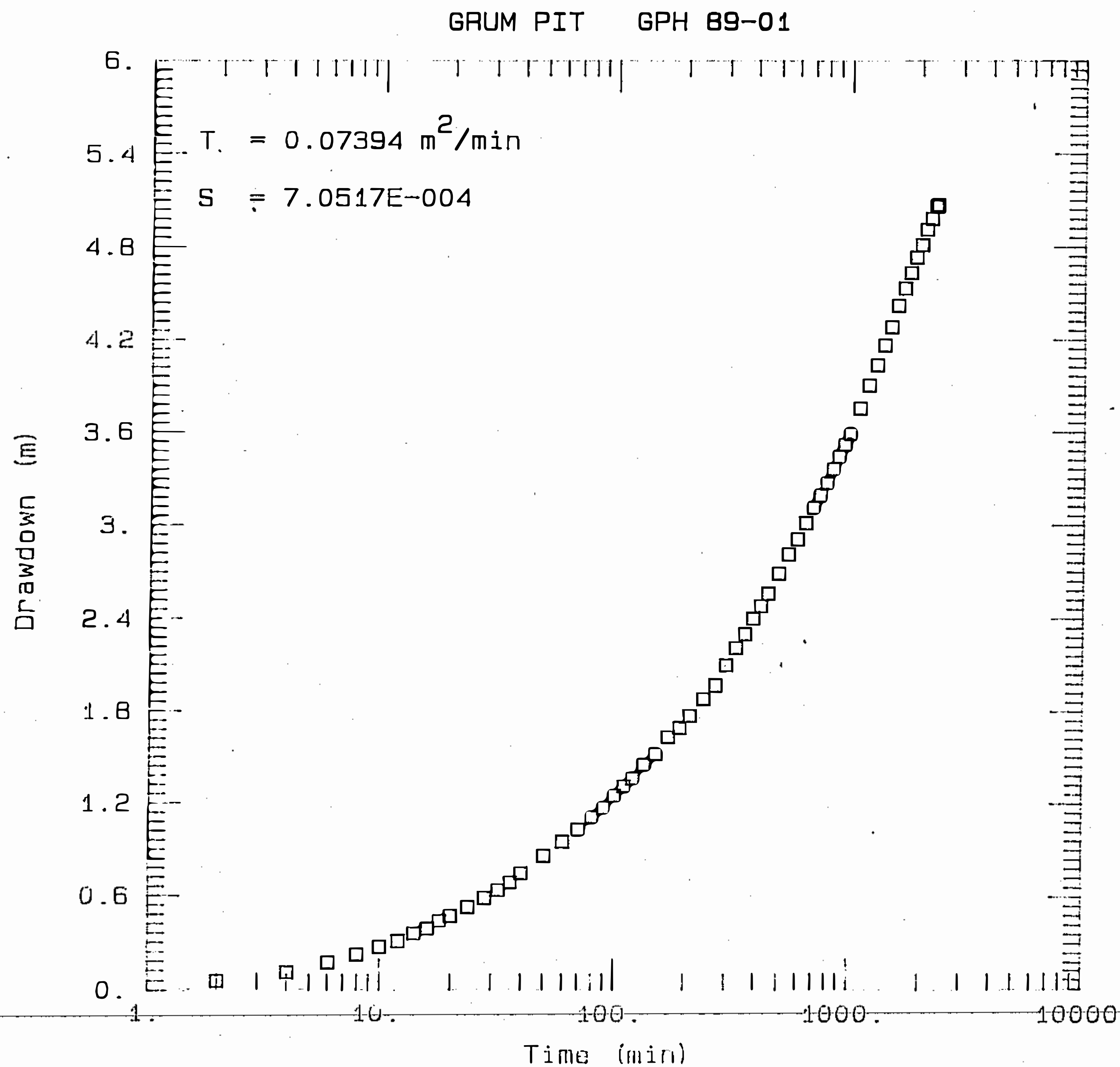
APPENDIX A

PLOTS OF OBSERVATION WELL DRAWDOWN DURING GDW 89-01 PUMP TEST

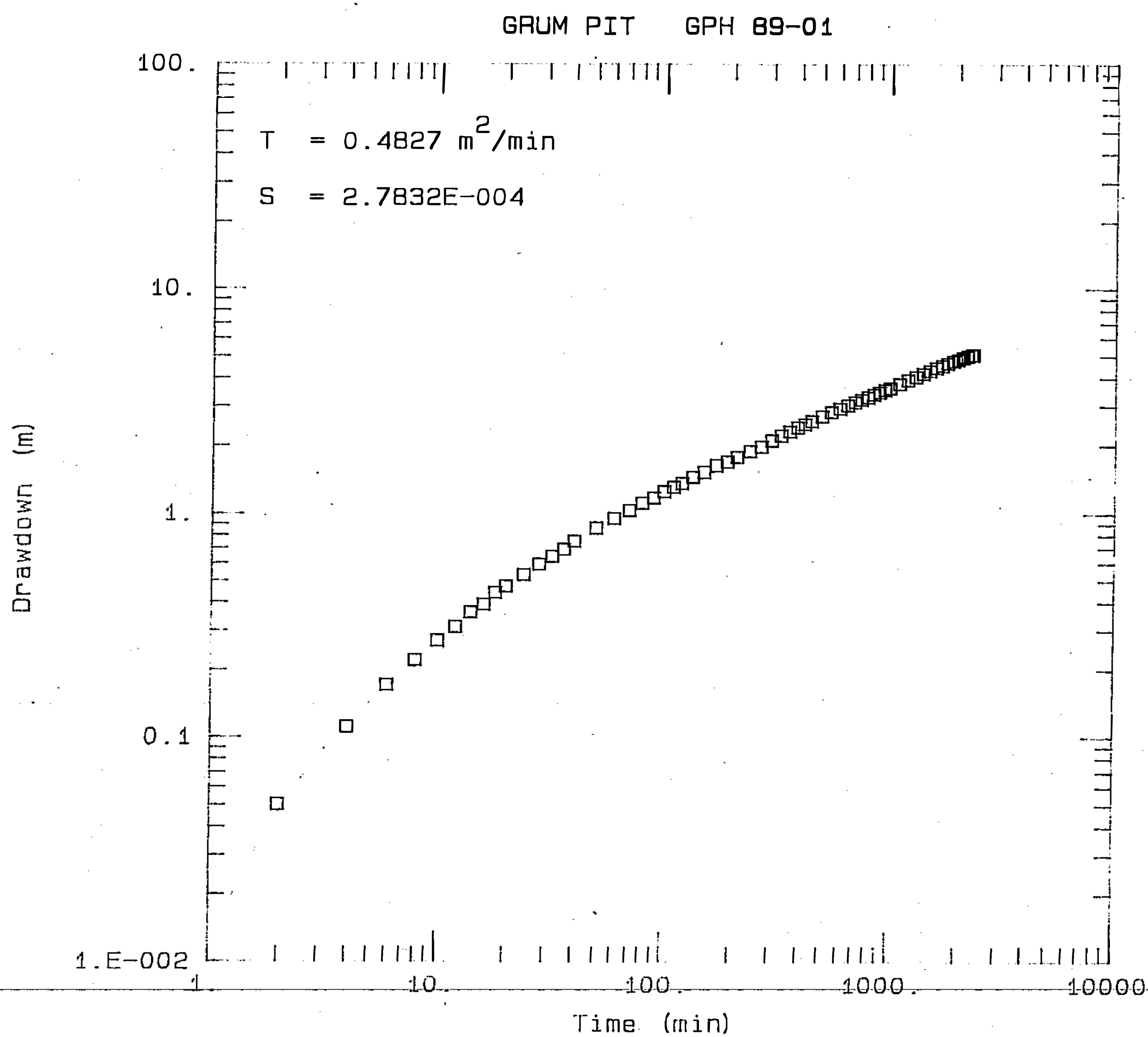


GRUM PIT GPH 89-01
T = 0.2869 m2/min
S = 2.0824E-004
Drawdown (m)
6.
5.4
4.8
4.2
3.6
3.
2.4
1.8
1.2
0.6
0.
1.
10.
100.
1000.
10000
Time (min)

PLOT A-1

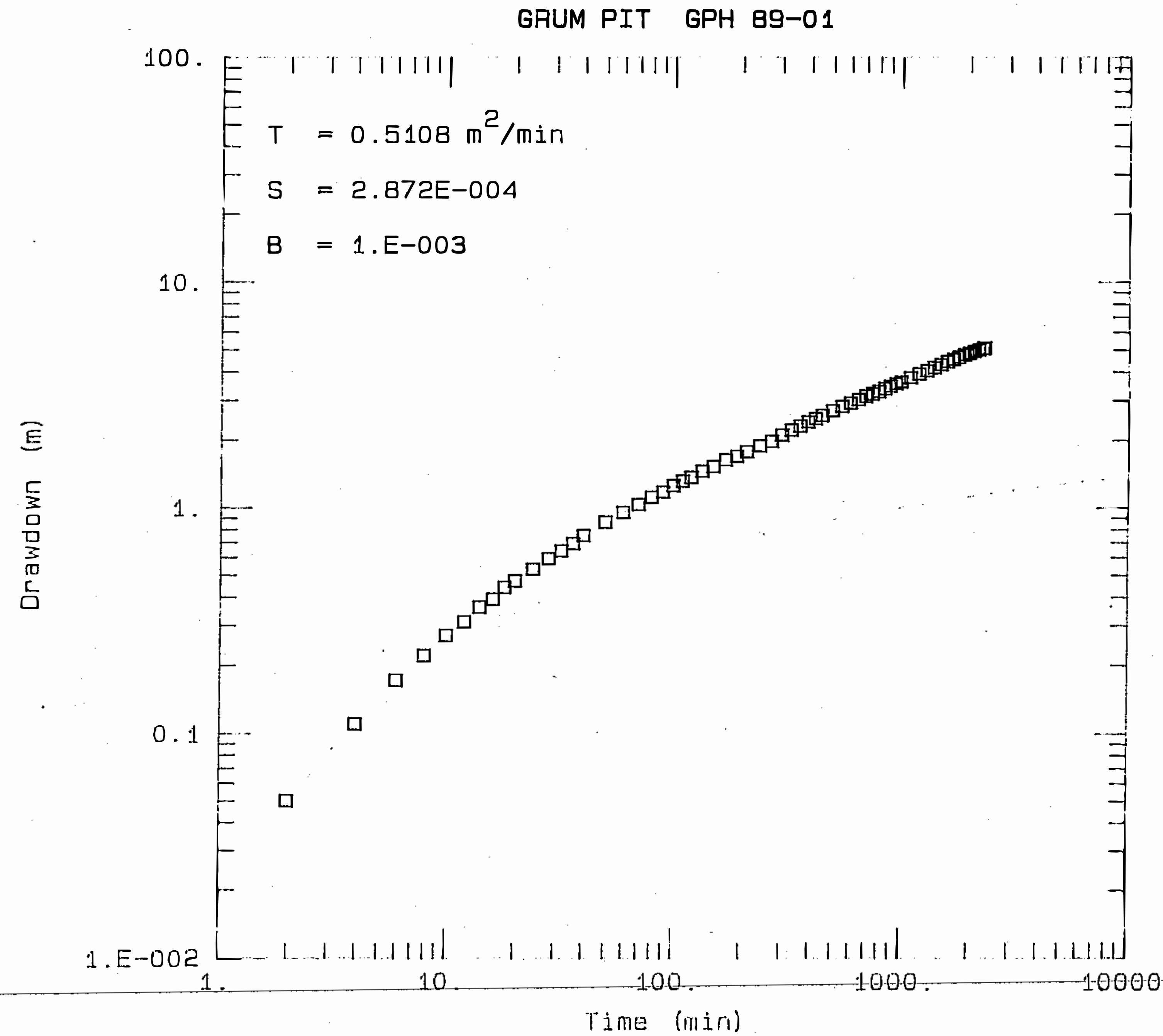

GRUM PIT GPH 89-01
T = 0.07394 m2/min
S = 7.0517E-004
Drawdown (m)
Time (min)
6.
5.4
4.8
4.2
3.6
3.
2.4
1.8
1.2
0.6
0.
1.
10.
100.
1000.
10000

PLOT A-2



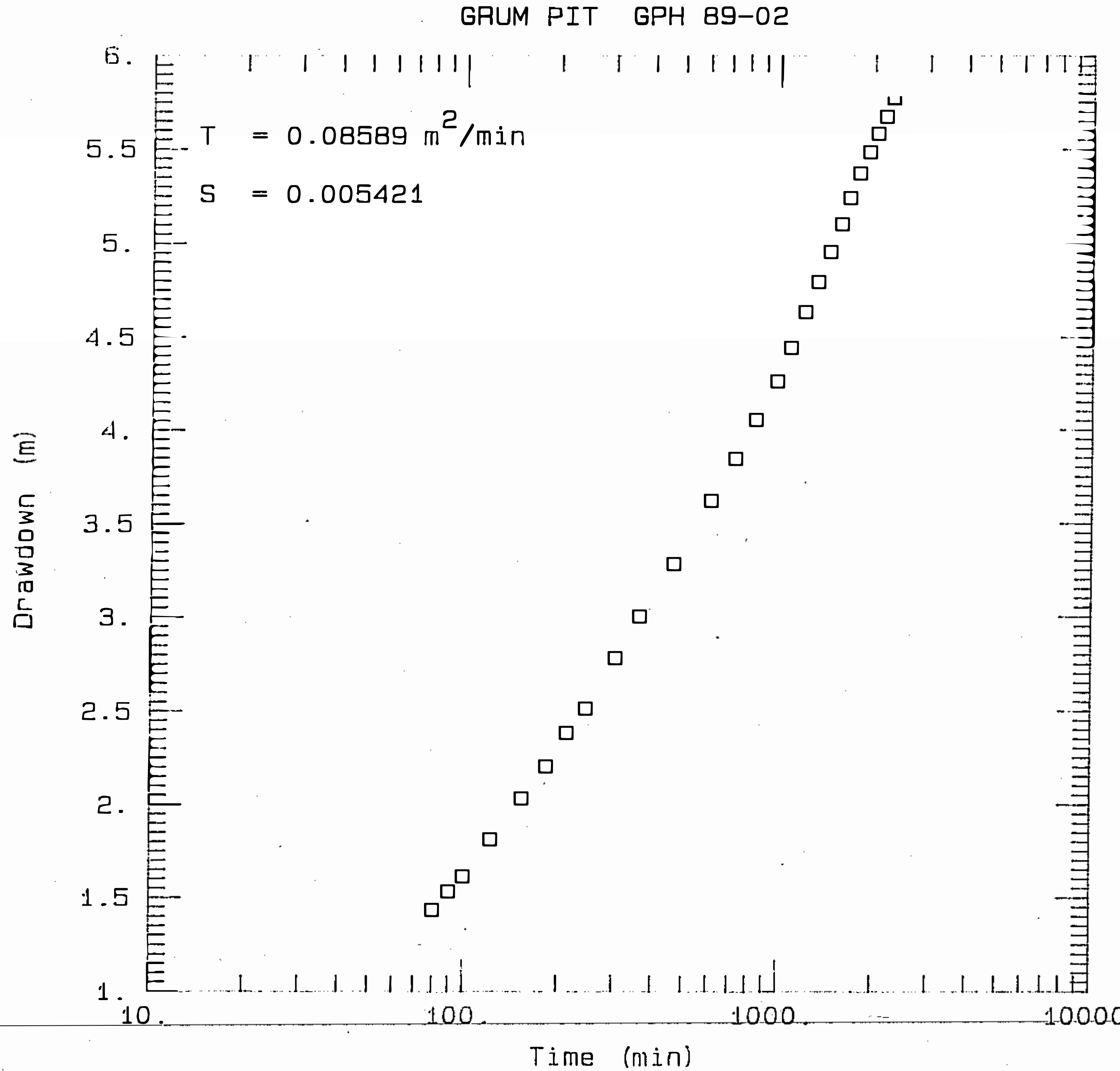
GRUM PIT GPH 89-01
T = 0.4827 m2/min
S = 2.7832E-004
100.
10.
1.
0.1
1.E-002
Drawdown (m)
1.
10.
100.
1000.
10000
Time (min)

PLOT A-3

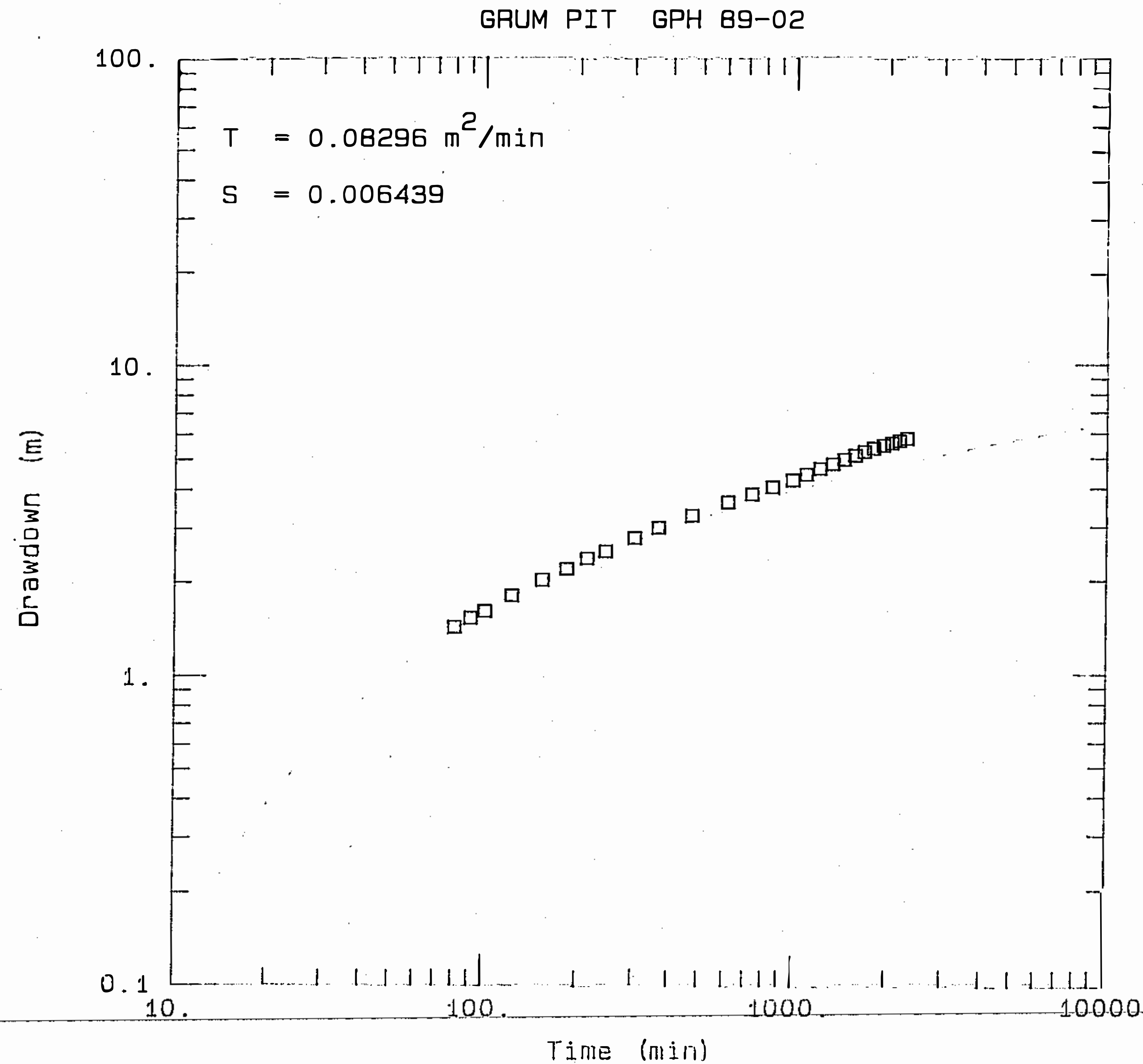


GRUM PIT GPH 89-01
T = 0.5108 m2/min
S = 2.872E-004
B = 1.E-003
Drawdown (m)
100.
10.
1.
0.1
1.E-002
1.
10.
100.
1000.
10000
Time (min)

PLOT A-4

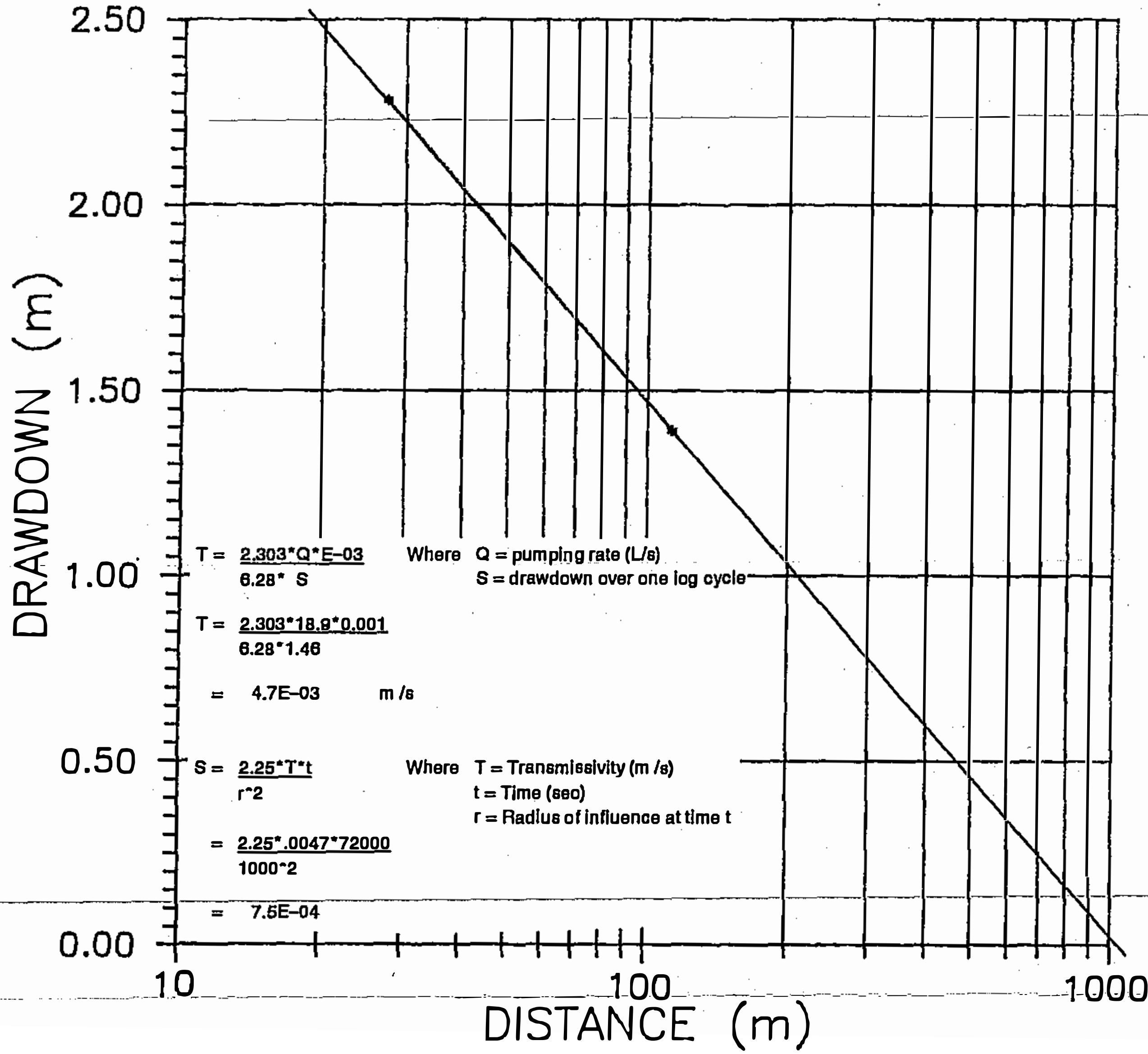

GRUM PIT GPH 89-02
T = 0.08589 m2/min
S = 0.005421
Drawdown (m)
Time (min)
6.
5.5
5.
4.5
4.
3.5
3.
2.5
2.
1.5
1.
10.
100.
1000.
10000

PLOT A-5


GRUM PIT GPH 89-02
T = 0.08296 m2/min
S = 0.006439
Drawdown (m)
100.
10.
1.
0.1
10.
100.
1000.
10000
Time (min)

PLOT A-6

DISTANCE DRAWDOWN PLOT FOR GPH 89-01 AND GPH 89-02 TIME = 460 MIN



PLOT A-7