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MISCELLANEOUS REPORTS

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REPORT

RESULTS OF PERMEABILITY TESTS AND ANALYSES OF WATER SAMPLES FROM DEEP EXPLORATION HOLES, CRANDON OREBODY CRANDON, WISCONSIN

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Prepared By

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RESULTS OF PERMEABILITY TESTS AND ANALYSES OF WATER SAMPLES FROM DEEP EXPLORATION HOLES, CRANDON OREBODY CRANDON, WISCONSIN

INTRODUCTION

PURPOSE

Tests were made in 6 deep exploration holes to assess the water-producing ability, permeability of the orebody and the rocks, and to obtain water samples for chemical analyses. These tests were performed using inflatable packers to isolate selected zones between two packers or between a single packer and the bottom of the hole. The results of the permeability tests can be used to evaluate potential water handling problems during construction and prospecting of the orebody, and during the mining operation. The water quality information will be useful in evaluating necessary treatment, if any, of water from the orebody before disposing.

SCOPE

During this investigation, Dames & Moore furnished a Lynes Injector Packer, supervised operation of the packer, made permeability tests of selected zones, and collected water samples. Exxon Minerals Company, U.S.A. selected certain zones in Holes 155, 165, 70, 66, 91 and 77 for testing. Dames & Moore was instructed not to make an extensive interpretative report.

8837-069-07

METHOD OF INVESTIGATION

The injection straddle packer is manufactured by Lynes Incorporated of Houston, Texas and owned by Dames & Moore. The packer consists of 2 water-inflatable rubber elements separated by a spacer or mandrel. The unit can be operated either as a single packer or as a straddle packer depending upon the orientation of internal valves.

In general the operation was as follows:

1 - The hole to be tested and sampled was sounded by Exxon Minerals Company, U.S.A. to determine if the hole was open to the drilled depth and to assess the condition of the fluids in the hole;

2 - The hole was investigated by borehole geophysical logging to determine hole conditions and rock and fluid characteristics;

3 - The hole was flushed by running in a string of hollow drilling rods and injecting air or water to remove the fluids from the hole. Air flushing was the preferred method;

4 - The packer assembly, predressed with new O-rings and the valves oriented so as to operate the assembly either as a single or as a straddle packer, was run (tripped) into the hole on N-rods to the lowest desired test depth for the particular hole;

5 - A Bean positive displacement pump was connected to the N-rods and water under about 800 to 1000 psi pressure was forced into the system to inflate the packer(s);

6 - The N-rods were rotated to lock the pressure into the packer elements and then raised to disengage and by-pass the valves so as to establish continuity with the borehole through a port located below the top packer;

7 - Additional pressure was applied to assess if the port was open and if the packers had inflated and sealed against the walls of the borehole. If a seal was not made, bubbles and water would rise up the borehole above the packer assembly;

8 - If this test was satisfied, the pressure was released from the N-rods and the connection with the pump removed;

9 - A string of A-rods, up to 800-900 feet long, were then placed in the N-rods, packer position permitting;

10 - The ground water levels in the A-rods were then monitored until a trend or equilibrium was established;

11 - Compressed air was injected through the A-rods under pressure to 400 psi. This creates an air-lift pumping phenomenon, whereas the water-air mist rises in the annulus between the N-rods and the A-rods. The air pressure was adjusted and possibly the A-rods lowered into the hole depending upon the productivity of the zone below the top packer and the depth of the packer. If too much air pressure was applied, a back pressure would result because of the limited annular area above the bottom of the A-rods. This back pressure would equilibrate in the hole and largely prevent water from rising through the annulus;

12 - The hole would be "pumped" by air lift for sufficient time to remove the water in the drilling rods above the packer and to withdraw water from the test zone;

13 - The quantity of water air lifted from the hole was monitored by volume changes as a function of time in a calibrated tank receiving the discharge or by catching the discharge in a bucket and timing with a stopwatch. Water samples were also collected from the discharge and the field conductivity was measured. The pH of several water samples was also measured;

14 - At the cessation of the air lift pumping, the water levels in the hole were measured through the A-rods with respect to time for a period of about 2½ hours. These data were then used to assess the permeability of a selected zone in the hole;

15 - The collected bulk water samples were filtered as soon as practical and placed in "treat" or "no treat" sample bottles and refrigerated until shipped to Aqualab of Streamwood, Illinois, a certified laboratory, for chemical analyses; and,

16 - At the completion of monitoring water level recovery, the A-rods were removed from within the N-rods, the packer released by reversing the setting procedure, and the packer assembly raised to the next test position and the method repeated.

RESULTS OF TESTS

PERMEABILITY

The air lift pumping lowered the water level from the borehole below the single packer or within the straddle packer interval. This created drawdown of water levels during the pumping. Because of the depth, small borehole diameter, air lift pumping, and types of equipment used, it was not possible to monitor the drawdown. It was possible, however, to monitor the recover of water levels as a function of time.

These water level recovery data were plotted on semi-logarithmic paper as shown on the plots of data in the Appendix. Ideally, the points should describe a straight line. In practice, however, entrance losses and casing storage effects commonly create a nonlinear water level recovery for the early time periods. According to calculations using permeability, hole and casing geometry, and an equation presented by Schafer (1978, p. 4), some casing storage effects would be in the data for the first 6 to 8 hours after recovery. It was not practical, however, to pump the test zones for long periods of time and to monitor the water level recovery for 8 to 12 hours in order to refine the data more precisely.

After plotting the water level recovery data as a function of time, the best fitting straight line was drawn through the later time data. These water level recovery plots are presented in the Appendix. The selection of the points to use was based in part on professional judgment and experience. The slope of the straight line was used as input to the modified Theis non-equilibrium equation (Ferris and others, 1962, p. 101).

$$T = \frac{264\ Q}{\Delta s}$$

where: T = transmissivity, in gallons per day per foot, is a measure of the ability of an aquifer to transmit water through a vertical column of water extending the full saturated thickness of the aquifer under a gradient of 100 percent. The transmissivity is the product of the permeability and the saturated thickness of the aquifer.

Q = rate of discharge in gallons per minute (gpm).

Δs = the change in feet in the drawdown or recovery over 1 log cycle of time.

This equation is based on the assumption that the aquifer is isotopic, homogeneous, and of infinite areal extent, the transmissivity is uniform, and the well receives water from the entire thickness of the aquifer. Obviously in the "real world" these conditions are not fully realized, but experience has shown the method to be fairly accurate.

It should be noted that Δs is usually expressed as residual drawdown which takens into account the projected drawdown at a given time after pumping ceased plus the measured recovery at that same time. Inasmuch as it was not practical to measure drawdown, the residual drawdown could not be calculated and the graph represents only the recovery of water levels expressed as depth of water below land surface. The theoretical change in residual drawdown in recovery per log cycle is somewhat larger than used in the calculations and thus the actual transmissivities, T, are probably 20 to 30 percent less than those resulting from these calculations.

Table 1 is a summary table of the results of physical tests of the aquifer properties. The permeability, an average value, was calculated by dividing the transmissivity by the length of the zone tested. The calculcated permeability range from about 10^{-7} cm/sec to 10^{-5} cm/sec. Several of the high 10^{-5} cm/sec permeability values (Hole 155, test intervals 2710-2723 feet, 2712-2725 feet, and 2202-2215 feet) are believed to be abnormally high

because of suspected leakage around the packer elements during the testing. The 10^{-5} cm/sec permeability values for the entire open hole portions of Holes 66 and 91 may be large because of possible leakage around the uncemented casing from the overlying glacial material.

In general the test results indicate that the rocks are tight and likely produce only low to moderate quantities of water from fracture or fault zones. It is somewhat likely that the fractures become tighter and have lower effective permeability with increasing depth. The test results, except those noted above, are believed to be accurate within an order of magnitude.

Any inaccuracies in data and results introduced by the methodology are considered minor because of the characteristic of the water-bearing materials. The permeability is a secondary phenomenon of fracturing and faulting of the rocks. The fracturing or faulting is not evenly distributed throughout the rock and orebody, thus significant variation in spatial permeability is expected. To more precisely identify the fracture permeability would likely entail more test holes, larger diameter test holes, and much longer periods of pumping. This more elaborate testing would require an extensive investment of time, money, equipment and personnel. It would be difficult to justify this elaborate an investigation because of the relatively low water-yielding ability of the rocks.

WATER QUALITY

Exxon Minerals Company, U.S.A. selected the water samples for analyses. The results of the water quality testing, as shown on Table 2, indicate that the pH is usually slightly basic and that there are low concentrations of heavy metals. Several water samples from the deep zones, such as the 2920-to 2971-foot interval in Hole 165 and the 2202-to 2215-foot and the 942-to 2202-foot intervals in Hole 155 contained very high concentrations of total dissolved solids, of which chlorides were notably high. This is probably the result of very little circulation of ground water in the deeper zones; it is not unlikely that some of this water is pre-glacial in age. The top of the brackish water is believed to occur below a depth estimated to be as shallow as 1500 feet to as deep as 2200 feet.

BIBLIOGRAPHY

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Schaefer, D.C., 1978, Casing Storage Can Affect Pumping Test Data: Johnson Drillers Journal, Jan.-Feb.

TABLE 1

SUMMARY OF HOLE CONSTRUCTION AND TEST RESULTS

HOLE NUMBER	HOLE CONSTRUCTION: INTERVAL TESTED (FEET)	HOLE CONSTRUCTION: TOTAL DEPTH	HOLE CONSTRUCTION: CASING	RECOVERY TEST SUMMARY: ACTIVITY	RECOVERY TEST SUMMARY: TOTAL PUMPING TIME (MINUTES)	RECOVERY TEST SUMMARY: TOTAL PUMPAGE GALLONS	RECOVERY TEST SUMMARY: AVERAGE RATE OF PUMPING (GPM)	RECOVERY TEST SUMMARY: CALCULATED PERMEABILITY (CM/SEC)	WATER SAMPLE SUMMARY: TIME DATE	WATER SAMPLE SUMMARY: TEMP °C	WATER SAMPLE SUMMARY: COND UMHOS	WATER SAMPLE SUMMARY: pH	WATER SAMPLE SUMMARY: GALLONS PUMPED
165	1600-2971	2971	4" casing to 1600 feet. HQ (3.78 inch) hole 1600 to 2971 feet.	2/16 & 2/17 Open hole blow out using 800 feet of A-rods.	346	3010	8.7	1.4×10^{-6}					
	2920-2971		Casing not cemented.	2/17 & 2/18 Recovery test at 2970 feet using single packer.	35	120	3.4 (0.25 sustained)	1.2×10^{-7}	0128 2-18-78	7	4400		120
	2297-2971			2/19 Recover test at 2300 feet using single packer.	90	465	5.2 (0.25 sustained)	5.8×10^{-7}	2000 2-19-78	9	4400		351
									2050 2-19-78	7	4700		465
155	161-3050	3050	4.5" casing to 161 feet not cemented. HQ hole 161 to 3050 feet.	2/21 Open hole blow out. Started with 780 feet of N-rods. Added rods to 1260 feet.	238	5000	21	5.8×10^{-7}					
	2710-2723			2/22 Recovery test using straddle packer on interval from 2710 to 2723 feet.	157 193	723	4.6	6.0×10^{-5}*	1540 2-22-78	16	4290		709
	2712-2725			2/24 Recovery test using straddle packer on interval from 2712 to 2725 feet.	100	838	8.4	6.3×10^{-5}*	0835 2-24-78	11.5	4700		838
	2202-2215			2/23 Recovery test using straddle packer on interval from 2201 to 2215 feet.	296	1101	3.75	3.8×10^{-5}	2035 2-23-78	11.5	4700		1101

*Some leakage around packers on these tests because of open fractures.

TABLE 1 - Continued

HOLE NUMBER	HOLE CONSTRUCTION: INTERVAL TESTED (FEET)	HOLE CONSTRUCTION: TOTAL DEPTH	HOLE CONSTRUCTION: CASING	RECOVERY TEST SUMMARY: ACTIVITY	RECOVERY TEST SUMMARY: TOTAL PUMPING TIME (MINUTES)	RECOVERY TEST SUMMARY: TOTAL PUMPAGE GALLONS	RECOVERY TEST SUMMARY: AVERAGE RATE OF PUMPING (GPM)	RECOVERY TEST SUMMARY: CALCULATED PERMEABILITY (CM/SEC)	WATER SAMPLE SUMMARY: TIME DATE	WATER SAMPLE SUMMARY: TEMP °C	WATER SAMPLE SUMMARY: COND UMHOS	WATER SAMPLE SUMMARY: pH	WATER SAMPLE SUMMARY: GALLONS PUMPED
155 (Cont.)	2102-3050	3050	4.5" casing to 161 feet not cemented. HQ hole 161 to 3050 feet.	2/25/78 Recovery test using a single packer set at 2102 feet.	145	1023	7.1	7.9×10^{-7}	0755 2-25-78	10.5	5000		1023
	1372-3050			2/25/78 Recovery test using a single packer set at 1372 feet.	176	1300	7.4	5.4×10^{-7}	2010 2-25-78	12.0	6000		1300
	1042-3050			2/26/78 Recovery test using a single packer set at 1042 feet.	180	1344	7.5	4.8×10^{-7}	2000 2-26-78	10.5	6800		1344
	942-3050			2/27/78 Recovery test using a single packer set at 942 feet.	183	1310	7.2	3.9×10^{-7}	0715 2-27-78	12.5	7500		1310
	842-3050			2/27/78 Recovery test using a single packer set at 842 feet. 740 feet of A-rods used.	188	1228	6.5	4.7×10^{-7}	1700 2-27-78	14.2	8800		1228
	562-3050			2/28/78 Recovery test using a single packer set at 562 feet. 500 feet of A-rods used.	167	975	5.8	4.3×10^{-7}	0159 2-28-78	9.0	7700		940
	362-3050			2/28/78 Recovery test using a single packer set at 362 feet. 300 feet of A-rods used.	199	645	3.2	4.9×10^{-7}	1315 2-28-78	9.8	700		625

TABLE 1 - CONTINUED

HOLE NUMBER	INTERVAL TESTED (FEET)	HOLE CONSTRUCTION TOTAL DEPTH	CASING	ACTIVIY	RECOVERY TEST SUMMARY TOTAL PUMPING TIME (MINUTES)	TOTAL PUMPAGE GALLONS	AVERAGE RATE OF PUMPING (GPM)	CALCULATED PERMEABILITY (CM/SEC)	WATER SAMPLE SUMMARY TIME DATE	TEMP °C	COND UMHOS	pH	GALLONS PUMPED
155 (Cont.)	161-3050	3050	4.5" casing to 161 feet not cemented. HQ hole 161 to 3050 feet.	2/28/78 Attempted recovery test with packer at 161 feet failed due to lack of sufficient discharge.									
77	261-1049	1049	4" casing to to 151 feet, HQ casing from 151-261 feet, and HQ hole 261 to 1049. Casings are not cemented.	3/01/78 Open hole blow out starting with 850 feet of A-rods in the hole and added rods to 990 feet.	120 @ 850' 40 @ 910' 90 @ 990' 250 total	Unable to measure discharge due to lack of appropriate collection equipment.			0845 3-01-78		275		
	602-1049			3/01/78 Recovery test using a single packer set at 602 feet.	170	634	3.7	5.7×10^{-6}	1940	12.5	278		604
66	228-1078	1078	NW casing to 228 feet and NQ hole 228 to 1078 feet. Casing not cemented.	3/2/78 Open hole blow out starting with 850 feet of A-rod in the hole adding rods in 3 steps to 1060 feet in the hole.	40 @ 850' 40 @ 910' 20 @ 990' 84 @ 1060' 184 total	738 @ 850' 641 @ 910' 389 @ 990' 1676@1060' 3444 total	18.7	1.8×10^{-5}	1615 3-02-78	9.0	200		3400
	402-1078			3/2 & 3/3/78 Recovery test using a single packer set at 402 feet	71	419	5.9	9.0×10^{-6}	0013 3-03-78	8.5	200		419

TABLE 1 - Continued

HOLE NUMBER	INTERVAL TESTED (FEET)	HOLE CONSTRUCTION: TOTAL DEPTH	HOLE CONSTRUCTION: CASING	HOLE CONSTRUCTION: ACTIVITY	RECOVERY TEST SUMMARY: TOTAL PUMPING TIME (MINUTES)	RECOVERY TEST SUMMARY: TOTAL PUMPAGE GALLONS	RECOVERY TEST SUMMARY: AVERAGE RATE OF PUMPING (GPM)	RECOVERY TEST SUMMARY: CALCULATED PERMEABILITY (CM/SEC)	WATER SAMPLE SUMMARY: TIME DATE	WATER SAMPLE SUMMARY: TEMP °C	WATER SAMPLE SUMMARY: COND UMHOS	WATER SAMPLE SUMMARY: pH	WATER SAMPLE SUMMARY: GALLONS PUMPED
70	176-1334	1334	4" casing to 165 feet, HQ casing from 165 to 176 feet, and NQ hole from 176 to 1334 feet. Casings not cemented.	3/4/78 Open hole blow out starting with 850 feet of A-rods in the hole and adding rods in 4 steps to 1190 feet.	77 @ 850' 78 @ 950' 46 @ 1050' 63 @ 1150' 52 @ 1190' 316 total	1434 @ 850' 1100 @ 950' 843 @ 1150' 1253 @ 1150' 898 @ 1190' 5528 total	17.5	inconclusive	1155 3-04-78	9.7	231	8.80	5528
	552-1334			3/4/78 Recovery test using a single packer set at 552 feet.	127	890	7.0	8.6×10^{-6}	2245 3-04-78	8.3	220	8.05	877
	302-1334			3/5/78 Recover test using a single packer set at 302 feet.	136	538	4.0	1.2×10^{-5}	0615 3-04-78	6.0	200	8.15	520
91	179-1415	1415	4" casing to 73 feet. HQ casing from 73 to 179 feet and NQ hole from 179 to 1415 feet. Casings not cemented.	3/5/78 Open hole blow out with 850 feet of A-rod in the holes.	109	3435	31.5	1.3×10^{-5}	1913 3-05-78	9.6	205	8.05	3200
	852-1415			3/6/78 Recovery test using a single packer at 852 feet.	73	1350	18.5	1.9×10^{-5}	0405 3-06-78	8.0	225	8.10	1270

TABLE 2

RESULTS OF CHEMICAL ANALYSES OF GROUND WATER SAMPLES

HOLE NUMBER	66	70	70*	70	77
ZONES (FEET)	402' - 1078'	552' - 1334'	552' - 1334'	302' - 1334'	602' - 1049'
DATE/TIME COLLECTED	3/2/78 2413 Hr.	3/4/78 2245 Hr.	3/4/78 2245 Hr.	3/5/78 0615 Hr.	3/1/78 1940 Hr.
pH	8.11 @25°C	7.98 @25°C	7.99 @25°C	7.83 @25°C	7.75 @25°C
Alkalinity	142. mg/l	160. mg/l	160. mg/l	156. mg/l	134. mg/l
Hardness	175. mg/l	194. mg/l	194. mg/l	193. mg/l	170. mg/l
Solids, Tot. Dis.	198. mg/l	170. mg/l	178. mg/l	188. mg/l	170. mg/l
Conductivity	289. µmhos	312. µmhos	310. µmhos	295. µmhos	298. µmhos
Nitrogen, Nitrate	0.34 mg/l	0.15 mg/l	0.15 mg/l	0.24 mg/l	<0.05 mg/l
Phosphorus	0.01 mg/l	0.08 mg/l	0.08 mg/l	0.01 mg/l	0.01 mg/l
Chloride	2. mg/l	8. mg/l	8. mg/l	6. mg/l	20. mg/l
Sulfate	18. mg/l	16. mg/l	15. mg/l	8. mg/l	2. mg/l
Iron	0.02 mg/l	0.02 mg/l	0.02 mg/l	0.03 mg/l	0.02 mg/l
Manganese	0.121 mg/l	0.119 mg/l	0.120 mg/l	0.090 mg/l	0.102 mg/l
Magnesium	19.3 mg/l	23.4 mg/l	23.4 mg/l	22.8 mg/l	20.5 mg/l
Zinc	0.042 mg/l	0.062 mg/l	0.060 mg/l	0.045 mg/l	0.041 mg/l
Cadmium	<0.001 mg/l	0.009 mg/l	0.008 mg/l	0.003 mg/l	0.004 mg/l
Copper	0.015 mg/l	0.006 mg/l	0.005 mg/l	0.001 mg/l	0.003 mg/l
Arsenic	0.017 mg/l	0.008 mg/l	0.009 mg/l	0.004 mg/l	<0.001 mg/l
Aluminum	0.07 mg/l	<0.01 mg/l	<0.01 mg/l	0.10 mg/l	0.11 mg/l
Chromium	<0.001 mg/l	<0.001 mg/l	<0.001 mg/l	<0.001 mg/l	<0.001 mg/l
Cobalt	0.01 mg/l	<0.01 mg/l	<0.01 mg/l	<0.01 mg/l	<0.01 mg/l
Lead	0.03 mg/l	<0.01 mg/l	<0.01 mg/l	0.03 mg/l	0.04 mg/l
Mercury	<0.0001 mg/l	<0.0001 mg/l	<0.0001 mg/l	<0.0001 mg/l	<0.0001 mg/l
Molybdenum	<0.01 mg/l	<0.01 mg/l	<0.01 mg/l	<0.01 mg/l	<0.01 mg/l
Nickel	0.01 mg/l	0.01 mg/l	0.01 mg/l	0.01 mg/l	<0.01 mg/l
Selenium	0.016 mg/l	0.009 mg/l	0.009 mg/l	0.007 mg/l	<0.001 mg/l
Calcium	38.6 mg/l	39.1 mg/l	39.2 mg/l	39.5 mg/l	34.6 mg/l
Cyanide	<0.001 mg/l	<0.001 mg/l	<0.001 mg/l	<0.001 mg/l	<0.001 mg/l
Thallium	<0.25 mg/l	<0.25 mg/l	<0.25 mg/l	<0.25 mg/l	<0.25 mg/l

*Quality assurance check duplicate sample analyses.

TABLE 2

RESULTS OF CHEMICAL ANALYSES OF GROUND WATER SAMPLES

HOLE NUMBER	91	155	155	165	
ZONES (FEET)	852' - 1415'	2202' - 2215'	942' - 3050'	2920' - 2971'	
DATE/TIME COLLECTED	3/6/78 0405 Hr.	2/23/78 2035 Hr.	2/27/78 1700 Hr.	2/18/78 0128 Hr.	
pH	8.12 @25°C	7.36 @25°C	7.27 @25°C	5.89 @25°C	
Alkalinity	164.	86.	80.	12.	mg/l
Hardness	196.	961.	1,532.	1,930.	mg/l
Solids, Tot. Dis.	164.	3,524.	6,112.	5,194.	mg/l
Conductivity	313.	6,500.	10,000.	7,200.	µmhos
Nitrogen, Nitrate	0.15	2.29	<0.05	<0.05	mg/l
Phosphorus	0.05	0.06	0.01	0.02	mg/l
Chloride	12.	2,200.	3,600.	2,650.	mg/l
Sulfate	5.	5.	8.	<1.	mg/l
Iron	0.03	2.36	1.15	77.	mg/l
Manganese	0.146	0.421	0.526	0.306	mg/l
Magnesium	22.0	75.0	127.	185.	mg/l
Zinc	0.048	0.051	0.144	1.46	mg/l
Cadmium	<0.001	0.007	0.012	0.027	mg/l
Copper	<0.001	0.004	0.006	0.269	mg/l
Arsenic	0.012	<0.001	<0.001	0.004	mg/l
Aluminum	0.02	0.48	0.14	5.86	mg/l
Chromium	<0.001	0.010	0.003	0.018	mg/l
Cobalt	<0.01	<0.01	0.02	0.04	mg/l
Lead	0.03	0.10	0.03	0.39	mg/l
Mercury	<0.0001	<0.0001	0.0042	0.0003	mg/l
Molybdenum	<0.01	<0.01	<0.01	0.02	mg/l
Nickel	0.01	0.05	0.05	0.34	mg/l
Selenium	0.001	<0.001	<0.001	<0.001	mg/l
Calcium	42.2	261.	404.	468.	mg/l
Cyanide	<0.001	<0.001	<0.001	0.003	mg/l
Thallium	<0.25	<0.25	<0.25	<0.25	mg/l

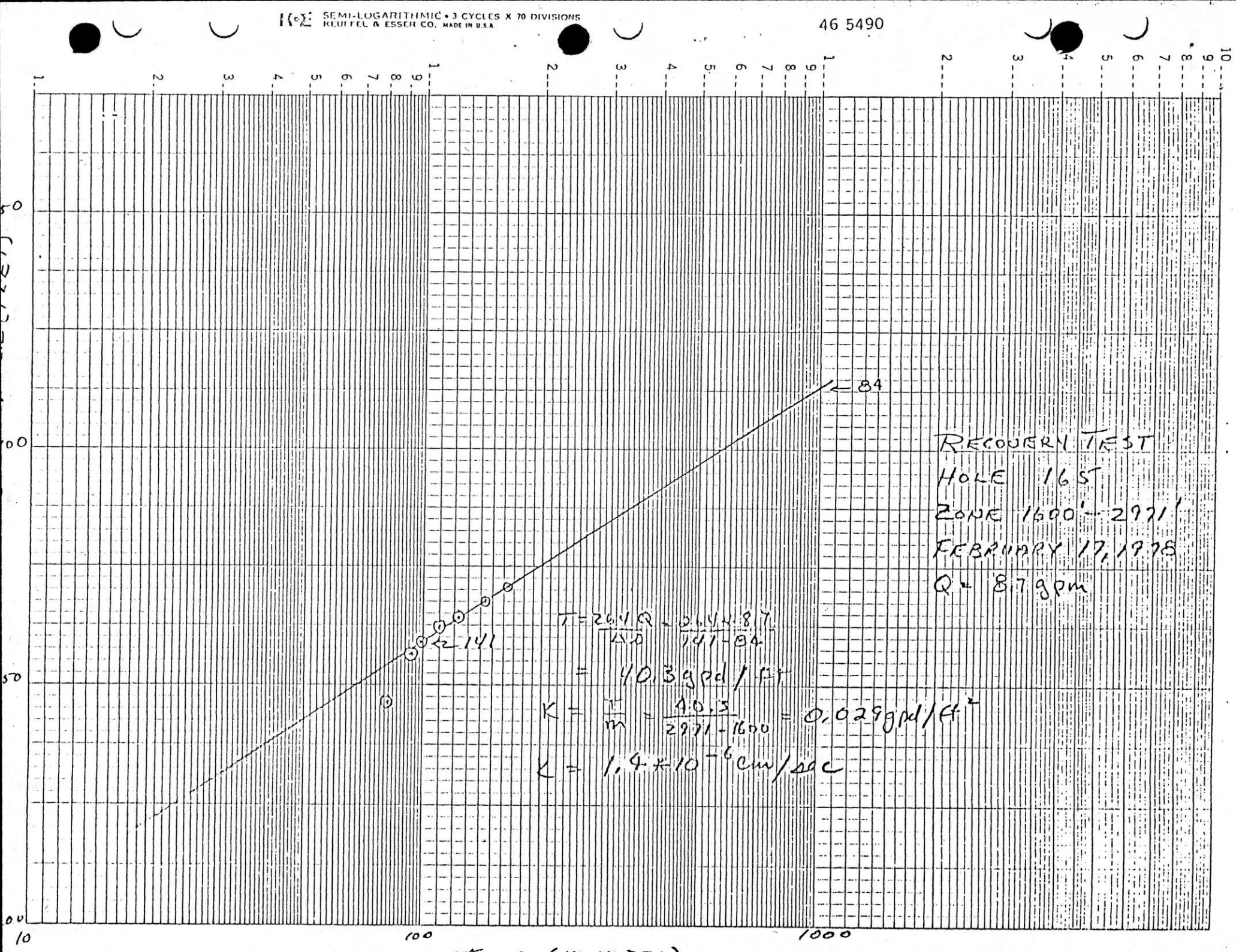
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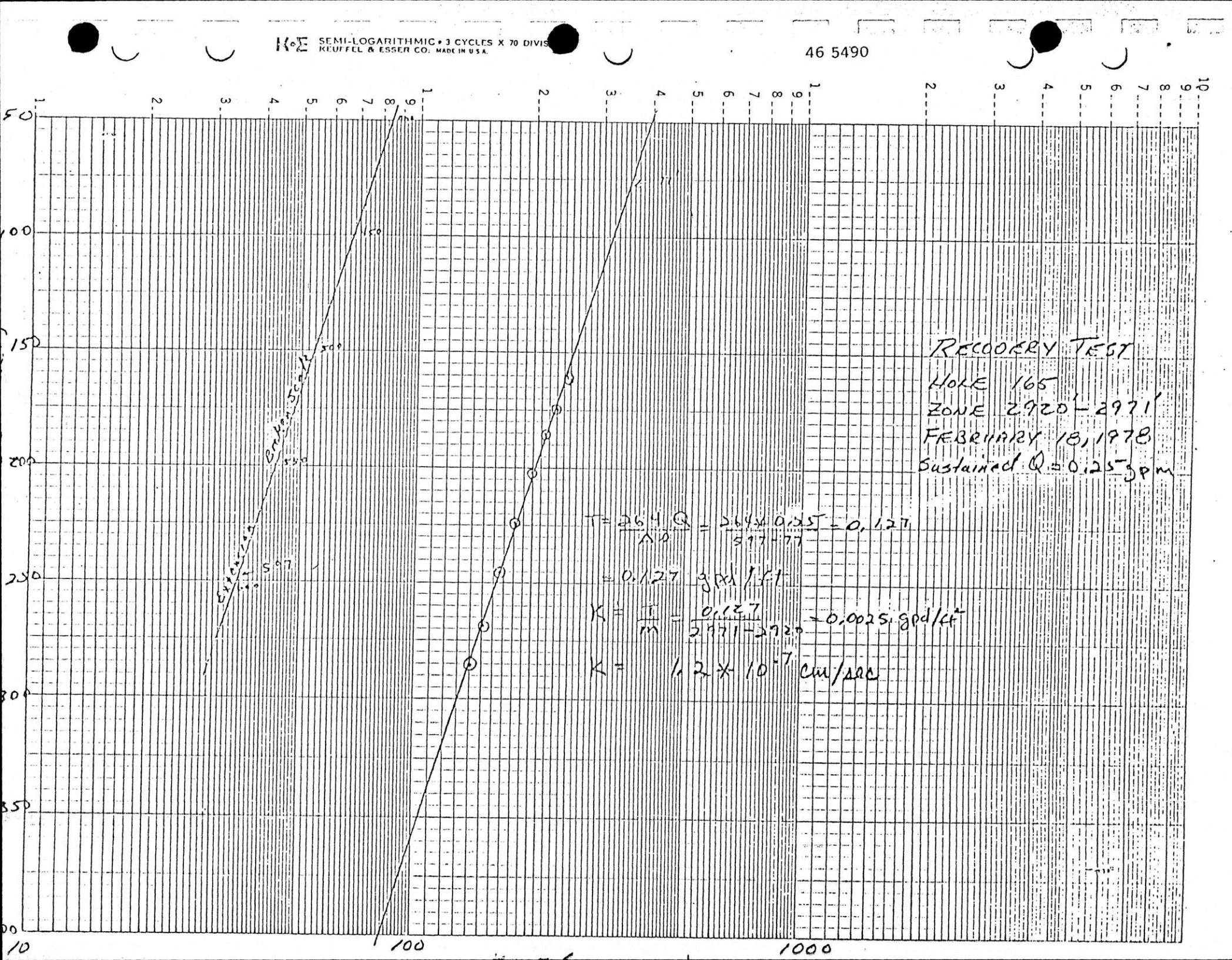
WATER LEVEL RECOVERY DATA

HOLE NUMBER	INTERVAL TESTED
165	1600'-2971'
165	2920'-2971'
165	2297'-2971'
155	161'-3050'
155	2710'-2723'
155	2712'-2725'
155	2202'-2215'
155	2102'-3050'
155	1372'-3050'
155	1042'-3050'
155	942'-3050'
155	842'-3050'
155	562'-3050'
155	362'-3050'
77	602'-1049'
66	228'-1078'
66	402'-1078'
70	176'-1334'
70	552'-1334'
70	302'-1334'
91	179'-1415'
91	852'-1415'

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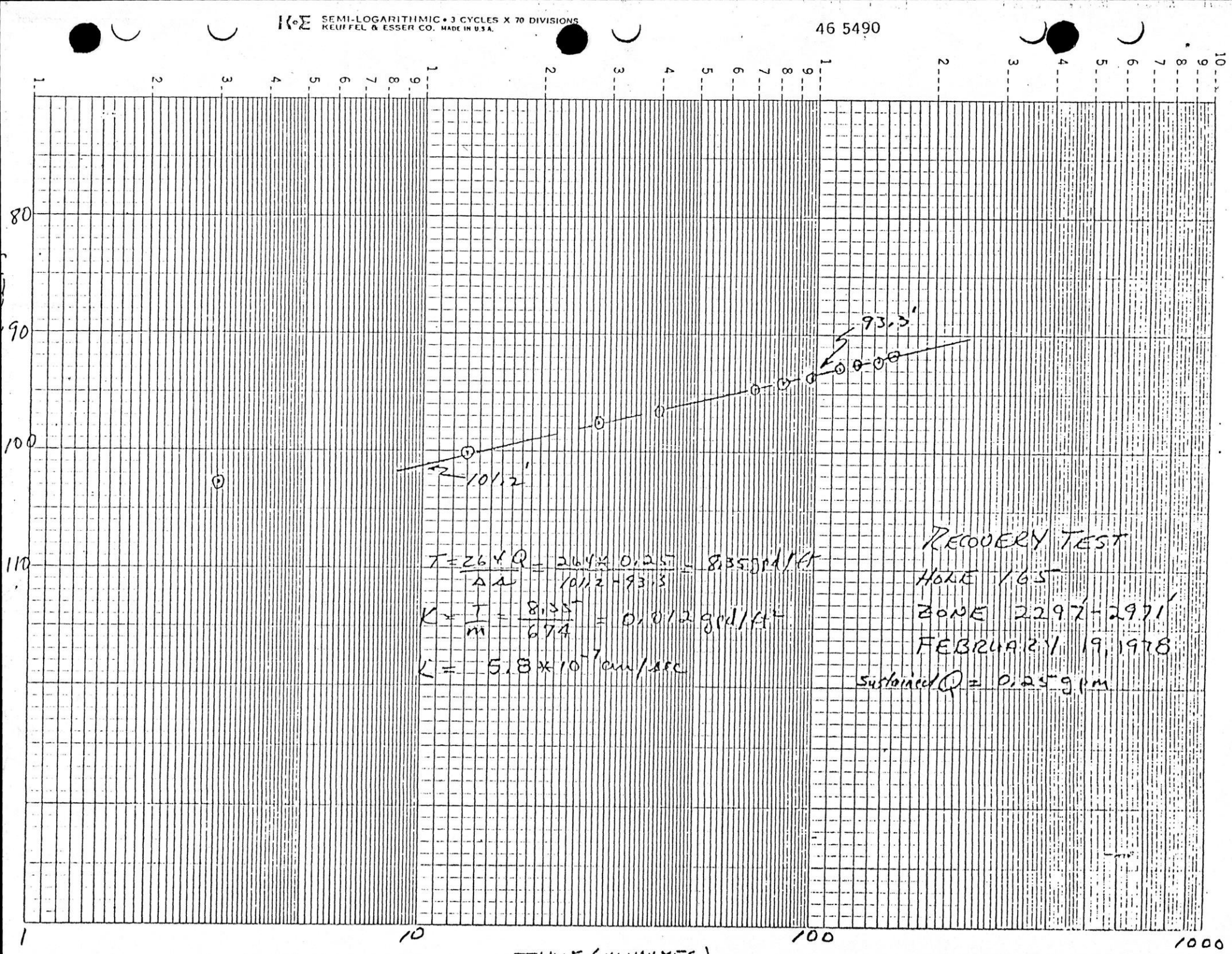
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RECOVERY TEST
HOLE 165
ZONE 2920'-2971'
FEBRUARY 18, 1978
Sustained Q = 0.25 gpm
T = 264 Q/Δs = 264 × 0.25 / 517-77 = 0.127
= 0.127 gpd/ft
K = T/m = 0.127 / 2971-2920 = 0.0025 gpd/ft²
K = 1.2 × 10⁻⁷ cm/sec
10
100
1000

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$T = \frac{264 Q}{\Delta s} = \frac{264 \times 0.25}{101.2 - 93.3} = 8.35 \text{ gpd/ft}$

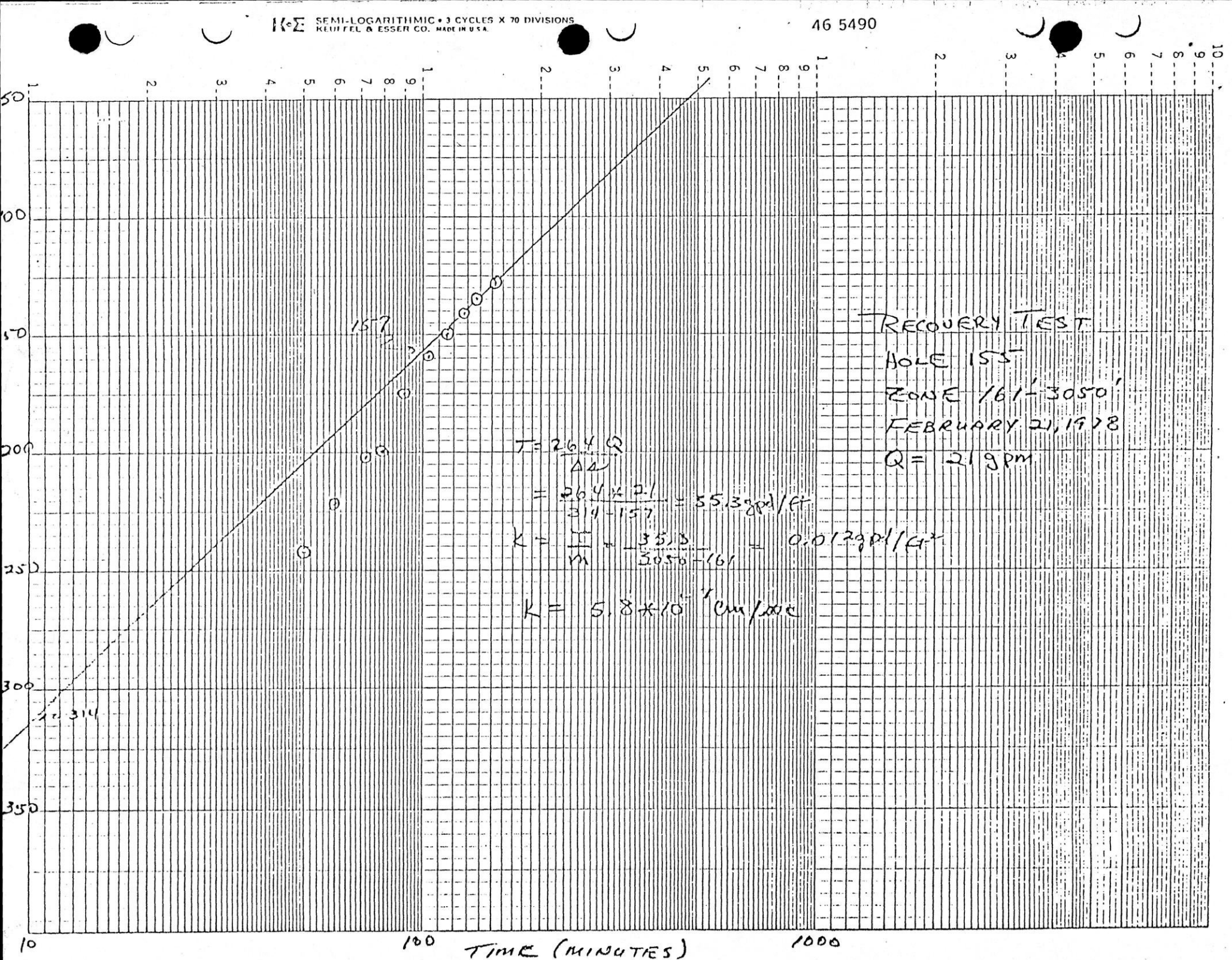
$K = \frac{T}{m} = \frac{8.35}{674} = 0.0129 \text{ gpd/ft}^2$

$K = 5.8 \times 10^{-7} \text{ cm/sec}$

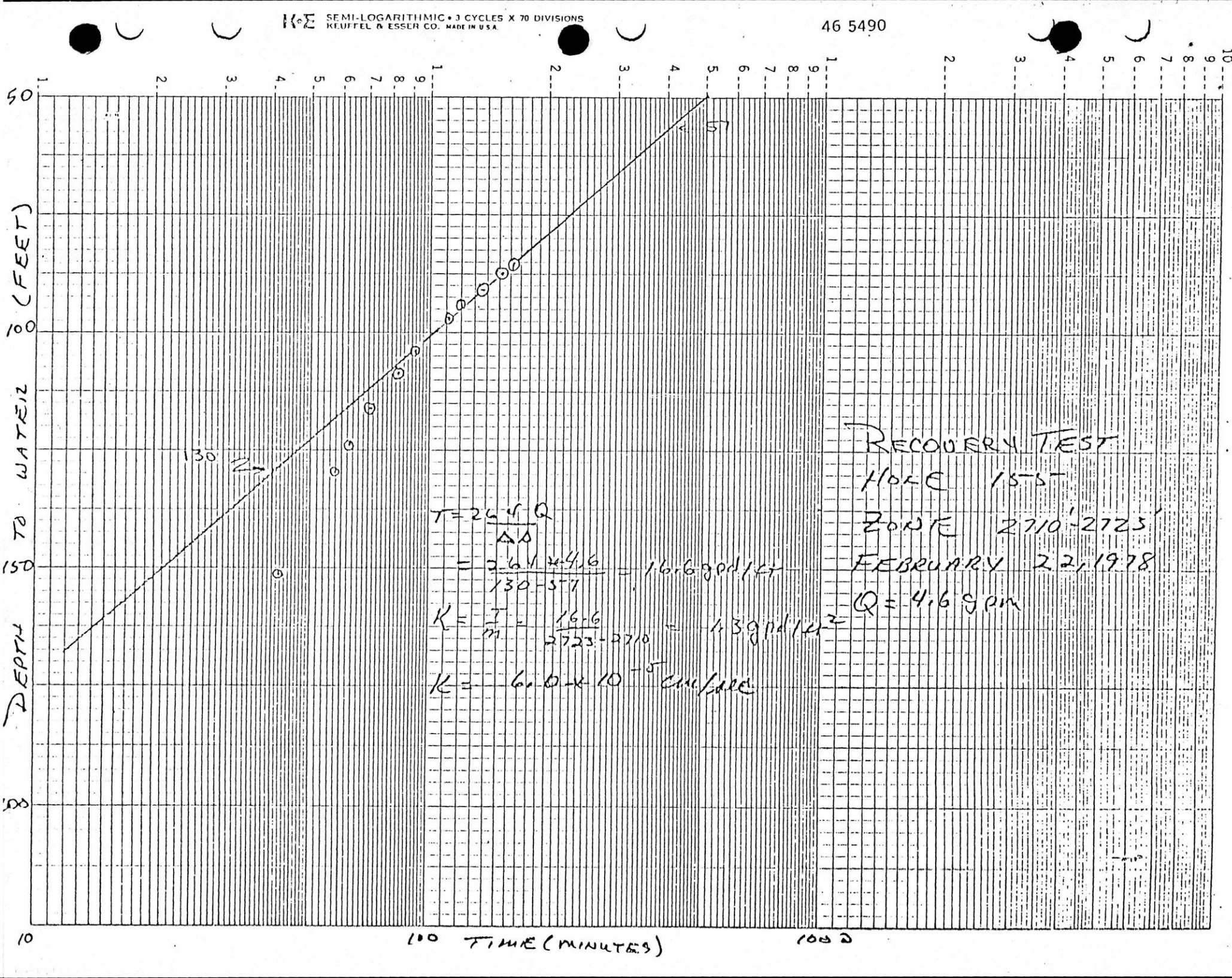
RECOVERY TEST
HOLE 165
ZONE 2297-2971'
FEBRUARY 19, 1976
Sustained Q = 0.25 gpm

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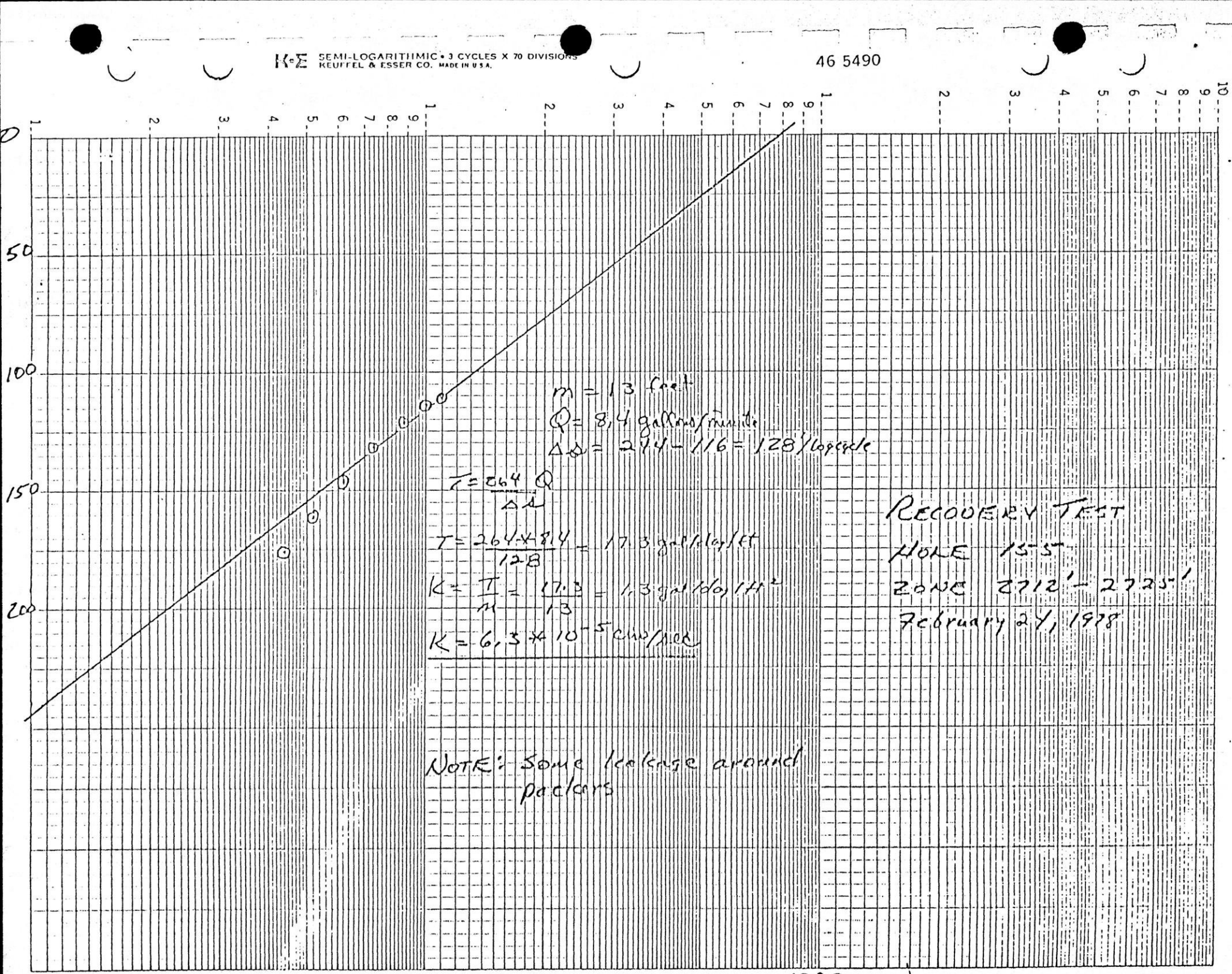


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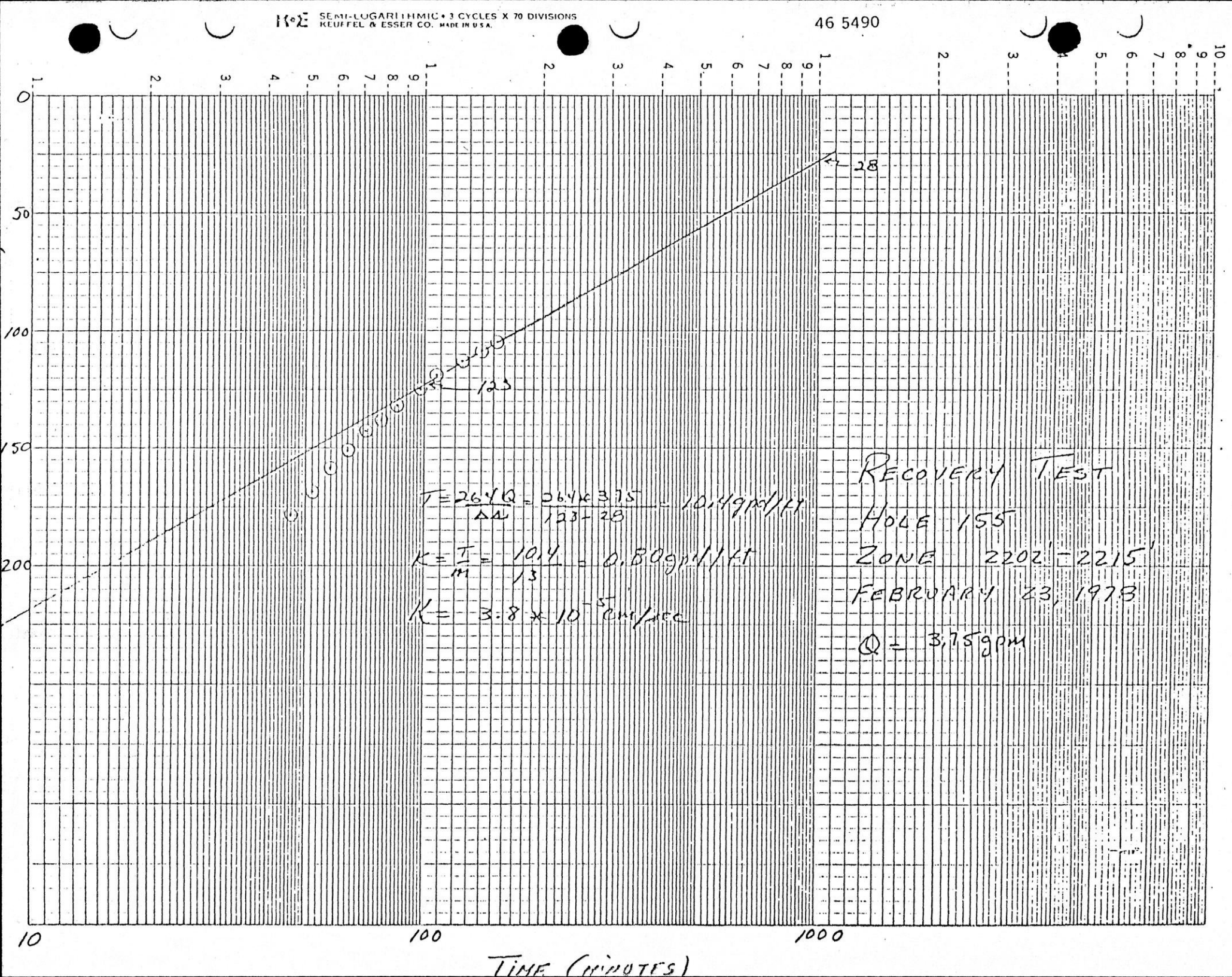


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$$T = \frac{26.4Q}{\Delta h} = \frac{26.4 \times 3.75}{123 - 28} = 10.4 \text{ gpd/ft}$$

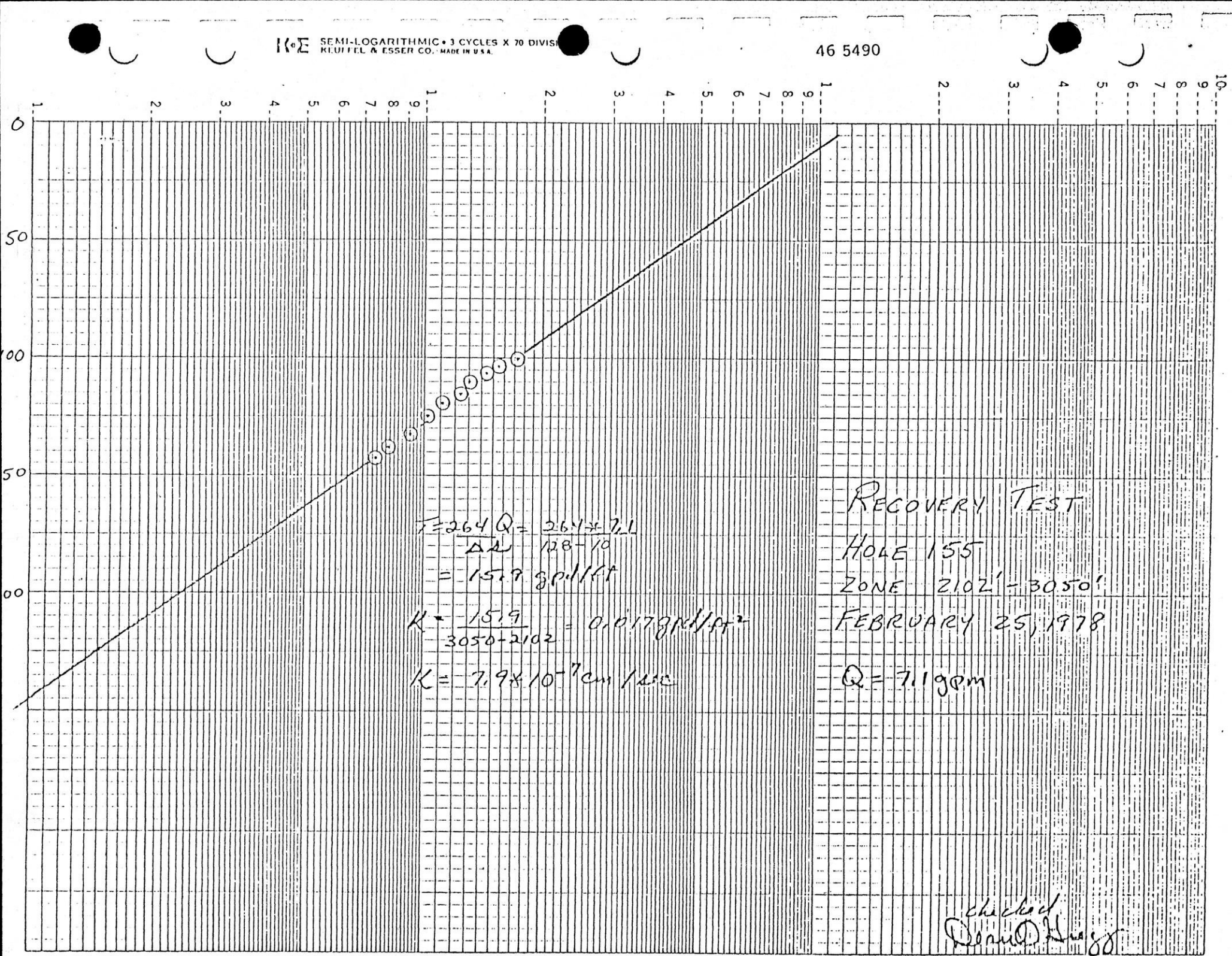
$$K = \frac{T}{m} = \frac{10.4}{13} = 0.80 \text{ gpd/ft}^2$$

$$K = 3.8 \times 10^{-5} \text{ cm/sec}$$

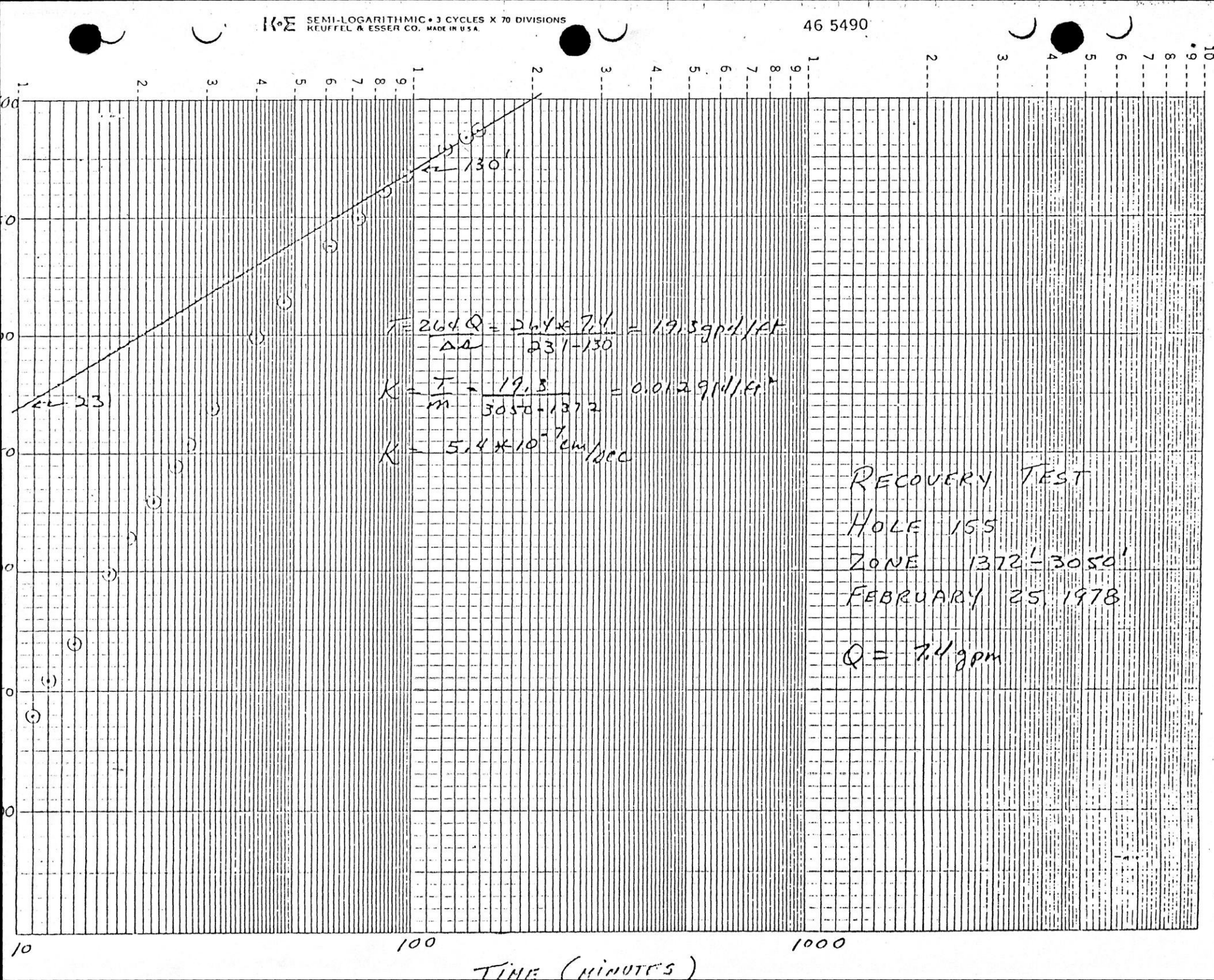
RECOVERY TEST
HOLE 155
ZONE 2202' - 2215'
FEBRUARY 23, 1978
Q = 3.75 gpm

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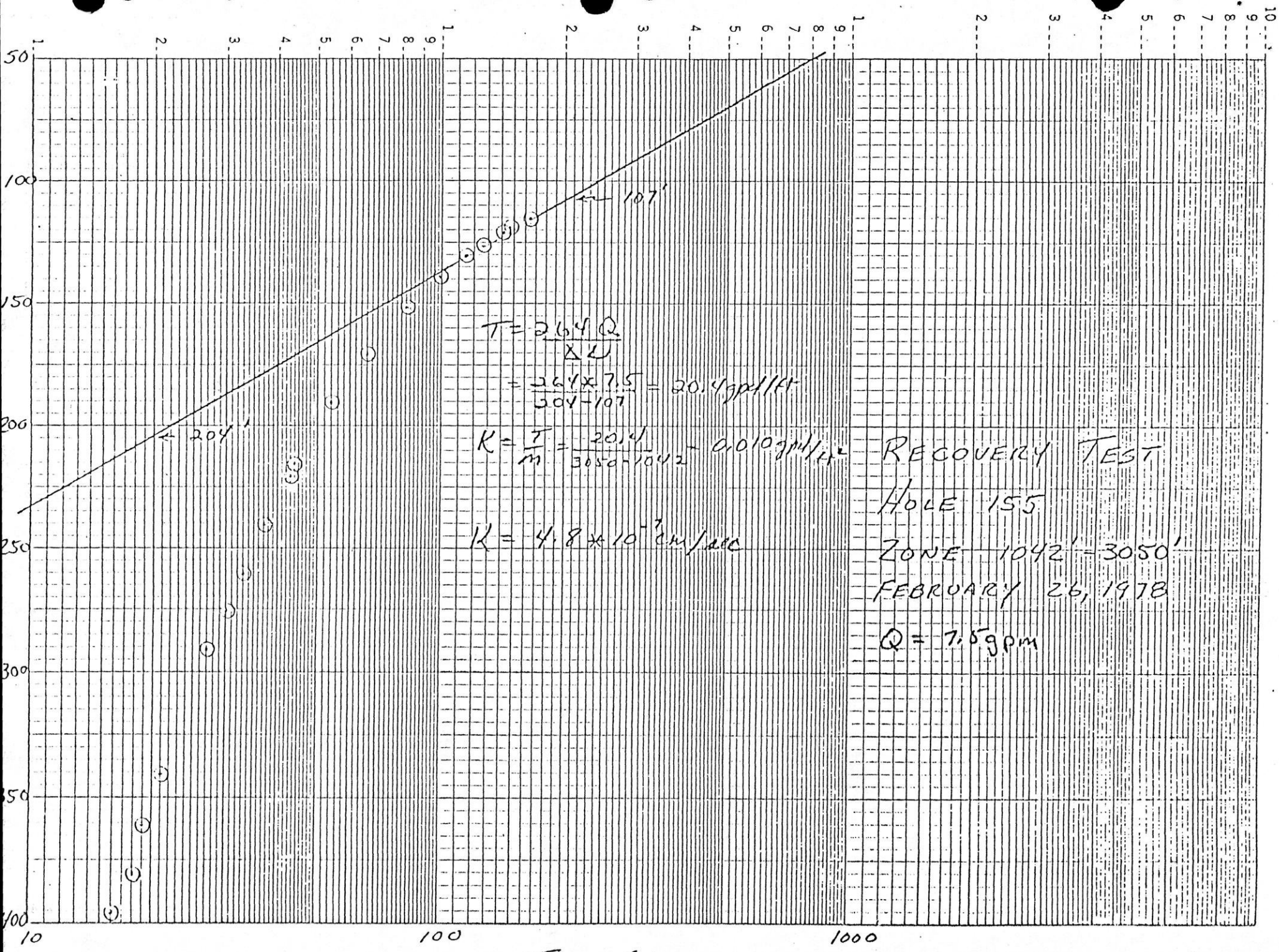
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$$T = \frac{264\,Q}{\Delta s}$$

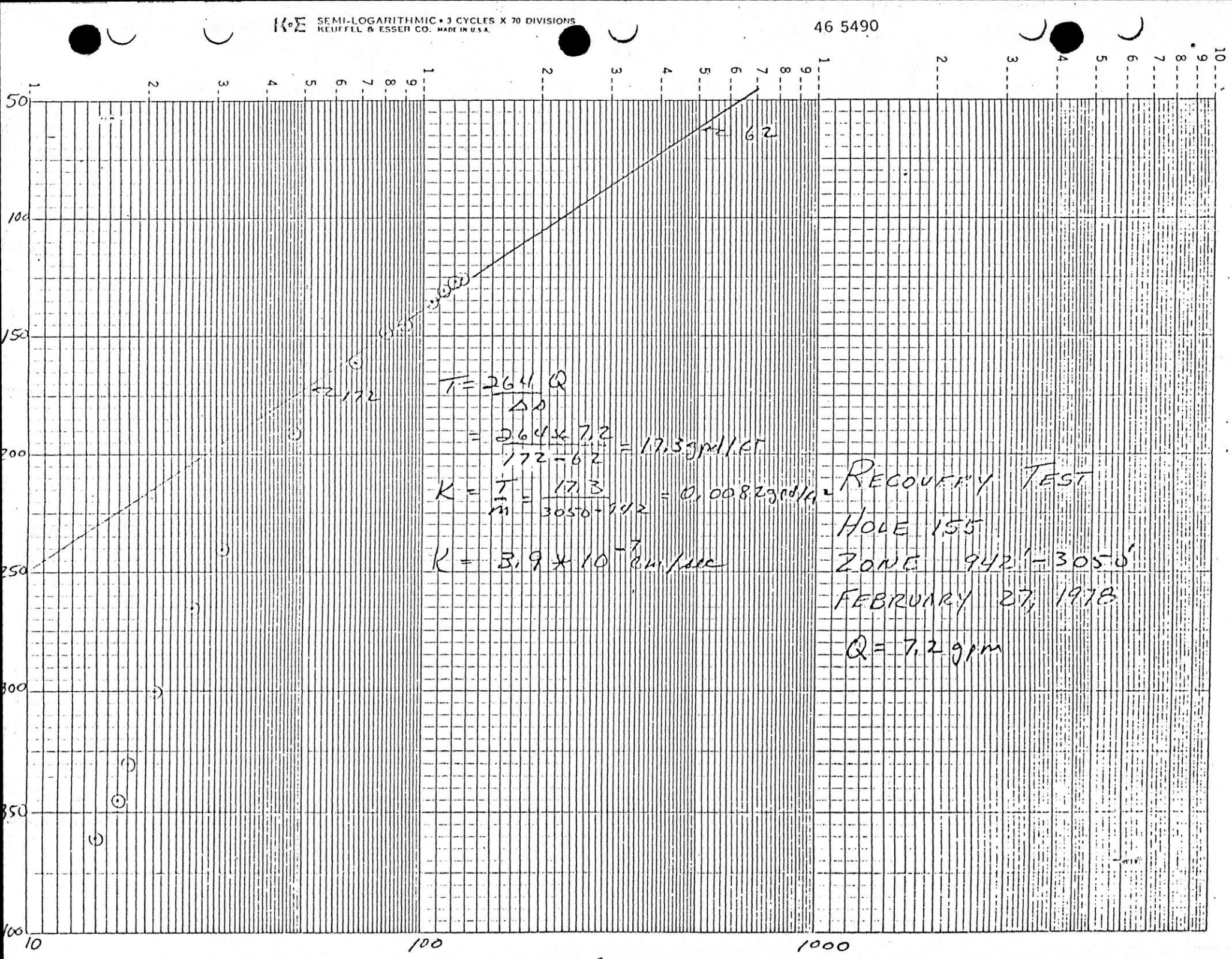
$$= \frac{264 \times 7.5}{304 - 107} = 20.4 \text{ gpd/ft}$$

$$K = \frac{T}{m} = \frac{20.4}{3050 - 1042} = 0.010 \text{ gpd/ft}^2$$

$$K = 4.8 * 10^{-3} \text{ cm/sec}$$

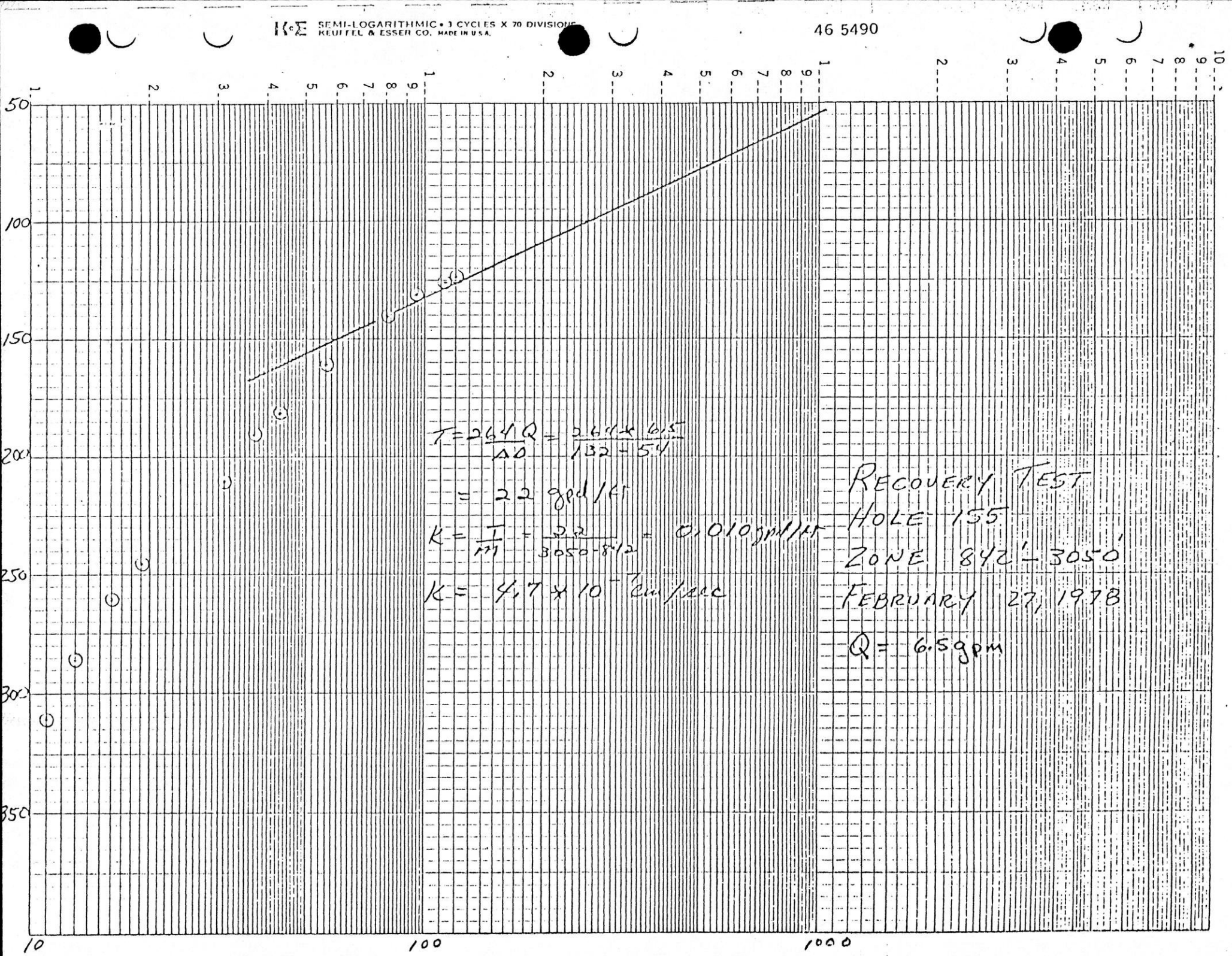
RECOVERY TEST
HOLE 155
ZONE 1042' - 3050'
FEBRUARY 26, 1978
Q = 7.5 gpm

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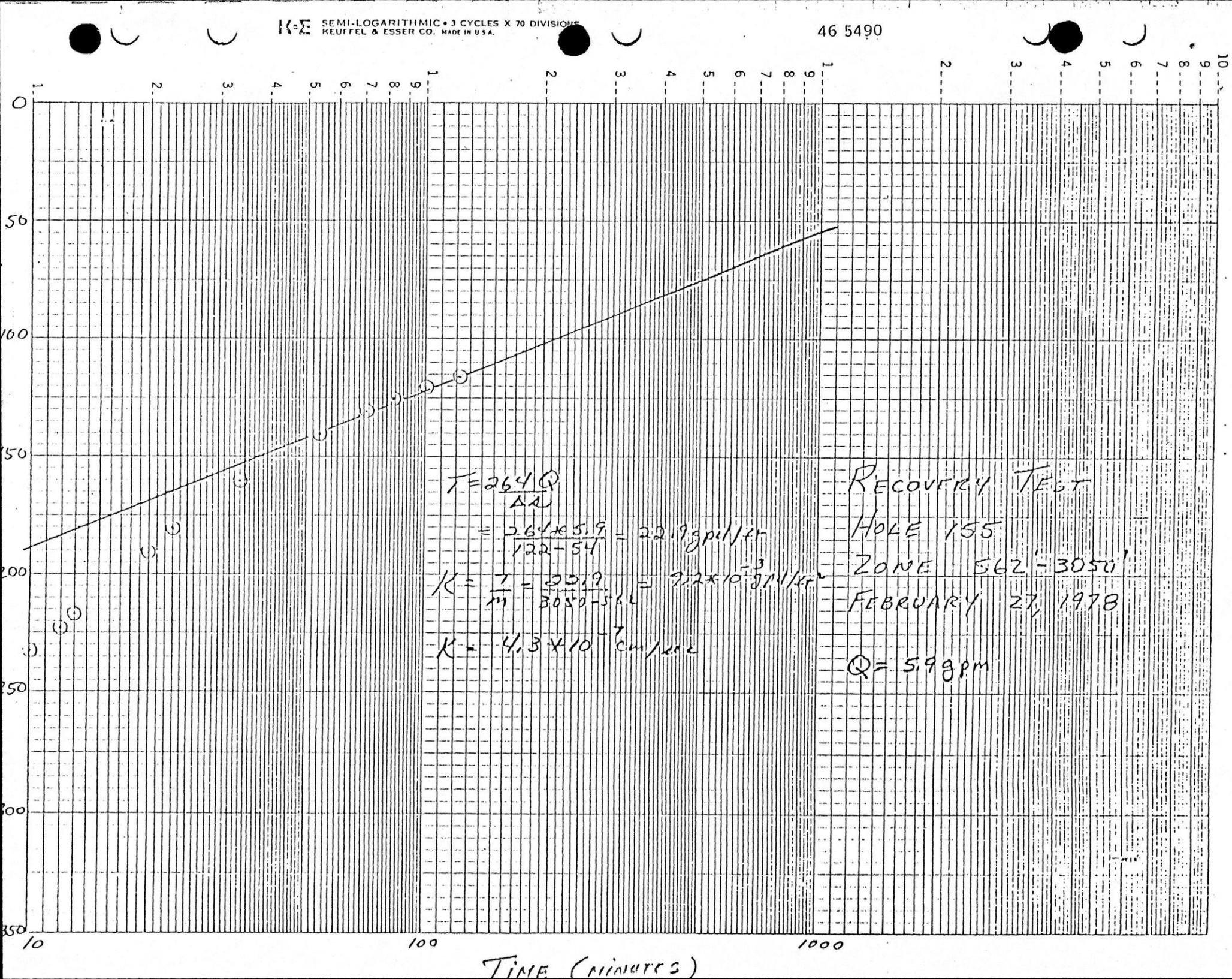
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$T = \frac{264Q}{\Delta s}$

$= \frac{264 * 5.9}{132 - 54} = 22.9$ gpd/ft

$K = \frac{T}{m} = \frac{22.9}{3050 - 564} = 9.2 * 10^{-3}$ gpd/ft²

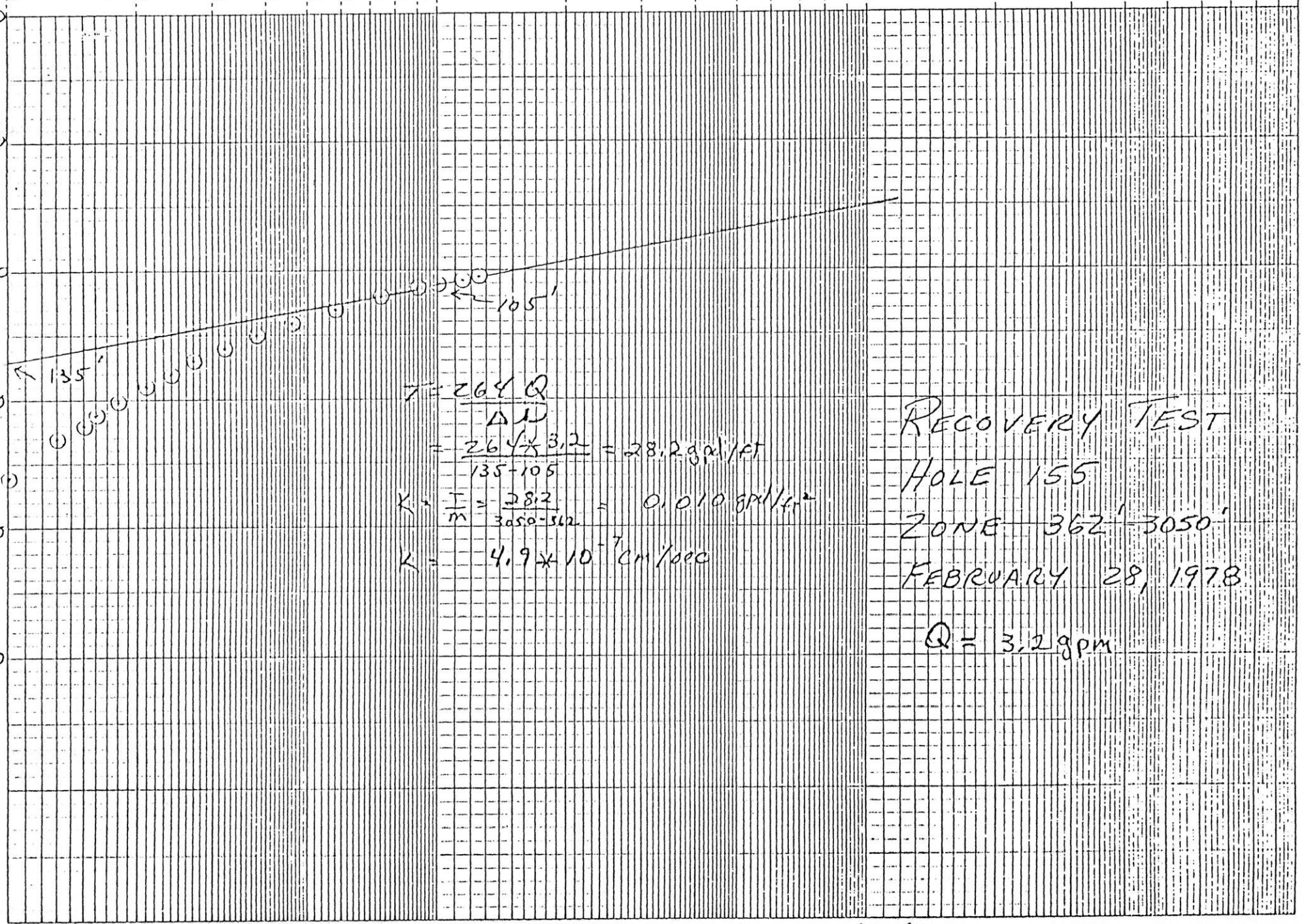
$K = 4.3 * 10^{-7}$ cm/sec

RECOVERY TEST
HOLE 155
ZONE 562 - 3050'
FEBRUARY 27, 1978

Q = 5.9 gpm

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$$T = \frac{264\,Q}{\Delta h}$$

$$= \frac{264 \times 3.2}{135 - 105} = 28.2 \text{ gal/ft}$$

$$K = \frac{T}{m} = \frac{28.2}{3050 - 362} = 0.010 \text{ gpd/ft}^2$$

$$K = 4.9 \times 10^{-3} \text{ cm/sec}$$

RECOVERY TEST

HOLE 155

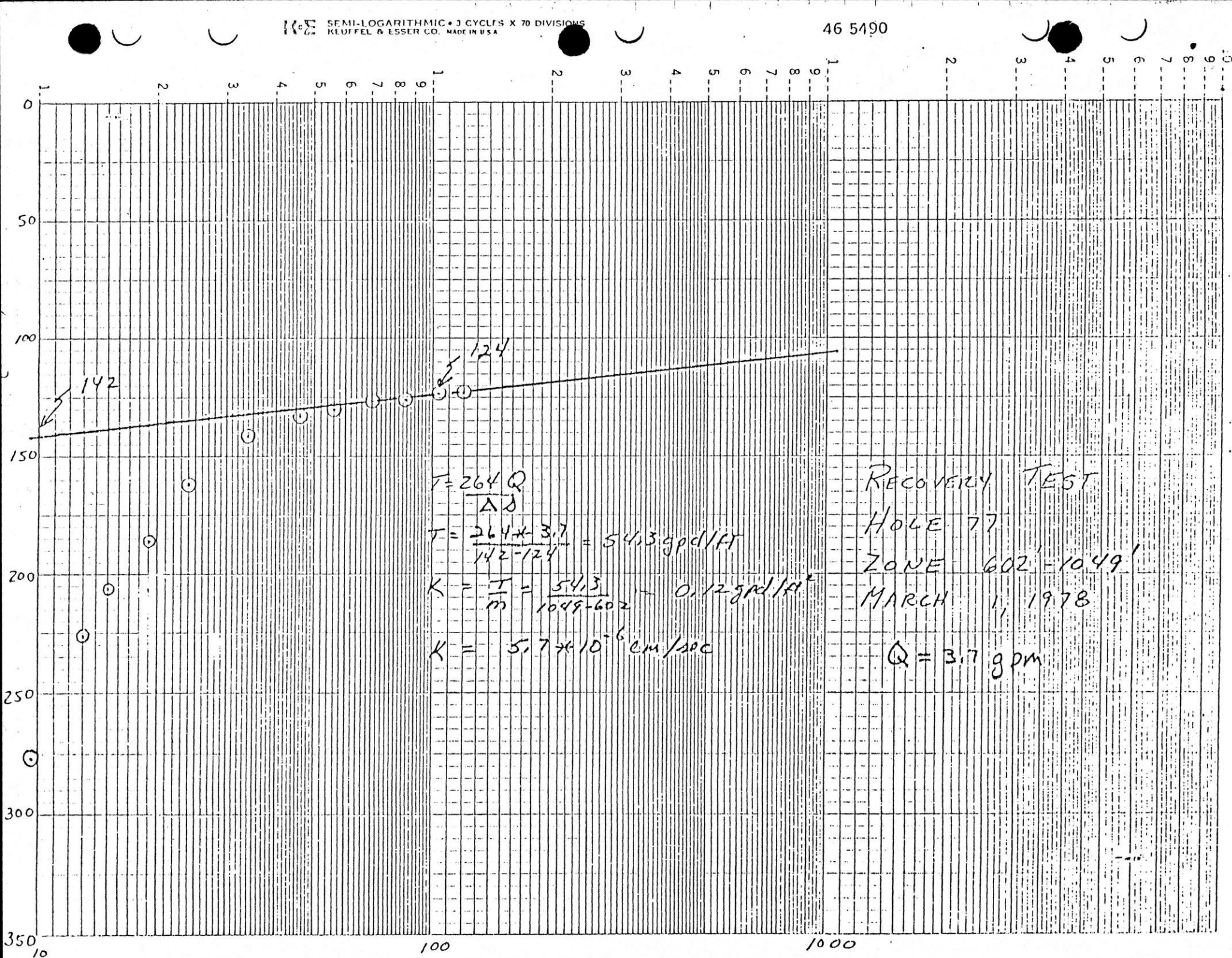
ZONE 362'-3050'

FEBRUARY 28, 1978

Q = 3.2 gpm

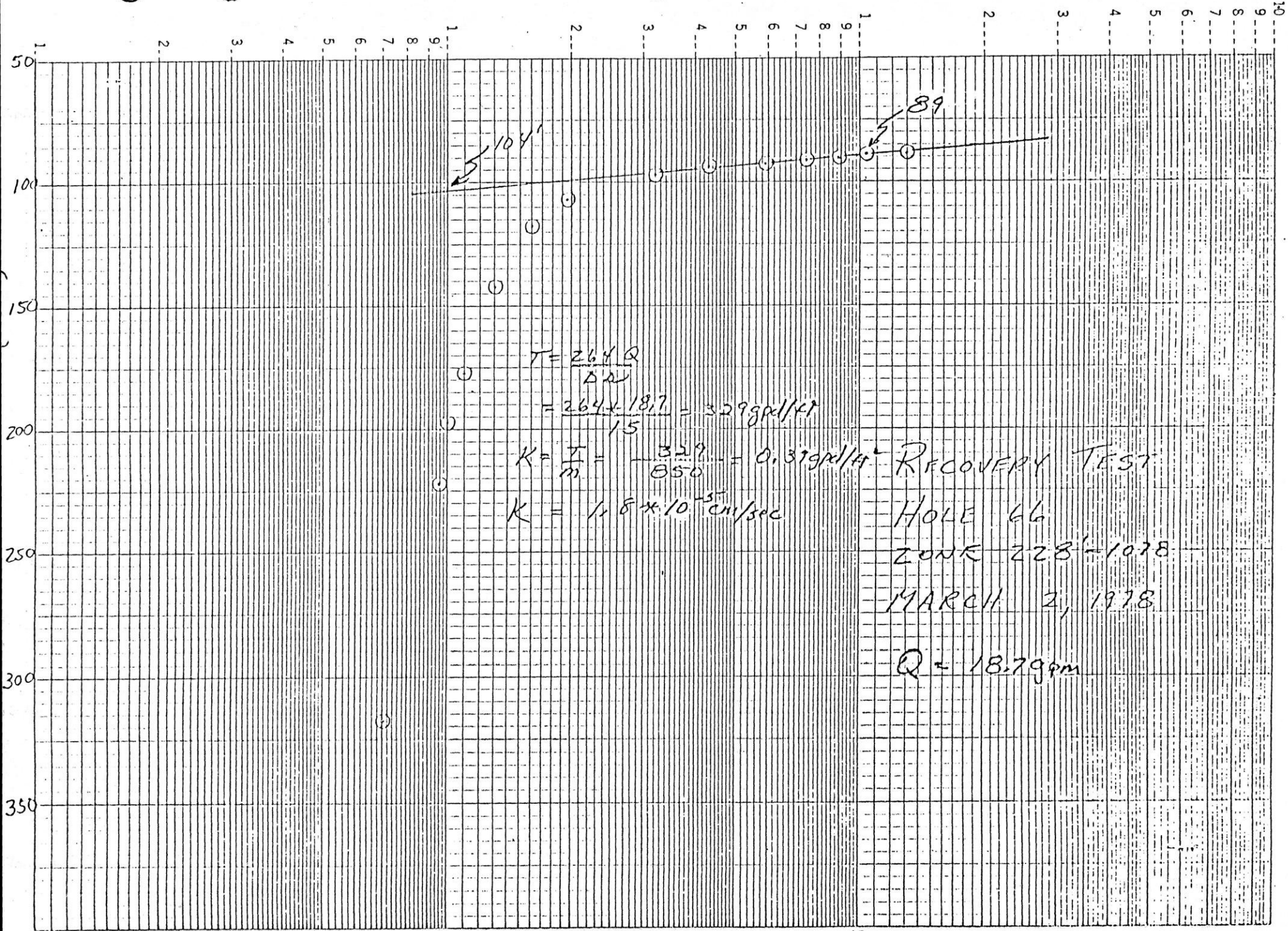
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$$T = \frac{264\,Q}{\Delta s}$$

$$= \frac{264 \times 18.7}{15} = 329\ \text{gpd/ft}$$

$$K = \frac{T}{m} = \frac{329}{850} = 0.39\ \text{gpd/ft}^2$$

$$K = 1.6 \times 10^{-5}\ \text{cm/sec}$$

RECOVERY TEST

HOLE 66

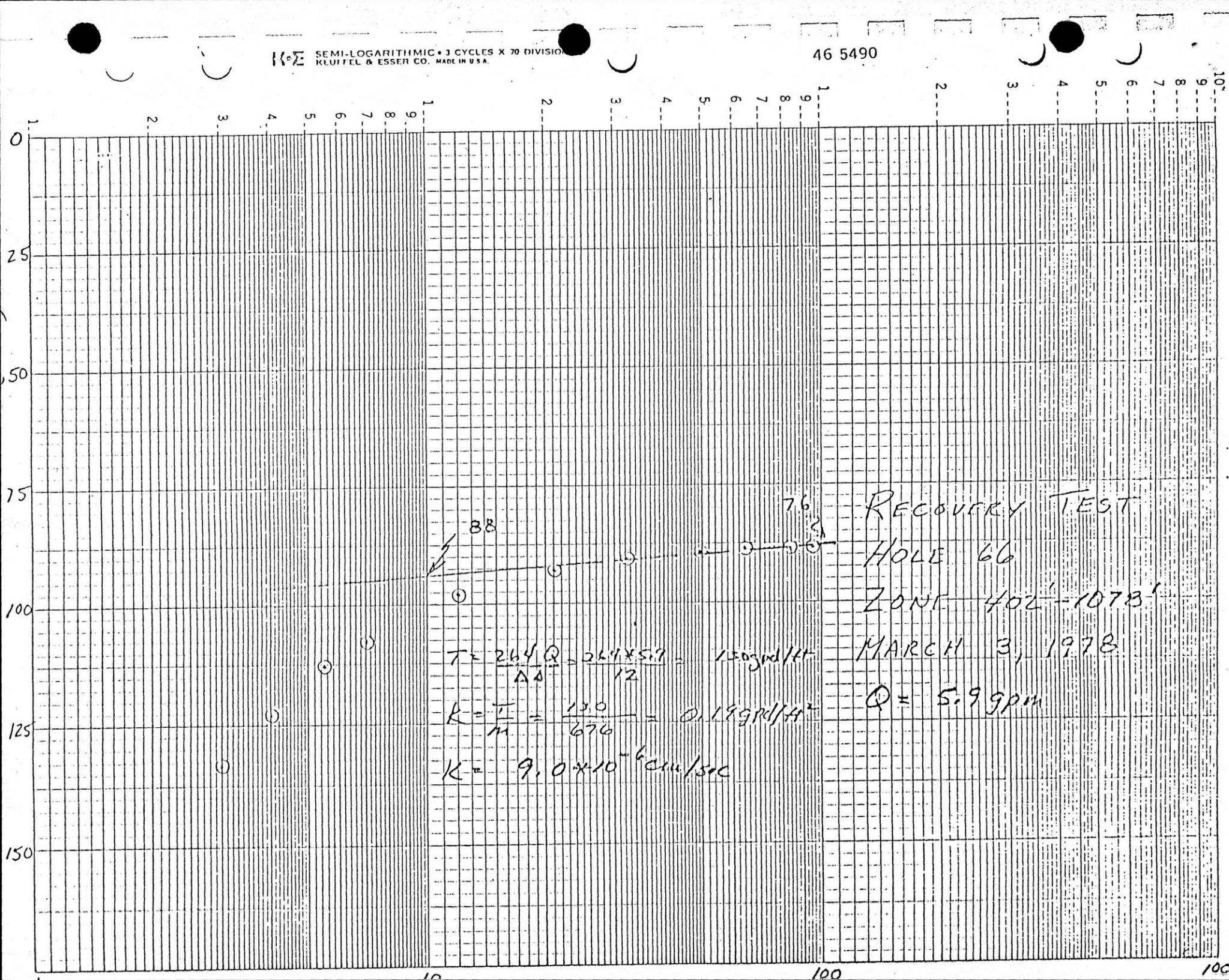
ZONE 228'-1078'

MARCH 21, 1978

Q = 18.7 gpm

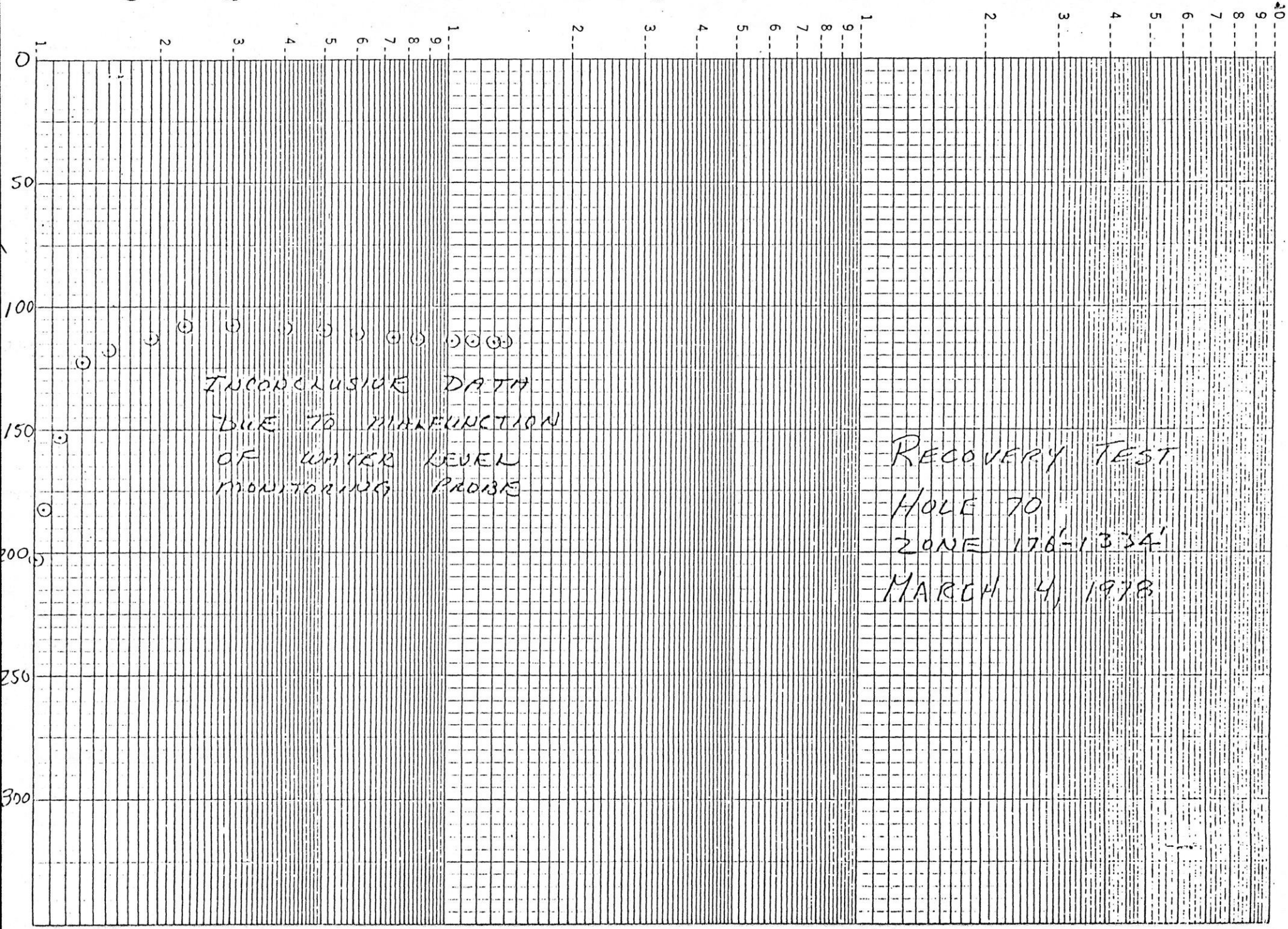
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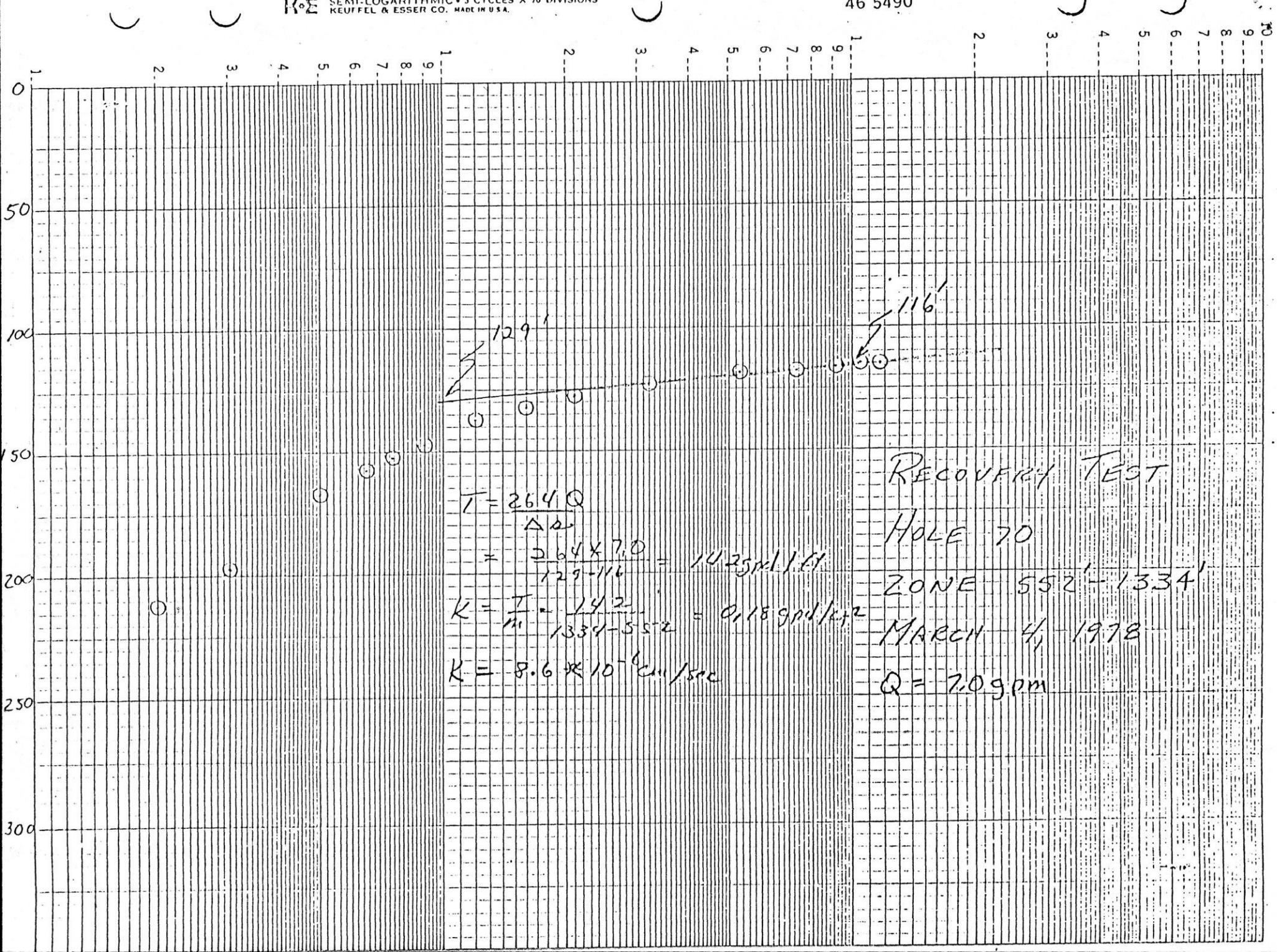


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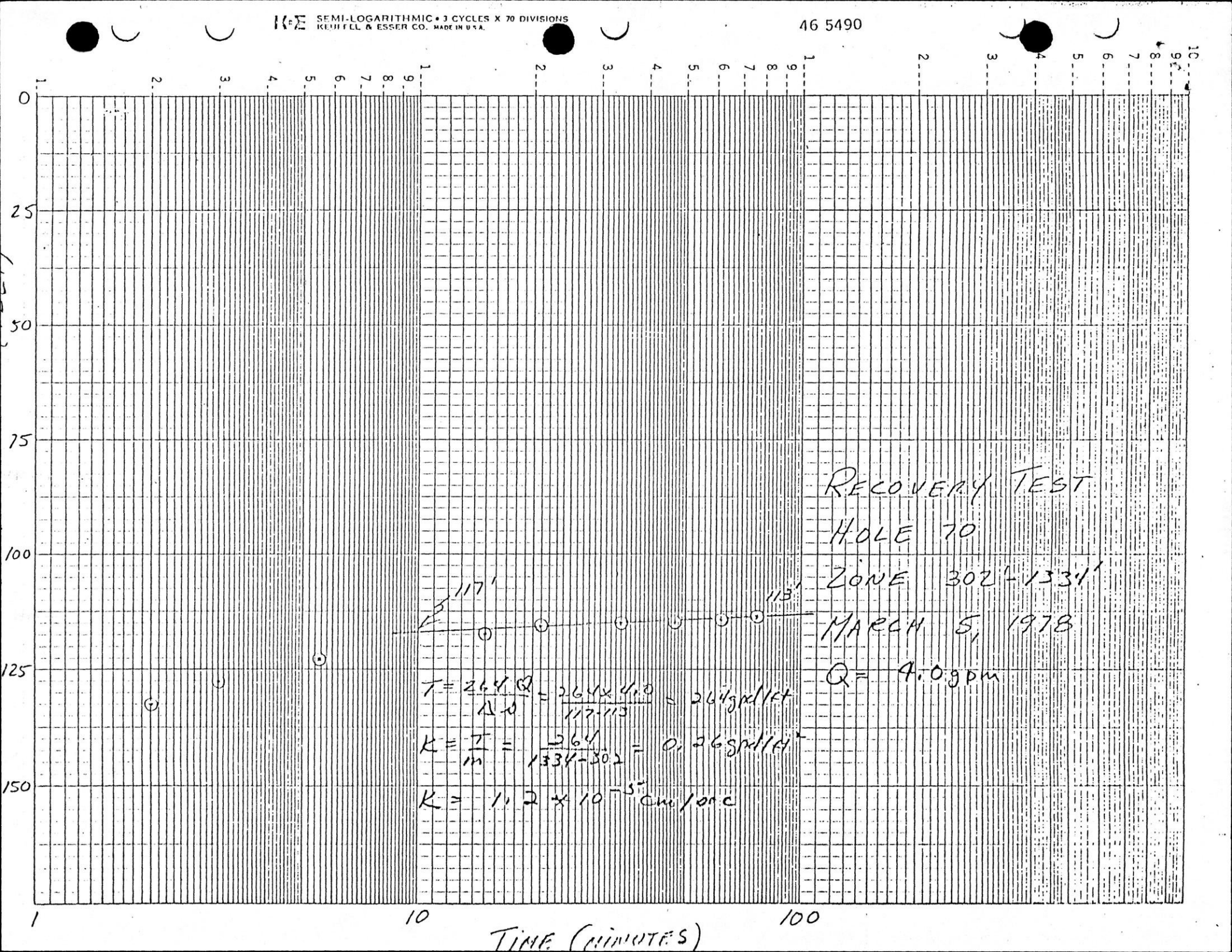


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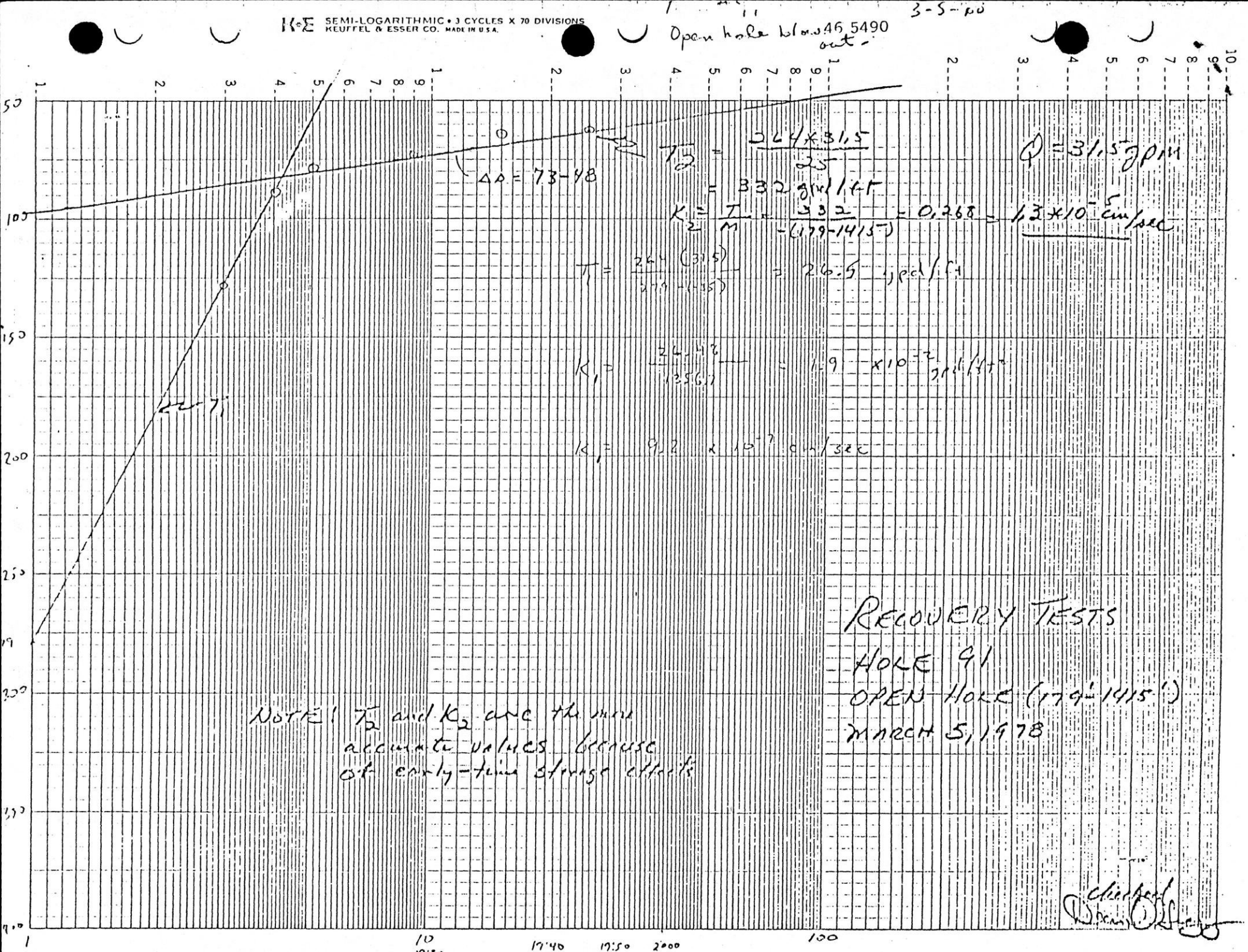
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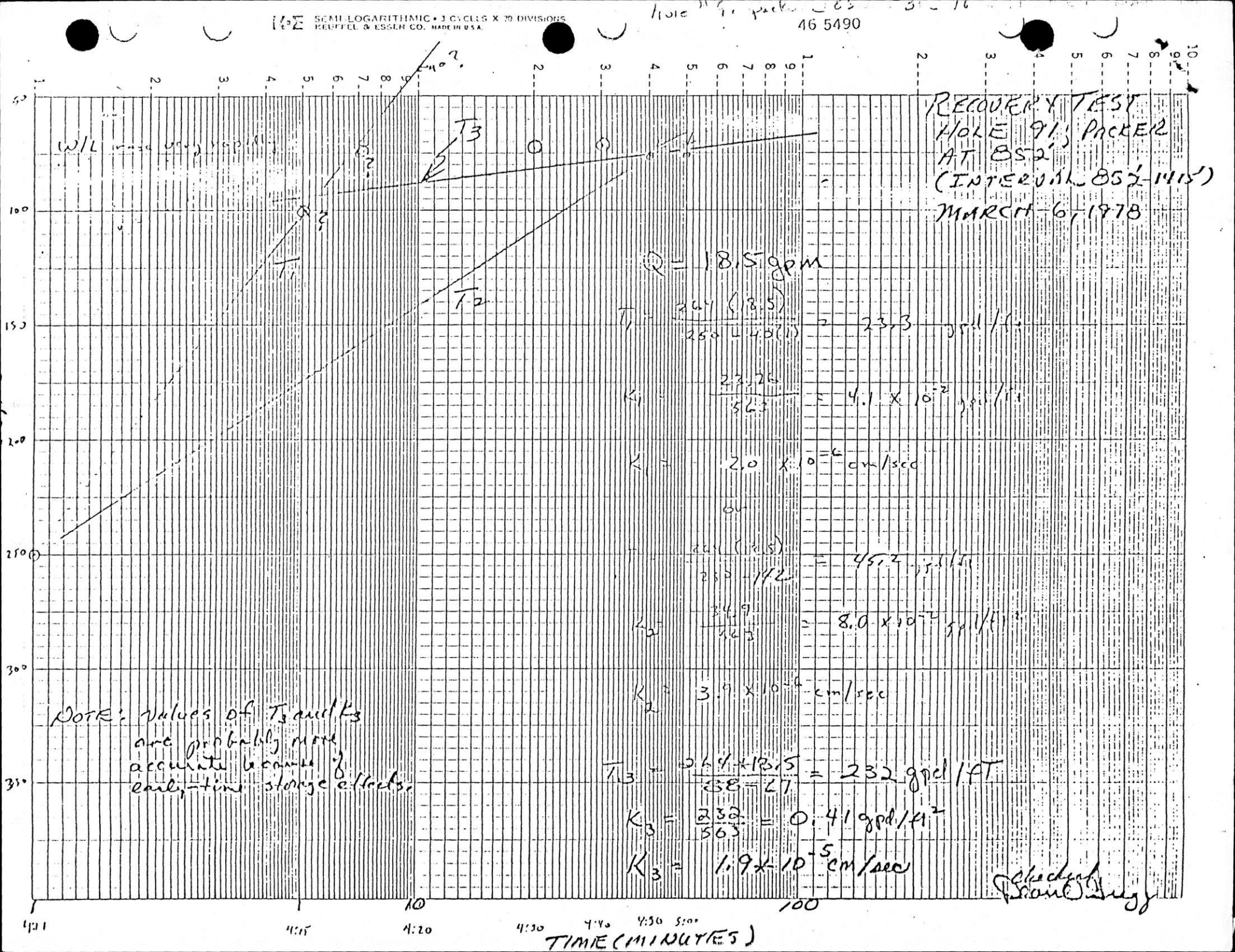
3-5-80

Open hole blow 46 5490 out -



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46 5490



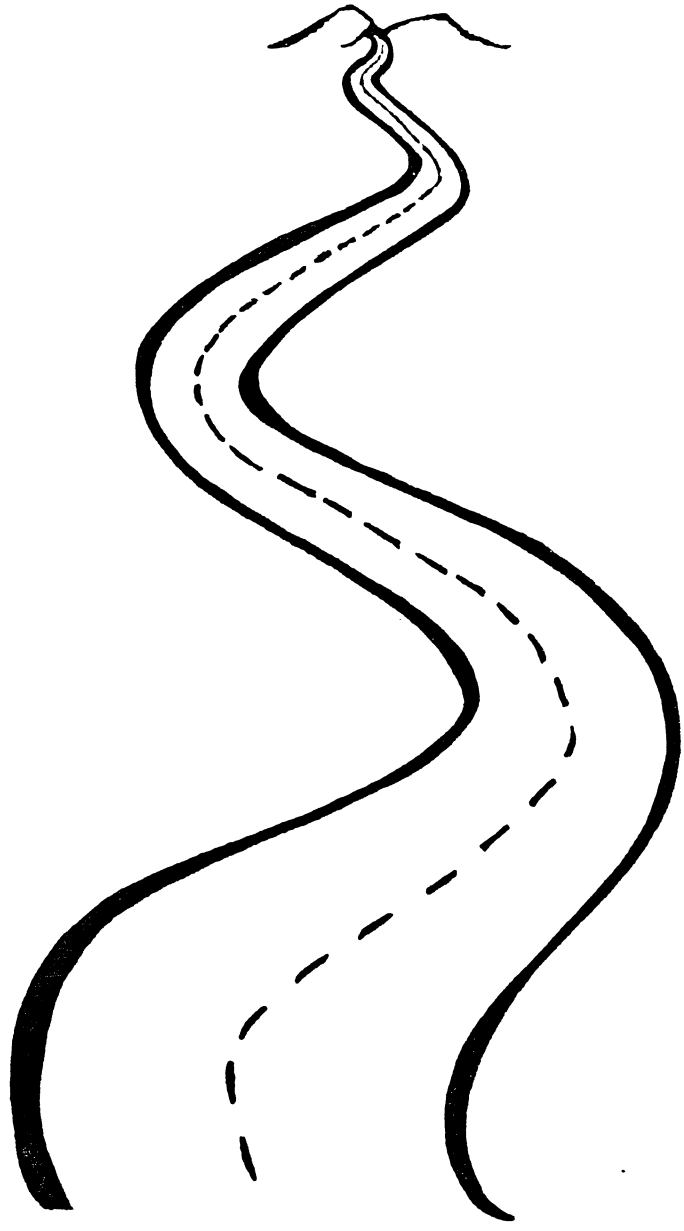
CRANDON PROJECT

MINE SITE ACCESS ROAD

ROUTE LOCATION STUDY

FOR

EXXON MINERALS COMPANY U.S.A.



INMAN, FOLTZ & ASSOCIATES, INC.
MINOCQUA, WISCONSIN

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CO., U. S. A.

MINE SITE ACCESS ROAD

ROUTE LOCATION STUDY
FOR
EXXON MINERALS COMPANY U.S.A.

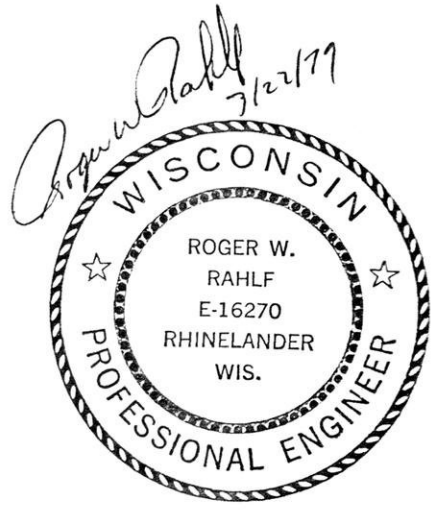
MARCH 22, 1979

PREPARED BY

inman-foltz and associates, inc.
minocqua, wisconsin 54548
715-356-9485

TABLE OF CONTENTS

Route Location Study	Pages
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INTRODUCTION

The purpose of the following study is to investigate various alternate access corridors to provide a direct, safe and economical roadway system to the proposed mine complex near Crandon, Wisconsin. The advantages and disadvantages of each of alternate routes have been set forth in this report along with the preliminary construction cost estimates.

In studying the various alternatives, several of the major items taken into consideration were:

1) Volume and types of vehicles to utilize the roadway.
2) Type of roadway required to handle the anticipated volume of traffic.
3) Various controlling horizontal and vertical design elements.
4) Areas of right of way required for construction.
5) Types of terrain which must be traversed.
6) Construction of major drainage structures.
7) Location of future electrical transmission lines to service the mine complex.
8) Total length of roadway to be constructed.
9) Possible reconstruction or upgrading of State Trunk Highway "55" within the existing right of way or the relocation of State Trunk Highway "55" around the village of Mole Lake.
10) Cost of Construction and design.
11) Existing road rights of way.
12) Property ownership.

Due to the problems of obtaining permission from various landowners along the proposed routes to conduct on the ground reconnaissance, this study has been based solely on aerial mapping of the area which has been furnished by Exxon Minerals Company, U.S.A. Portions of this mapping along the routes studied have been incorporated in the preliminary plans and profiles of the various routes which are a part of this study.

Since "on the ground" studies could not be conducted, many assumptions had to be made regarding various topographic features such as the exact size and depth of marshy areas, the suitability of other materials along the routes for roadbed material, depth and bottom material of Swamp Creek and the exact cross-section configuration at intervals

along each route. All assumptions that were made for the various alternates, we feel, are representative of the field conditions that can be expected to be encountered. In conjunction with the preparation of detailed engineering plans along the selected route, it will be required to obtain accurate topographic data and soils information from an "on the ground" survey.

The purpose of this report and associated plans is to provide the base information from which a decision can be made as to which alternate should receive additional detailed studies to provide the most economical and useful roadway system to provide access to the mine facilities.

DESIGN CONTROLS & CRITERIA

The design of a highway should be based upon factual data, among which are those relating to traffic. All data should be considered jointly. Financing, quality of soil conditions, availability of construction materials, cost of right of way and other factors have important bearings on the design, but traffic indicates the service for which the improvement is being made and directly affects the geometric features of design such as width, alignment, grades, right of way requirements and type of pavement.

The location and the design elements of a highway are influenced to a considerable degree by the topography, physical features and land use of the area traversed. Generally, topography is a major factor in determining the physical location of a rural highway. Steep hills, swamps or marshes, streams and lakes impose obvious restrictions upon the locations of the highway. Man-made features as well as the land use and ownership also have an effect on the locations of a highway. At this time a detailed and in depth study of the amount of traffic generated by activities at the mine site has not been conducted. It is beyond the scope of this study to determine the volume of traffic generated by the mining activities and from where the generated traffic will originate. It is anticipated that regardless of the point of origin of the traffic, the destination of a majority of the traffic will be the mining complex area and will utilize the alternate selected.

Traffic volumes used for this study have been based on the estimated number of workers that will be employed at the mine complex. The number of workers to be employed at the mine complex has been estimated by Exxon Minerals Company, U.S.A. to be approximately 900 persons during the construction phase and 800 persons during actual operation. In addition, a substantial amount of heavy truck traffic is anticipated to transport supplies and materials to the mine complex. The volume and type of truck traffic is not known at this time but this information will be developed as part of other ongoing impact studies of the area. In addition to the traffic which is destined to the mine complex, additional traffic may be generated due to residential development along the proposed route and traffic diverted from less desirable routes. Factors which may reduce the anticipated traffic volume will be the possible use of shuttle buses and car pools. Shuttle buses and car pools may become increasingly appealing as the cost of operating private vehicles increases.

Design Controls & Criteria

Based on the above information and the results of a meeting held on May 8, 1978 with Exxon personnel and representatives of the Wisconsin Department of Transportation, it was decided to estimate the average daily traffic at 3000 to 4000 passenger car equivalents per day. All the geometric design criteria for this study have been based on this assumption. It is suggested that when more detailed traffic data is available, the design criteria should be reviewed in order to determine if the highway will have the capacity to handle the proposed traffic, or if a lower standard design should be utilized because of lesser anticipated traffic. For the present, it appears that the assumed traffic volumes are relatively accurate for the purpose of this study.

ELEMENTS OF DESIGN AND CROSS-SECTIONS

All elements of design should be coordinated to maximize the service and minimize hazards at a reasonable cost. Various design elements are directly related to the volume of traffic, the desired operating speed and the type of terrain.

Based on the assumed traffic volumes set forth in the previous section and the rural nature of the area, it has been determined that the geometric design elements of a Class 5 highway, with a design speed of 60 miles per hour, as outlined by the Wisconsin Department of Transportation, should be used as a design guide for the alternate routes. Although all the design elements have not been discussed in this report, the major design elements such as:

1. Stopping sight distance (475)
2. Passing sight distance (2100)
3. Degree of horizontal curve (max. 5°00')
4. Vertical alignment (max. grade 4%)
5. Drainage
6. Pavement width (24 feet)
7. Shoulder width (10 feet)
8. Structure loading (HS 20)
9. Structure clear width (44 feet)

have been considered when choosing the location of the various alternatives and the development of the construction cost estimate. In addition to the above design elements, a suggested typical cross-section is shown on Sheet 8 of the accompanying plans. This typical cross-section depicts the ditch configuration and side slopes for various depths of cuts and fills.

Based on the typical cross-section, existing topographic data and the proposed centerline profile, an estimated cost of the major construction items were calculated. The breakdown of the estimate of quantities and costs of each route are included in the appendix this report.

The inability to gain access to several areas along the alternate routes, to conduct a soils investigation, has prevented this study from detailing the type pavement and aggregate base course which will be required for the various alternatives. In order to compare the alternate routes from an economic standpoint, an aggregate base course depth of 9 inches and a bituminous pavement thickness of 3 inches was chosen. Unless very adverse soil conditions are encountered along the routes, the assumed pavement design should prove adequate. However, after a route is selected, soil samples should be obtained along the route and a final pavement should be designed based on the result of the soils tests.

CONSTRUCTION ITEMS

Each of the three alternates presented in this study has been compared using the same design criteria and construction unit costs. It is virtually impossible at this point to determine the exact quantity and associated cost of all construction items. However, this study has attempted to include the major construction items as well as their estimated costs. Construction cost estimates for the various items have been partially based on bids received by the Wisconsin Department of Transportation, District #7, along with anticipated inflation rates. It is felt that this method of determining construction costs is the most accurate at this time and sufficient to compare the relative costs of each alternate. After the final engineering plans have been completed, a more detailed cost estimate can be prepared.

Right of way costs have not been included in any of the alternates since land costs in the vicinity of the proposed mine complex seem to vary a great deal. For some of the alternates, the lands needed for right of way may not be available for purchase. Once a route has been selected, land negotiations could be started by Exxon personnel to determine the costs and problem associated with obtaining the required right of way.

The proposed centerline location and the proposed profile were based on the design criteria set forth previously. The terminus of each route was selected to allow for future extension. Alternates "A and B" could be extended to the north and south to connect to State Trunk Highway "55". Alternate "C" has its northern terminous at State Trunk Highway "55" and could be extended to the south to connect to State Trunk Highway "55". The routes were also located to pass relatively close to the proposed mine complex. It was thought that it be in the best interest of safety and security to keep each of the routes approximately 1000 feet from the actual mine complex entrance. A 1000 foot mine complex entrance road would allow for adequate vehicle "stacking" for vehicles entering the complex without disrupting traffic along the selected route.

The lands required for the construction of each alternate (right of way) havebeen determined based on the depth of cut or fill along the proposed routes and also the requirement of the typical cross-section as shown on sheet 8 of the accompanying plans. Based on the above determining factors, a right of way width of 200 feet will be required. It should be pointed out that a right of way width of a narrower dimension (120 feet to 150 feet) could be utilized if there is not the possibility of the route being used as a state trunk highway in the future. For the purpose of this study a right of way width of 200 feet was used. The amount of clearing to be done to

allow construction and the area to receive seed and fertilizer has been based on this assumed right of way width of 200 feet.

The removal and reconstruction of fences, as the various routes pass through fields used for agricultural purposes, will be required. Normally, any area fenced, regardless of the condition of the fence, will require the construction of a new fence. The amount of fence required for removal and reconstruction was estimated based on the length of fields the proposed routes were to cross as determined from aerial mapping supplied by Exxon.

All of the proposed alternates must, at various locations, pass through wetland areas. The approximate size of these areas has determined based on information supplied by Exxon and prepared by Dames and Moore as part of their environmental studies. Although the approximate area of the wetland areas was delineated, the depths of the wetlands are unknown. In order to determine the volume of marshy material to be excavated from these wetlands, all wetlands were assumed to have a depth of eight feet.

As with any highway project of this nature, considerable earth must be located from one location to another. An attempt has been made to minimize the amount of earth which will have to be moved for each of the routes and yet meet the design criteria. The volume of earthwork has been calculated based on the existing centerline profile, the proposed centerline profile, the configuration of the typical cross-section, and the length of each route. A preliminary attempt has also been made to balance the amount of excavated material with the fill material required and to minimize the length of haul. Since it was not possible to obtain accurate existing cross-section at 100 foot intervals, it was assumed that each section had a level existing cross-section. After a route has been selected, the earth work computation will have to be revised to reflect the actual existing cross-sectional configuration.

The estimated quantity of gravel base course and bituminous pavement surface has been based on the preliminary pavement design of 9 inches of gravel, 44 feet wide and a bituminous pavement thickness of 3 inches, 24 feet wide. This design allows for 2 paved traffic lanes, 12 feet wide, plus a 10 foot gravel shoulder on each side. The 10 foot gravel shoulder will allow disabled vehicles to park completely off the travelled roadway and thus reducing a potential hazard.

Construction Items

Minor drainage structures (culverts) will be required at all natural drainage-ways, road intersections and driveways. An attempt has been made to identify, based on the aerial mapping, the possible locations of needed culverts. Culverts of various diameters will be required depending upon the anticipated volume of water required to be passed.

Since each of the three alternates presented will have to cross Swamp Creek, a rather substantial structure will have to be built. This structure could be constructed of reinforced concrete (cast in place or precast) or of corrugated metal. For the purpose of this study, the construction costs of a concrete structure were used for the cost comparison of each route. If a concrete structure were to be built, it would be of a similar design to that recently constructed by the Wisconsin Department of Transportation at the crossing of State Trunk Highway "55" and Swamp Creek. In order to construct a structure over navigable water (Swamp Creek is considered navigable), the approval of several governmental agencies is required. The approving authorities include: The U.S. Army Corps of Engineers, U.S. Fish & Wildlife, Wisconsin Department of Natural Resources, and possible County and Township approval. In all probability, an environment impact report will have to be submitted.

After one of the alternate routes has been selected, a detailed field survey of the selected route will have to be conducted as part of the design phase. The field survey of the selected route will consist of obtaining detailed information on:

1. Elevations along the proposed centerline @ 100 foot intervals.
2. Cross-sections of the existing ground @ 100 foot intervals.
3. Depth of swamps.
4. The accurate location of all topographic features (rock outcrops, drainage ways, fields, wooded areas).
5. The location of man-made features along the route (driveways, houses, barns, sheds, fences, wells, etc.).
6. Representative soil samples at various locations along the route.
7. Channel characteristics of Swamp Creek.

Construction Items

The information obtained from the field survey will be utilized during the preparation of the final construction plans. It is important that the field survey be accurate in order to reduce the number of questions and problems arising during construction.

Final detailed construction plans will have to be prepared in order to furnish a contractor with the necessary information to construct the alternate selected.

In order to insure the proper construction of the highway, it will be required that the location of various construction items be field located in order to assist the contractor. In addition, a resident inspector should supervise the construction in order to insure that all construction items meet the design specifications and answer questions that may arise during construction.

The cost associated with the field survey, preparation of construction plans, field staking and inspection/supervision has been expressed as percentage of total cost of the construction items.

Although the maintenance of a highway can not be considered as a construction item, it is, nevertheless, an item which must be budgeted for. Maintenance items include; the grading of the gravel shoulder, patching the bituminous surface, mowing grass along the roadside, brush control, replacing signs as well as snow removal and sanding and salting operations in the winter. Maintenance costs, exclusive of snow removal and sanding and salting operations in the winter, average approximately $1000 per mile per year over a 15 year period. The first few years after the construction, the maintenance costs will be less than average but near the end of the 15 year period the maintenance costs will exceed the average. Snow removal and sanding and salting operations in the winter are estimated to cost approximately $1000 per mile per year. This cost will vary depending on the severity of the winter weather.

DISCUSSION OF ALTERNATES

General

It was the initial intent of this report to investigate four alternate means of gaining access to the mine complex. Routes "A" and "B" were to start near the Crandon Municipal Airfield in Section 12, T35N, R12E; Route "C" to start near the center of Section 23, T35N, R12E; Route "D" to start near the point where State Trunk Highway "55" crosses Swamp Creek. Each of the routes were to terminate near the southeast corner of Section 25, T35N, R12E.

After an analysis of the property ownership along Route "D", it was decided not to pursue this alternate since the route would have to pass through parts of the Mole Lake Indian Reservation. This fact may have made it difficult to obtain the necessary right of way, and would have increased the traffic and congestion on State Trunk Highway "55" in the Mole Lake area and created a potentially hazardous intersection. No plan, profile or construction cost estimates were therefore prepared for alternate "D".

Route "A" (length 2.7 miles)

This route proceeds in a generally southerly direction from the east one-quarter corner of Section 13, T35N, R12E, to a point near the southeast corner of Section 25, T35N, R12E and passes approximately 900 feet west of the entrance to the proposed mine complex. Route "A" will cross Swamp Creek approximately 100 feet east of the township line between the Townships of Nashville and Lincoln. A detailed location of the route is shown on Sheets 2 and 3 of the associated plans.

The following items are considered to be advantages to selecting Route "A":

1. Most direct route from north - This route provides the best access from the north. (Crandon - 6.5 miles, U.S.H. "8" 6 miles)

2. Follows anticipated route for electrical transmission line - Information from Wisconsin Public Service Corporation indicate that this route is being considered for the location of transmission lines to service mine complex.

3. Provides a good alternate for future bypass of Mole Lake area By extending this alternate to the south and connecting with

State Trunk Highway "55", access to the mine complex from the South would be improved.

4. Follows existing town road for approximately one-half mile - thereby reducing the amount of right of way to be purchased.

The following items are considered to be disadvantages to selecting Route "A".

1. Major access point to this route is a Town Road - The nearest state or county highway is approximately 1-1/2 miles west or 2 miles north of the north end of this route.

2. Additional 2 miles of roadway must be constructed - In order to provide adequate access to State Trunk Highway "55". This would involve the upgrading of the existing town road (Airport Road) including the purchase of additional right of way.

3. Possible problems in obtaining required right of way - Based on information presented by Exxon at least one of the property owners along this route would object to selling his property.

4. Does not provide improved access from the south - Vehicles traveling from the south on State Trunk Highway "55" would still have to use Sand Lake Road for access to the mine complex.

ROUTE "B" (Length 3.5 miles)

Route "B" origin is near the southeast corner of Section 12, T35N, R12E, from there heading generally southeast to the center of Section 18, T35N, R13E. From this point the route would turn south, crossing Swamp Creek near its intersection with the North-South quarter line of Section 19, T35N, R13E. The route would then turn southwesterly toward the southeast corner of Section 25, T35N, R12E. This route also passes approximately 900 feet west of the entrance to the proposed mine complex. A detailed location of the proposed route is shown on Sheets 4 and 5 of the plans.

The following items are considered as advantages to selecting this route:

1. Less marsh excavation - This route crosses less wetland than Routes "A" and "C".

Discussion of Alternates

2. Utilizing more existing town road rights of way - There may be less opposition to upgrading and widening an existing road than the construction of a completely new road.

3. Possibly less problem acquiring the required right of way - This route bypasses a known problem in the acquistion of right of way.

The following items are considered disadvantages to selecting Route "B":

1. Longest route - Causing increased construction cost and additional time for travel.

2. Additional 1.5 miles of roadway must be constructed - In order to provide adequate access to State Trunk Highway "55". This would involve the upgrading of the existing town road (Airport Road) including additional right of way.

3. More residences near route - This may cause an adverse reaction from adjoining owners due to increased traffic.

4. More property owners to purchase right of way from - Each additional purchase creates a potential problem.

5. Does not provide improved access from the south - Vehicles traveling from the south on State Trunk Highway "55" will have to use Sand Lake Road for access to the mine complex.

ROUTE "C" (Length 2.5 miles)

Route "C" has its origin at the intersection of State Trunk Highway "55" and Sherman Lane near the center of Section 23, T35N, R12E, thence running east and southeast to the southeast corner of Section 25, T35N, R12E, crossing Swamp Creek near the north-south quarter line of Section 24, T35N, R12E. This route passes approximately 1800 feet west of the entrance to the proposed mine complex. A detailed location of the route is shown on Sheets 6 and 7 of the plans.

Discussion of Alternates

The following items are considered as advantages to selecting this route.

1. Direct access to Highway "55" - Additional roadway will not have to be constructed when alternate "C" is completed.

2. Shortest alternate proposed - This route is 1 mile shorter than Route "B" and 0.2 mile shorter than Route "A".

3. Minimum number of property owners to obtain right of way from right of way purchase should be simplified since fewer property owners are involved.

4. Provides improved access from the south - Traffic from the south on State Trunk Highway "55" will be able to utilize this alternate with only a slight increase in distance as compared to the present Sand Lake Road access.

The following items are considered as disadvantages in selecting this route:

1. Longer distance to travel - This alternate provides the longest route from Crandon and U.S. Highway "8" as compared to the other alternates.

2. Does not substantially reduce the distance traveled on State Trunk Highway "55" - This alternate does not reduce vehicle travel time or improve the safety of State Trunk Highway "55"

3. More wetlands to cross - This alternate requires the most wetland excavation.

4. Steep grade south of Swamp Creek - Substantial earth work will be required to reduce this grade to meet the design criteria.

SUMMARY OF FINDINGS AND CONCLUSIONS

The data presented in this report should be utilized as the basic information from which a decision can be made as to which, if any, of the proposed alternates should be constructed to provide access to the mine facilities.

Each of the alternates investigated was studied in view of its engineering feasibility, directness of route, future construction, availability of right of way, and the cost of construction.

During the course of this study, several meetings were held with representatives from the Wisconsin Department of Transportation and personnel from Inman-Foltz and Associates, Inc., in order to determine what, if any, future plans the state has for improving or upgrading State Trunk Highway "55" between Crandon and Mole Lake. The results of those meetings indicated that the state has no plans to upgrade or improve State Trunk Highway "55" in the foreseeable future, other than normal maintenance. Department of Transportation personnel indicated that there exists the possibility that State Trunk Highway "55" could be rerouted along the existing location of County Trunk Highway "S" between State Trunk Highway "55" and U.S. Highway "8". This could become a reality if there is a substantial increase in the traffic utilizing County Trunk Highway "S" as a "short cut" from U.S. Highway "8" to State Trunk Highway "55", and if Forest County would be willing to take over the maintenance of existing State Trunk Highway "55" from the intersection of State Trunk Highway "55" and County Trunk Highway "S" to the City of Crandon. If State Trunk Highway "55" were rerouted along County Trunk Highway "S", access to the mine site for vehicles coming from the west on U.S. Highway "8" would be reduced by approximately four miles, while only increasing the distance which the vehicles from the east would have to travel by one mile. Vehicles from the east would either have to travel through the City of Crandon or follow the existing route of State Trunk Highway "55" in order to reach the mine facilities. The rerouting of State Trunk Highway "55" along the existing right of way of County Trunk Highway "S" would be most advantageous if route C were selected as a primary access to the mine site.

Although routes A and B would provide the most direct access to the mine complex from the north, additional construction would be required in order to gain direct access to a State Trunk Highway.

Summary of Findings and Conclusions

This additional construction would increase the cost of routes A and B by approximately $300,000. Also, routes A and B do not improve the access to the mine complex from the south. Route C has the advantage of direct access to STH "55" along with better access from the south than either routes A or B. Since route C has direct access to STH "55", no additional construction will be required except for the construction of the mine entrance road.

It should be pointed out, that although a proposed centerline location is shown for each of the routes, these are intended to show only a general location of the centerline. The proposed centerlines of each route can be changed to avoid problems which may arise in the acquisition of rights of way, environmental objections by various agencies, unforeseen subsurface conditions, and intersection problems with STH "55". The proposed centerlines were shown on the plans to indicate the location of possible corridors and terminus points for each of the routes.

Based on the engineering feasibility, costs of construction, access to STH "55", improved access from the south, the number of property owners from which rights of way will have to be purchased, and reduced maintenance costs, it appears that route C would be a better selection than either routes A or B. However, consideration should also be given to driver comfort and safety on STH "55" and CTH "S" and the monies budgeted for the project.

Once the final decision has been made as to which general location is to be utilized for access to the mine facilities, work should begin immediately on submitting the selected corridor to the Wisconsin Department of Natural Resources and the Wisconsin Department of Transportation for their review, suggestions, and recommendations. The abovementioned agencies should be given a wide corridor to study rather than a definite route, in order to allow flexibility in the final location of the route and allow for the agency input as to their suggested location for the route.

The Department of Natural Resources' main concern will be the relationship, and the effects the proposed highway construction will have on the wetlands, Swamp Creek and other environmentally sensitive areas. The Department of Transportation's review will be concerned with the location and potential impact of the intersection of the selected route with STH "55". The expected traffic and turning movements will indicate if additional turning lanes or channelization will be required. Early contact and involvement of the above agencies will help reduce possible future conflicts as the final engineering plans are being developed.

Contact should also be made with the Forest County Highway Commission, in order to determine if the county would be willing to include the selected route into the County Trunk system. If the county were to accept the newly constructed roadway as part of its trunk system, there would be advantages to both Forest County and Exxon. The advantage to Forest County would be that they could collect gas tax money for the maintenance of the road without the initial expenditures for construction. Exxon would benefit because the yearly expenditures for maintenance would be eliminated since they would be picked up by the county.

If Forest County would be willing to accept the road selected into its roadway system, the route will have to be designed and constructed to meet or exceed the minimum standards for a Forest County road. Forest County has not established its own design criteria for the type of road required, but it is anticipated that the design criteria for a Class 5 highway will meet or exceed the county's requirements.

In order to complete all the necessary design, prepare the required submissions for approvals, obtain competitive bids from contractors, and construct the route, a minimum time period of 18 months will be required. A more realistic and workable time table for completion would be 24 months. If time is a major factor in this project, it is recommended that a decision be made as to which route is to be selected and the final design should be commenced as soon as practical.

APPENDIX

Route "A" - Construction Cost Estimate

1) Right of Way (Acres)

a)	Crandon Municipal Airfield	1.5
b)	Hillstead, Robert	1.2
c)	Craycroft, Ronald	8.5
d)	Moe, S. N.	7.6
e)	Paul, Lynn	6.2
f)	Hess, John	12.2
g)	Mihalko Land and Logging Company	0.5
h)	Exxon	24.3
	Total	62.0 (45)*

2) Clearing (Acres)

58.1 acres @ $600/acre = $34,860 ($25,450)

3) Removing Fence (ln. ft.)

7000 ln. ft. @ $1.00/ln. ft. = $7000 ($7000)

4) Marsh Excavation (cu.yd.)

100600 cu.yd. @ $0.80/cu.yd. = $80,480 ($70,820)

5) Unclassified Excavation (cu.yd.)

144600 cu.yd. @ $1.15/cu.yd. = $166,290 ($124,700)

6) Crushed Aggregate Base Course (cu.yd.)
(Assume 9" thick, 44' wide)

17350 cu.yd. @ $5.70/cu.yd. = $98,900 ($98,900)

7) Bituminous Base Course (Ton)
(Includes bituminous material, assume 1-3/4" thick, 24' wide)

3480 tons @ $15.00/ton = $52,200 ($52,200)

8) Bituminous Concrete Pavement (Ton)
(Includes bituminous material, assume 1-1/4" thick, 24' wide)

2485 tons @ $15.00/ton = $37,280 ($37,280)

*Figures in parenthesis are estimates based on the typical cross-section shown on Sheet No. 8 of the plans with a modification as to the width of the safety section and back slopes.

Route "A"

9) Corrugated Metal Culvert Pipe (ln. ft.)

a) 18" - 384 ln. ft. @ $14.60/ln. ft. = $ 5,610
b) 24" - 192 ln. ft. @ $20.00/ln. ft. = $ 3,840
c) 36" - 64 ln. ft. @ $31.00/ln. ft. = $ 1,980

Total = $11,430 ($11,430)

10) Metal Apron Endwalls for Culvert Pipes (each)

a) 18" - 12 @ $65.00/each = $ 780
b) 24" - 6 @ $108.00/each = 650
c) 36" - 2 @ $204.00/each = 410

Total $1,840 ($1,840)

11) Woven Wire Fencing Construction (ln. ft.)

7000 ln. ft. @ 1.85/ln. ft. = $12,950 ($12,950)

12) Seeding (2 lbs./100 sq. ft.)

2130000 sq. ft. x 2 lbs./1000 sq. ft. x $3.50/lbs. = $14,910 ($12,950)

13) Fertilizer (10 lbs./1000 sq. ft.)

2130000 sq. ft. x 10#/1000 sq. ft. x $20/100 lbs. = $4,260 ($2,560)

14) Steel Plate Beam Guard (ln. ft.)

1400 ln. ft. @ $10.00/ln. ft. = $14,000 ($14,000)

15) Anchorages for Steel Plate Beam Guard (Each)

4 each @ $135/each = $540 ($540)

16) Riprap (cu. yd.)

250 cu.yd. @ $24/cu.yd. = $6,000 ($6,000)

17) Concrete Masonry Culvert (cu. yd.)

307 cu.yd. @ $160/cu.yd. = $49,120 ($49,120)

18) Granular Backfill (cu.yd.)

940 cu.yd. @ $2.87/cu.yd. = $2,700 ($2,700)

Route "A"

Total Major Construction Items	$594,760 ($517,490)
Construction Contingencies (10%)	$ 59,480 ($ 51,750)
Construction Total	$654,240 ($569,240)
Engineering Design (7%)	$ 45,800 ($ 39,850)
Construction Staking & Inspection (12%)	$ 78,510 ($ 68,310)
Total Alternate Cost	$778,550 ($677,400)
Length of Route 2.7 miles	
Cost Per Mile	$288,350/mile ($250,890/mile)

Route "B" - Construction Cost Estimate

1) Right of Way (Acres)

a)	Crandon Municipal Airfield	0.6	
b)	Wait, Douglas	6.5	
c)	Hillstead, Robert	7.0	
d)	Craycroft, Joseph	2.0	
e)	Zelanowski, Leo	2.0	
f)	Zebella, Emil	2.0	
g)	Lewis, Donald	2.0	
h)	O'Connor, Leonard	2.0	
i)	W.S.	0.5	
j)	Kennedy, Robert	1.5	
k)	Paller, William	0.5	
l)	Kennedy, Russell	3.5	
m)	Hess, John	6.2	
n)	Exxon	12.6	
	Total	48.9	(35.7)

2) Clearing (Acre)

41.8 acres @ $600/acre = $25,080 ($18,310)

3) Removing Fence (ln. ft.)

3100 ln. ft. @ $1.00/ln. ft. = $3,100 ($3,100)

4) Marsh Excavation (cu.yd.)

75,900 cu. yd. @ $0.80/cu.yd. = $60,720 ($53,430)

5) Unclassified Excavation (cu.yd.)

148000 cu.yd. @ $1.15/cu.yd. = $170,200 ($127,650)

6) Crushed Aggregate Base Course (cu.yd.)
(Assume 9" thick, 44' wide)

22,400 cu.yd. @ $5.70/cu.yd. = $127,680 ($127,680)

7) Bituminous Base Course (Ton)
(includes bituminous material, assume 1-3/4" thick, 24' wide)

4483 tons @ $15.00/ton = $67,250 ($67,250)

8) Bituminous Concrete Pavement (Ton)
(includes bituminous material, assume 1-1/4" thick, 24' wide)

3200 tons @ $15.00/ton = $48,000 ($48,000)

9) Corrugated Metal Culvert Pipe (ln. ft.)

a) 18" - 320 ln. ft. @ $14.60/ln.ft. = $ 4,670
b) 24" - 192 ln. ft. @ $20.00/ln.ft. = $ 3,840
c) 36" - 64 ln. ft. @ $31.00/ln.ft. = $ 1,980

Total $10,490 ($10,490)

10) Metal Apron End Walls for Culvert Pipes (each)

a) 18" - 10 @ $65/each = $ 650
b) 24" - 6 @ $108/each= $ 650
c) 36" - 2 @ $204/each= $ 410

Total $1,710 ($1,710)

11) Woven Wire Fencing Construction (ln. ft.)

3100 ln. ft. @ $1.85/ln. ft. = $5,740 ($5,740)

12) Seeding (2 lbs/1000 sq. ft.)

2745000 sq. ft. x 2 lbs/1000 sq. ft. x $3.50/lbs = $19,220 ($11,530)

13) Fertilizer (10 lbs/1000 sq. ft.)

2745000 sq. ft. x 10 lbs/1000 sq. ft. x $20/100 lbs. = $5,490 ($3,300)

14) Steel Plate Beam Guard (ln. ft.)

1000 ln. ft. @ $10.00/ln. ft. = $10,000 ($10,000)

15) Anchorages for Steel Plate Beam Guard (each)

4 each @ $135 = $540 ($540)

16) Riprap (cu.yd.)

250 cu.yd. @ $24/cu.yd. = $6,000 ($6,000)

17) Concrete Masonry Culvert (cu.yd.)

307 cu.yd. @ $160/cu.yd. = $49,120 ($49,120)

18) Granular Backfill (cu.yd.)

940 cu.yd. @ $2.87/cu.yd. = $2,700 ($2,700)

Route "B"

Total Major Construction Items	$613,040 ($546,550)
Construction Contingencies (10%)	$ 61,300 ($ 54,600)
Construction Total	$674,340 ($601,210)
Engineering Design (7%)	$ 47,200 ($ 42,080)
Construction Staking & Inspection (12%)	$ 80,920 ($ 72,160)
Total	$802,460 ($715,450)

Length of Route 3.5 miles

Cost per Mile $229,270 ($204,410)

Route "C" - Construction Cost Estimate

1) Right of Way (Acres)

a)	Mihalko, Thomas	4.7
b)	Torgerson, Theo	2.3
c)	Meister, Otto	2.7
d)	Mihalko Land and Logging Company	20.7
e)	Hoffman, Ray	6.1
f)	Exxon	18.9
	Total	55.4 (40.4)

2) Clearing (Acres)

44.7 acres @ $600/acre = $26,820 ($19,580)

3) Removing Fence (ln.ft.)

6000 ln.ft. @ $1.00/ln.ft. = $6,000 ($6,000)

4) Marsh Excavation (cu.yd.)

103200 cu.yd. @ $0.80/cu.yd. = $82,560 ($72,650)

5) Unclassified Excavation (cu.yd.)

135180 cu.yd. @ $1.15/cu.yd. = $155,460 ($116,590)

6) Crushed Aggregate Base Course (cu.yd.)
(Assume 9" thick, 44' wide)

15900 cu.yd. @ $5.70/cu.yd. = $90,630 ($90,630)

7) Bituminous Base Course (Ton)
(Includes bituminous material, assume 1-3/4" thick, 24' wide)

3185 tons @ $15.00/ton = $47,780 ($47,780)

8) Bituminous Concrete Pavement (Ton)
(Includes bituminous material, assume 1-1/4" thick, 24' wide)

2275 tons @ $15.00/ton = $34,130 ($34,130)

9) Corrugated Metal Culvert Pipe (ln. ft.)

a)	18" - 256 ln.ft. @ $14.60/ln. ft.	= $3,740
b)	24" - 128 ln.ft. @ $20.00/ln. ft.	= $2,560
c)	36" - 64 ln.ft. @ $31.00/ln. ft.	= $1,980
	Total	$8,280 ($8,280)

Route "C"

10) Metal Apron Endwalls for Culvert Pipe (each)

a) 18" - 8 @ $65/each = $ 520
b) 24" - 4 @ $108/each = $ 430
c) 36" - 2 @ $204/each = $ 410

Total $1,360 ($1,360)

11) Woven Wire Fencing Construction (ln.ft.)

8400 ln.ft. @ $1.85 = $15,540 ($15,540)

12) Seeding (2 lbs./1000 sq. ft.)

1950000 sq. ft. x 2 lbs/1000 sq. ft. x $3.50/lbs = $13,650 ($8,190)

13) Fertilizer (10 lbs/1000 sq. ft.)

1950000 sq. ft. x 10 lbs/1000 sq. ft. x $20/100 lbs. = $3,900 ($2,340)

14) Steel Plate Beam Guard (ln.ft.)

1000 ln.ft. @ $10.00/ln.ft. = $10,000 ($10,000)

15) Anchorages for Steel Plate Beam Guard (each)

4 each @ $135/each = $540 ($540)

16) Riprap (cu.yd.)

250 cu.yd. @ $24/cu.yd. = $6,000 ($6,000)

17) Concrete Masonry Culvert (cu.yd.)

307 cu.yd. @ $160/cu.yd. = $49,120 ($49,120)

18) Granuler Backfill (cu.yd.)

940 cu.yd. @ $2.87/cu.yd. = $2,700 ($2,700)

Route "C"

Total Major Construction Items	$554,470 ($491,430)
Construction Contingencies (10%)	$ 55,450 ($ 49,140)
Construction Total	$609,920 ($540,570)
Engineering Design (7%)	$ 42,690 ($ 37,840)
Construction Staking & Inspection (12%)	$ 73,190 ($ 64,870)
Total	$725,800 ($643,280)
Length of Route 2.5 miles	
Cost per Mile	$290,320 ($257,310)

EXXON MINERALS COMPANY, U.S.A.

Wisconsin Clay Deposits
December 1979

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INTRODUCTION

Purpose of the Report: The purpose of this report is to document occurrences of clay-rich soils in Wisconsin which could be considered for mining and use at Crandon as a disposal basin lining material. The availability of commercially produced clay from outside of the state has been considered in the second section of this two part report for the purpose of a cost comparison with local clay sources.

Conclusions and Recommendations: It is concluded that clay deposits within Wisconsin are generally poorly sorted, of limited size, and consist of calcium montmorillonite or illite. This places the in-state deposits at a distinct disadvantage when compared to cleaner, much larger, Western sodium montmorillonite deposits. The sodium based clays are preferred because of their superior swelling characteristics.

High land usage within Wisconsin greatly adds to exploration, development and acquisition costs. Many of the better clay deposits within this state cannot be considered for mining because they occur beneath or adjacent to environmentally sensitive areas such as wetlands and wildlife preserves. Land conflicts, where high yield farmlands overlie or surround some Wisconsin deposits could result in sensitive socioeconomic issues being raised. Therefore, the obtaining of a permit to mine clay could become another uncertainty that cannot be clearly quantified as to time nor cost.

It is recommended that commercially produced Western clays be considered as a primary source of tailings basin lining material.

Proposed Waste Disposal Rules and Regulations: State and federal rules and regulations have not been promulgated for the disposal of mine waste products nor lining of impoundment areas within Wisconsin. A part of the legislative delay is the government's uncertainty whether to classify pyrite-rich mine wastes as hazardous or non-hazardous products.

The Environmental Protection Agency (EPA) proposed rules on hazardous waste guidelines and regulations, and the Resource Conversation and Reclamation Act indicate that various thicknesses of in-place or introduced impermeable soils are required to line waste disposal areas. The impermeable soil thickness is dependent upon the waste classification and upon the facility design. Such design considerations as membrane liners or leachate collection systems may reduce the amount, but not the quality of the soil liner.

Soils Specifications: The EPA has proposed that earth materials used in a soils liner or natural in-place soil barrier shall meet the following minimum criteria:

- Be classified under the Unified Soil Classification System CL, CH, SC, and OH (ASTM Standard D2487-69),
- Allow greater than 30 percent passage through a no. 200 sieve (ASTM Test D1140),
- Have a liquid limit equal to or greater than 30 units (ASTM Test D423),
- Have plasticity greater than or equal to 15 units (ASTM Test D424),
- Have a pH or 7.0 or higher, and
- Have a permeability not adversely affected by anticipated waste.

Soils Liner Requirements Needed at Crandon: The latest proposed waste storage facility design encompasses an area of about 400 hectares (1000 acres). However,

segregation of potential leachate producing waste from chemically inert waste could reduce the area to be lined to about 240 hectares (600 acres).

It is estimated that a minimum of 3.7 million cubic meters (4.84 million cubic yards) of clay soils will be required to line 240 hectares to a depth of 1.5 meters (5 feet). A maximum of 385,000 cubic meters (13.6 million cubic yards) is anticipated to cover the same area to a depth of 3 meters (10 feet).

Data Source: Data sources used during the compilation of this report may be found under references and in Appendix I.

PART I THE OCCURRENCE OF CLAY-RICH SOILS IN WISCONSIN

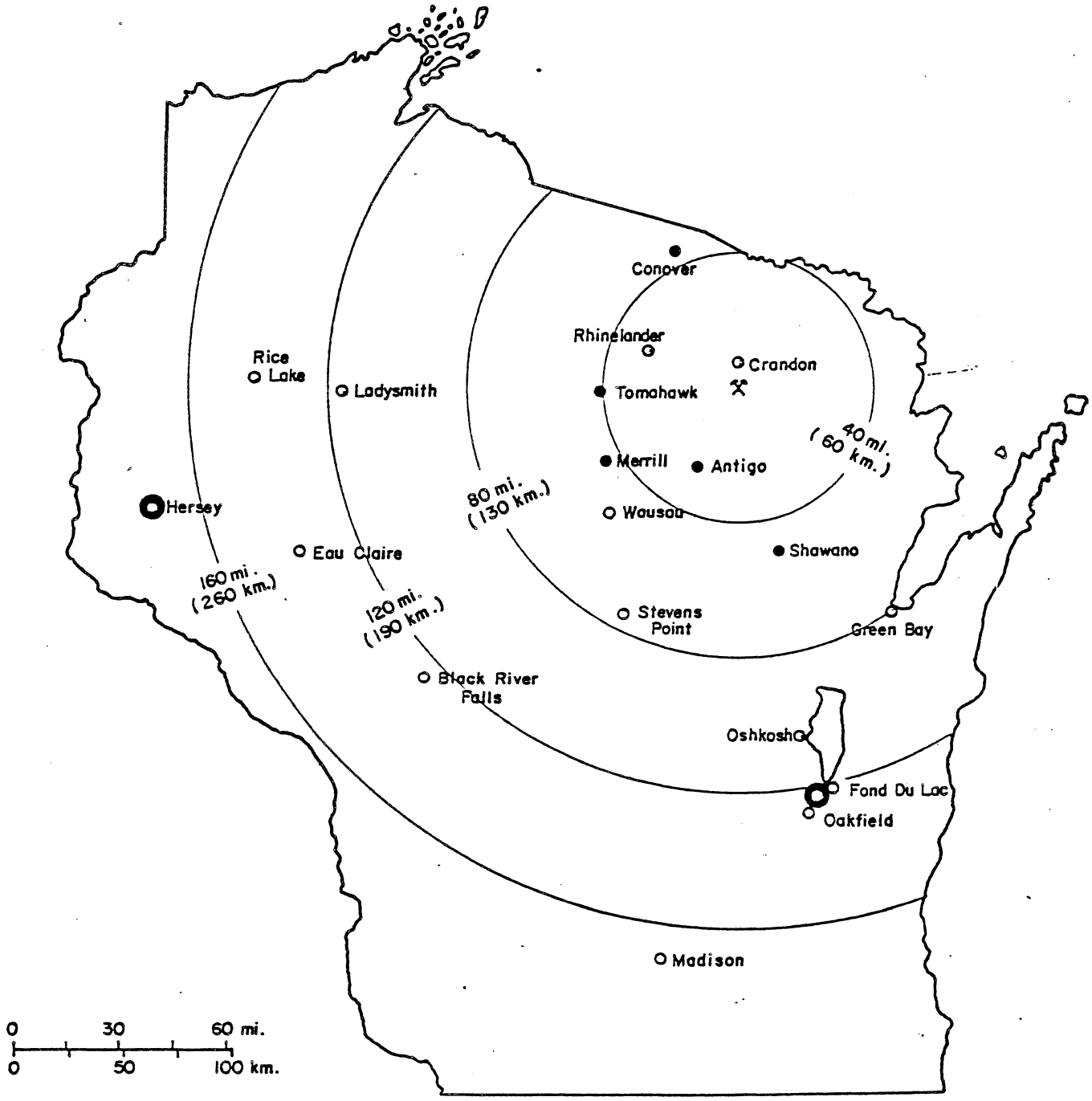
History of Clay Production: Near the turn of the century, Wisconsin had a large clay industry that produced building bricks, tile, and terra cotta products. In 1897, 225,000,000 bricks were produced from clay pits scattered across the state. Wisconsin also contained one of the nations' largest kaolinite clay pits located near Hersey, in Dunn County (Figure 1). A decline in the use of brick, lack of plant modernization, high energy costs, and the opening of larger, better quality clay deposits in other states led to the rapid decline of this once thriving industry.

Today, Wisconsin has only one producing clay mine. This pit, located near Oakfield (Figure 1), produces about 1800 tonnes (2000 tons) of illite clay per year.

Wisconsin Soil Classes: Wisconsin clay rich soils can be divided into two classes, residual and transported. Residual soils are formed by in-place bedrock

FIGURE 1

CLAY PRODUCING OR POTENTIAL CLAY PRODUCING AREAS
WISCONSIN



● ABANDONED CLAY PITS CLOSEST TO CRANDON

◎ PRODUCING OR POTENTIAL CLAY PRODUCING AREAS

disintegration. Transported soils are those soils which have been moved from the original bedrock and redeposited at another location (Krynine, 1957). Transporting agents were ice (glacial), water (fluvial and lacustrine), and wind (aeolian). The general distribution of residual and transported soils is illustrated on Figure 2.

Distribution of Residual Soils: Residual soils are located in the southwest and west central areas of Wisconsin. A narrow band of clay altered shale extends from the west side of the Door Peninsula south parallel to Lake Michigan and into Illinois. At least two areas of residual soils have been noted in northwest Wisconsin near the communities of Rice Lake and Ladysmith.

Residual Clay Deposits: Four distinct kinds of residual clay soils have been found within Wisconsin:

Precambrian Saprolite

Ordovician Maquoketa Shale

Cretaceous Hersey Soil

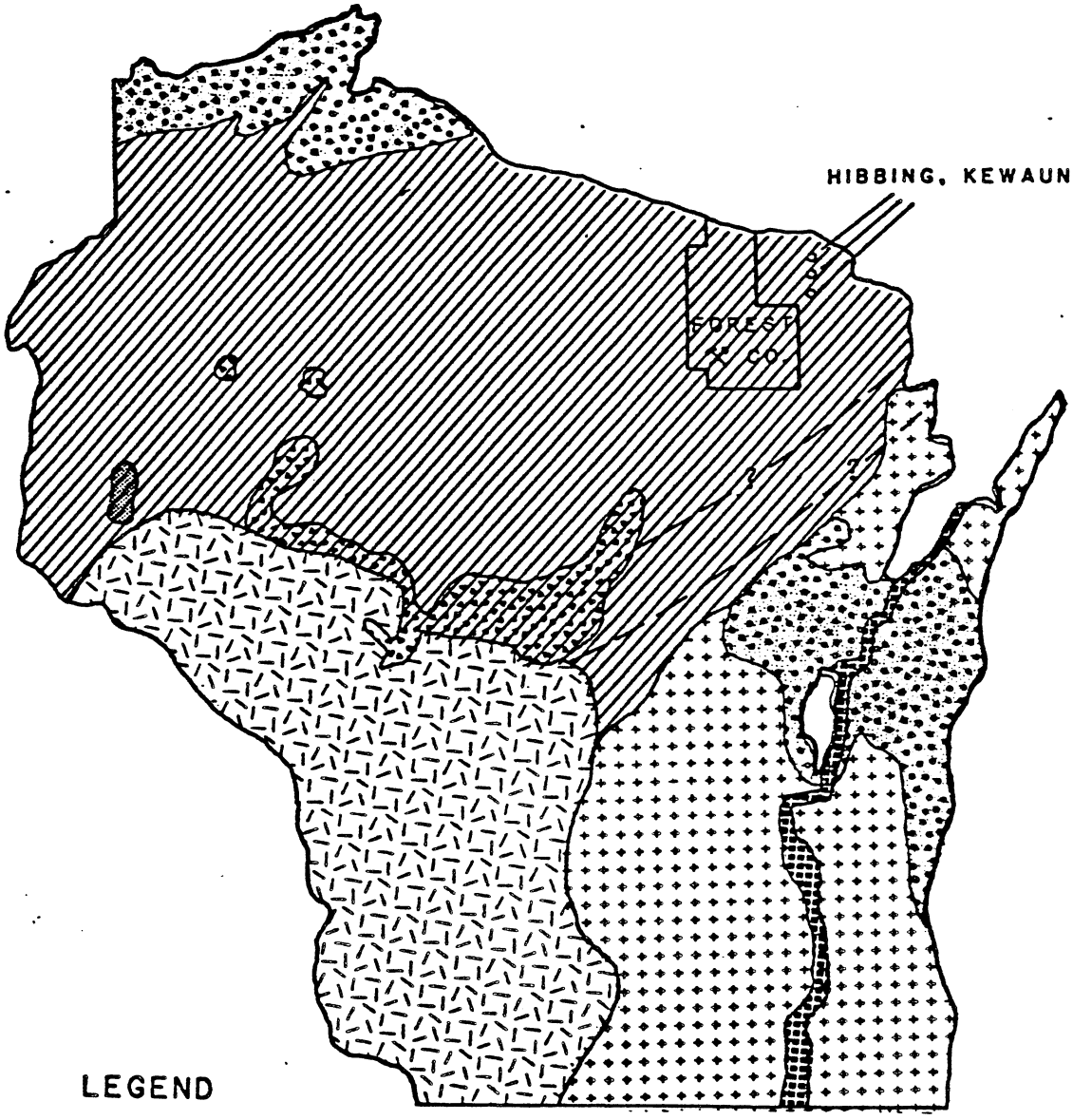
Driftless Area Soils

(a) Precambrian Saprolite. Saprolite does not appear to be a viable source of clay. Environmental concerns, variable permeabilities, and high development costs negate further consideration of the saprolite for use at Crandon.

Saprolite is a general term to denote a thoroughly decomposed, earthy, and untransported material. In Wisconsin, saprolite is a Precambrian soil that has been protected from erosion by deposits of Paleozoic rocks.

FIGURE 2

SIMPLIFIED RESIDUAL – TRANSPORTED SOILS MAP OF WISCONSIN



LEGEND

- IRON RIVER, GOGEBIC, WITHEE, OMEGA
- ONAWAY, DODGE
- HIBBING, KEWAUNEE
- THE DRIFTLESS AREA
- CRETACEOUS (?) HERSEY
- ORDOVICIAN MAQUOKETA
- PRECAMBRIAN SAPROLITE

KILOMETERS 0 80 160

MILES 0 50 100

Saprolite deposits are found scattered along or close to the contact between the steeply dipping Precambrian and more gently dipping Paleozoic rocks. This contact arcs convexly across Central Wisconsin from Rice Lake east to Wausau. Erosion has dissected the contact in several places to expose saprolite along river channels and at the base of valley wall bluffs. Saprolite soils have been recorded where the Chippewa, Black, and Wisconsin Rivers and some of their tributaries cross the Precambrian-Paleozoic rock contact.

Saprolite has not been recorded east of Wausau even though the Precambrian-Paleozoic contact continues to arc convexly to the northeast. Perhaps the absence of a major river system necessary to deeply erode the contact or thick moraine deposits, or a combination of these and other factors could explain why saprolite has not been recorded in this area.

The saprolite mineralogy and permeability can vary erratically since these physical properties are dependent upon the chemical composition of the underlying parent rock. Areas of saprolite underlain by volcanic rock having an intermediate composition are likely to contain micron-sized chlorite, quartz and mica. A granite parent rock would probably produce kaolinite. Materials other than clay are often present such as quartz fragments or blocks of partially decomposed bedrock. These materials often increase the saprolite permeability.

Saprolite was mined in the late 1990s' 95 km (60 Miles) southeast of the Crandon project site near Stevens Point. However, this and other more distal saprolite pits were small producers. Most pit operators mined about

a 3 meter thick layer of saprolite. One of the largest pits was located at Black River Falls, where a 4 meter (12 foot) thick layer was mined for a distance of 400 meters (1300 feet) along the Black River.

It is concluded that Precambrian saprolite is not a viable source of clay. Saprolite is generally found along major river channels which have dissected the Precambrian-Paleozoic rock contact. A pit developed this close to a navigable stream would present environmental difficulties. Most saprolite occurrences are capped by Paleozoic rock which increase the stripping ratio and removal cost.

(b) Ordovician Maquoketa Shale. The Oakfield-Fond du Lac area may warrant consideration for mining of a clay-rich shale. In many locations dolomite needs to be stripped to expose the Maquoketa formation. This dolomite could be a potential source of acid neutralizing rock.

The Maquoketa Shale formation is of late Ordovician age and is underlain by middle Ordovician Galena dolomite. The age of the overlying rock is in question, since the upper contact probably represents an erosional surface. In places, the Neda Iron Formation, which could be either late Ordovician or early Silurian, overlies the Maquoketa; elsewhere, the Silurian Mayville Dolomite forms the cap rock. All these rocks are a part of the Paleozoic sequence that forms the western edge of the Michigan sedimentary basin. These formations strike in a north-south direction, and dip very gently, about 5 degrees to the east. In areas of minor folding or flexturing, the beds may be horizontal or the dip reversed. The Maquoketa shale (Figure 2) has a true thickness of approximately 30 meters (100 feet).

Abandoned clay pits are scattered along the Maquoketa shale south of Green Bay. The Oakfield pit, a one man operation, is the only Wisconsin producing clay mine. Production records have been held confidential since 1976, but prior records indicate an annual production of 1800 tonnes from a 20 meter (60 foot) face. Personal communications with the State Geologist suggest that the Maquoketa shale has been altered to illite with minor chlorite. In some places, the decomposed shale can contain as much as 10 to 20 percent montmorillonite.

Mining the Maquoketa shale appears to be unfeasible north of Fond du Lac because of its close proximity to water (Lake Winnebago, Fox River and Green Bay), and because of overlying intensely productive farmland. Farmland, forest and Horicon Wildlife Refuge cover the shale south of Oakfield. Therefore, land acquisition costs will be high and social acceptance probably low to unacceptable in these areas.

The Oakfield-Fond du Lac area may warrant further consideration as a source of clay and dolomite that could be used to neutralize acid leachates. This area contains several abandoned dolomite quarries that overlie the Maquoketa shale. During quarrying, the Mayville dolomite was mined close to the shale, leaving a few meters of dolomite in the pit floor. The Oakfield area is accustomed to mining, so that its renewal would probably be tolerated. Some lower parts of the Maquoketa shale could contain deposits of sufficient size to supply a large plant (U. S. Geological Survey, 1976).

No engineering test data for Maquoketa shale samples has been found during preparation of this report. There is question in the authors' minds as to

whether the physical properties of the Maquoketa shale can meet the minimum EPA soil criteria. Adverse economics are anticipated due to high land acquisition, transportation, and environmental costs.

(c) Cretaceous (?) Hersey Soil. The Hersey area contains relatively thick, continuous, near surface, expandable clay soils. Mining of clay on a limited scale is considered viable at Hersey, but there is serious question whether there is sufficient quantity to meet the projected Crandon requirement.

Montmorillonite deposits of possible Cretaceous age occur scattered throughout the eastern part of St. Croix and the west part of Dunn Counties, (Figure 1). Several studies (Akers, 1961, Clum et al, 1976, and Dasgupta et al, in print) have been conducted to determine the suitability of this clay as a binder in pelletizing magnetite concentrates. It was concluded that these clays were suitable provided certain additives were used.

Seventy years ago, this area produced kaolinite which, at the peak of mining activity, ranked Wisconsin as one of the nation's largest producers. The kaolinite deposits are reported to be exhausted, but large volumes of calcium-rich montmorillonite remain close to the surface. An estimated 3.2 million tonnes (3.5 million tons) of montmorillonite was identified by Wisconsin Survey Geologists during an evaluation of the state's clay resources (Dasgupta et al, in print). Exxon Geologists estimate that there are 1.9 million tonnes (2.1 million tons) using a 3 meter minimum mining thickness.

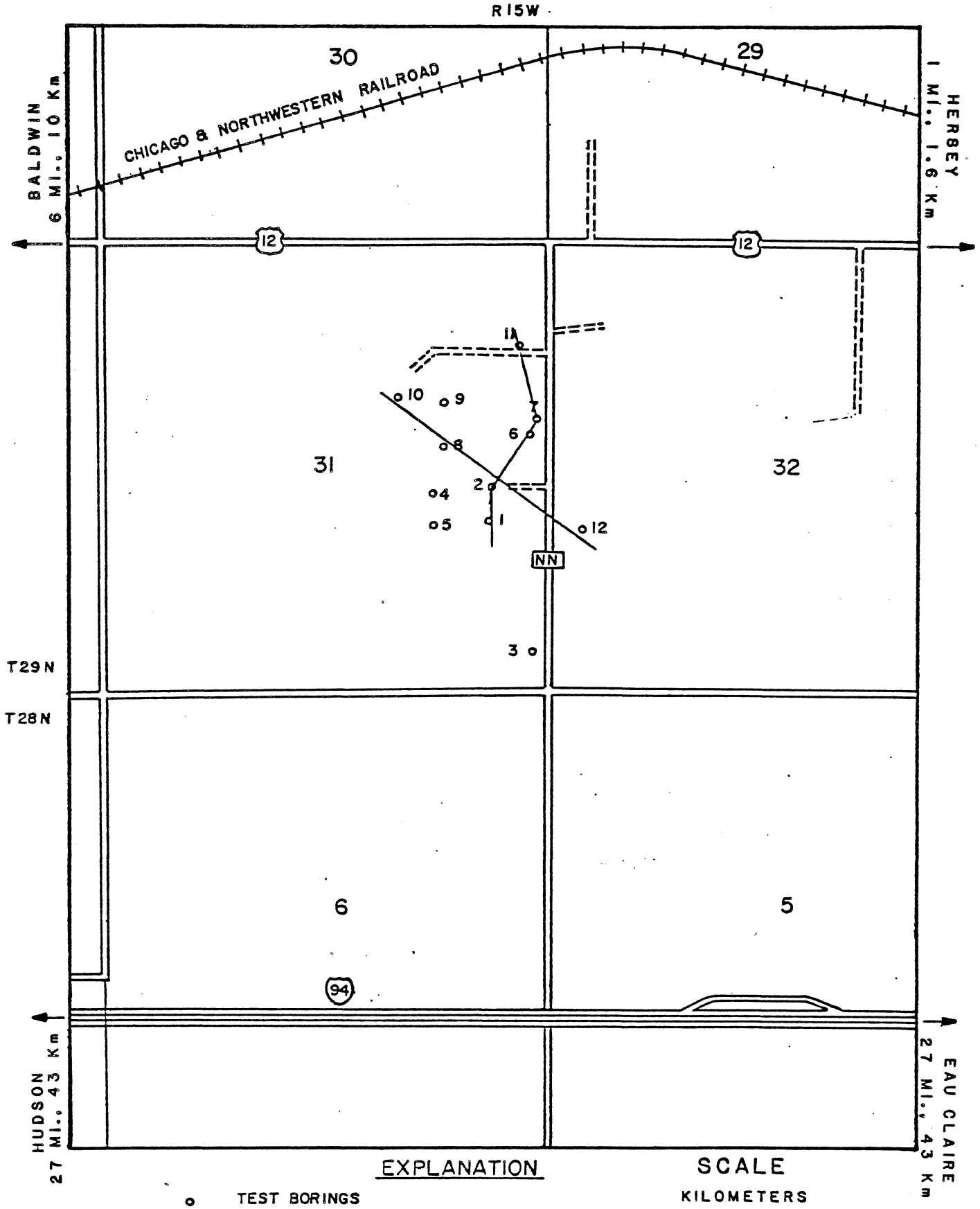
A twelve auger hole test program was undertaken by the Wisconsin Geological Survey in Section 31, Township 29 North, and Range 15 West, St. Croix County (Figure 3). Differential thermal and X-ray diffraction analyses of the augered samples determined that the clay consists of montmorillonite with quartz and trace amounts of vermiculite and illite (Wiederhdeft, et al, 1972). The -200 mesh sieve fraction contains 65 percent calcium montmorillonite. The chemical composition and size distribution data are summarized in Appendix II. No engineering test results have been found to determine whether this calcium montmorillonite clay meets EPA soil criteria. It is the authors' opinion that this clay would meet the EPA standards, but obviously samples need to be collected, tested, and the results analyzed in order to verify this conclusion.

A couple of fence diagrams were prepared from the twelve test boring logs. Fence I (Figure 4) was drawn in a north-south direction looking east; Fence II (Figure 5) was drawn at 45 degrees to Fence I and looking to the northeast. It is concluded that:

- the thickness of the clay soil increases to the east and northeast onto lands that were not test drilled,
- clay thicknesses range from a few centimeters to over 8 meters (26 feet). Drill logs are found in Appendix III.
- interbedded with the clay are narrow lenses of sand, silty sand and silty clay,
- an estimated 0.8 million cubic meters (1.040 million cubic yards) of clay is indicated.

FIGURE

Locations of clay test borings about 3 Km southwest of village of Hersey in the E 1/2 of Section 31 ,T 29 N, R 15 W, St. Croix County.

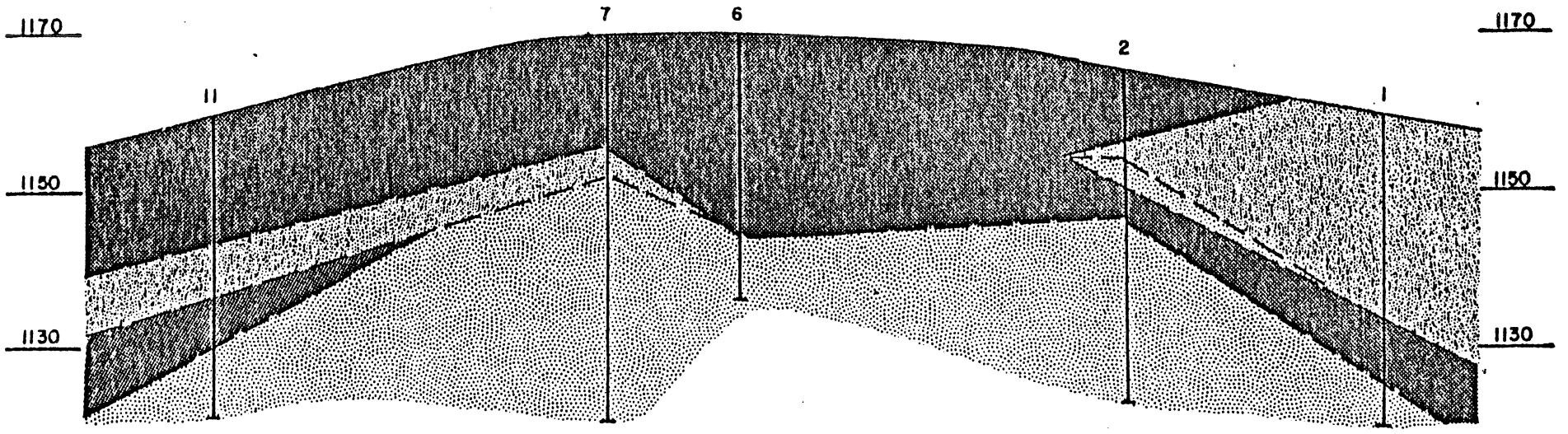


FENCE 1

SOIL SECTION
HERSEY, WISCONSIN

NORTH

SOUTH



LEGEND

CLAY

CLAYEY-SAND

SAND

HORIZONTAL SCALE 1" = 400'

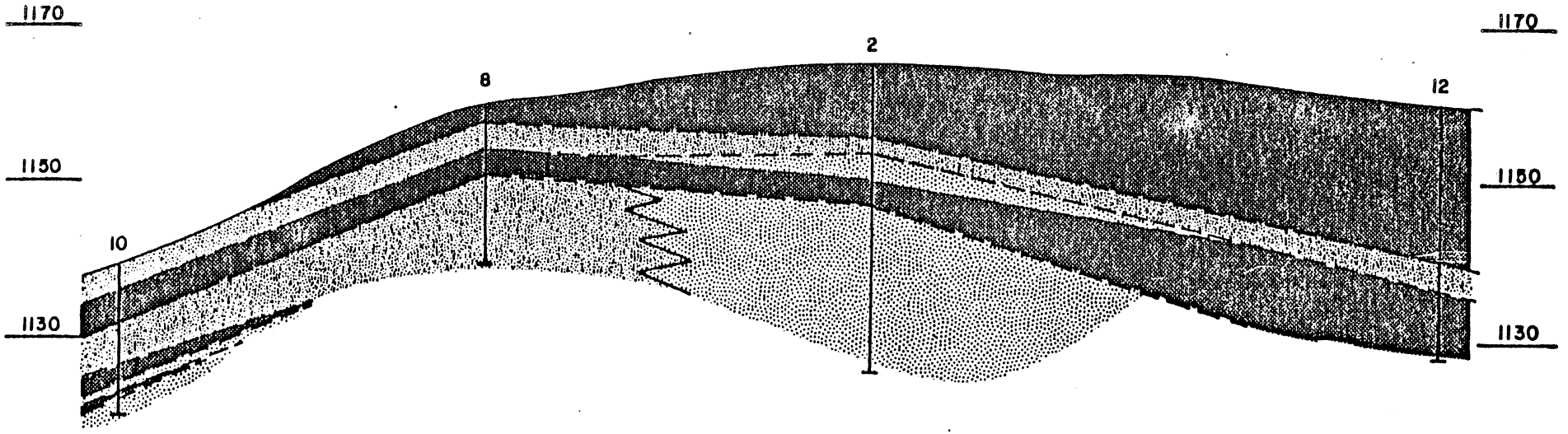
VERTICAL SCALE 1" = 20'

FENCE 11

SOIL SECTION
HERSEY, WISCONSIN

NORTHWEST

SOUTHEAST



HORIZONTAL SCALE 1" 400'

VERTICAL SCALE 1" 20'

LEGEND

CLAY

CLAYEY-SAND

SAND

This clay inventory could be significantly increased if drilling east of the road is successful.

The Hersey area offers one of the best sites for possible mining of relatively thick, continuous, calcium montmorillonite clay beds. Transportation facilities are excellent since the deposit lies within a mile of a railroad and interstate highway. However, transportation costs will be high to bring the clay 380 km (240 miles) to the Crandon site. Considerable amount of test work will need to be done in the field and laboratory to determine the quantity, and quality of the clay. Evaluation of these results would assist in concluding whether it is feasible to mine clay at Hersey versus the purchase of Western clays.

(d) Driftless Area Soils. The mining of the Driftless Area clay deposits can not be considered a viable alternative. These deposits are small, irregular in distribution, and are confined to the floors of steep sided valleys. Many of the deposits are poorly sorted, since they have been reworked and contaminated by glacially derived and more coarse grained material. It is recommended that this area receive no further consideration.

Underlying the Driftless Area are Upper Cambrian aged rocks that consist of the Elk Mound, Tunnel City and Trempealeau groups. These Cambrian sandstone, siltstone and dolomite rocks outcrop along and north of the Wisconsin River. Cambrian outcrops north of the Wisconsin River Valley are found at the headwaters of numerous short streams which flow west into the Mississippi. Ordovician dolomite with sandstone horizons is found on the highlands between the valleys in the west one half of the Driftless Area.

Residual clays have been formed by the alteration of the Cambrian and Ordovician calcareous rocks. Montmorillonite is the chief mineral with lessor amounts of kaolinite, illite and vermiculite. It has been suggested (Akers, 1961), that these residual clays have been contaminated by glacial and aeolian materials which were transported into the valleys. Therefore, most of these valley clay deposits are a mixture of poorly sort residual and transported materials.

The Driftless area soils are not considered a good source of clay suitable to line tailing ponds.

Distribution of Transported Soils: All of Wisconsin excluding the Driftless Area is mantled by a variety of transported soils. Forest County soils are transported by glacial, fluvial and lacustrine processes. For the purpose of this report, the transported soils have been divided into:

Pleistocene Glacial Drift, and
Pleistocene Lacustrine

(a) Pleistocene Glacial Drift. The mining of sufficient clay to meet Crandon needs does not appear to be viable in the Pleistocene Glacial Drift soil region. Clay soils that meet minimum EPA standards occur as thin beds. Considerable land disturbance would occur to mine clay in volumes sufficient to line the Crandon tailings ponds. Environmental restrictions preclude exploration of the most promising areas because of the close association between the clay-rich soils and wetlands.

The drift consists of very poorly sorted and highly interstratified morains

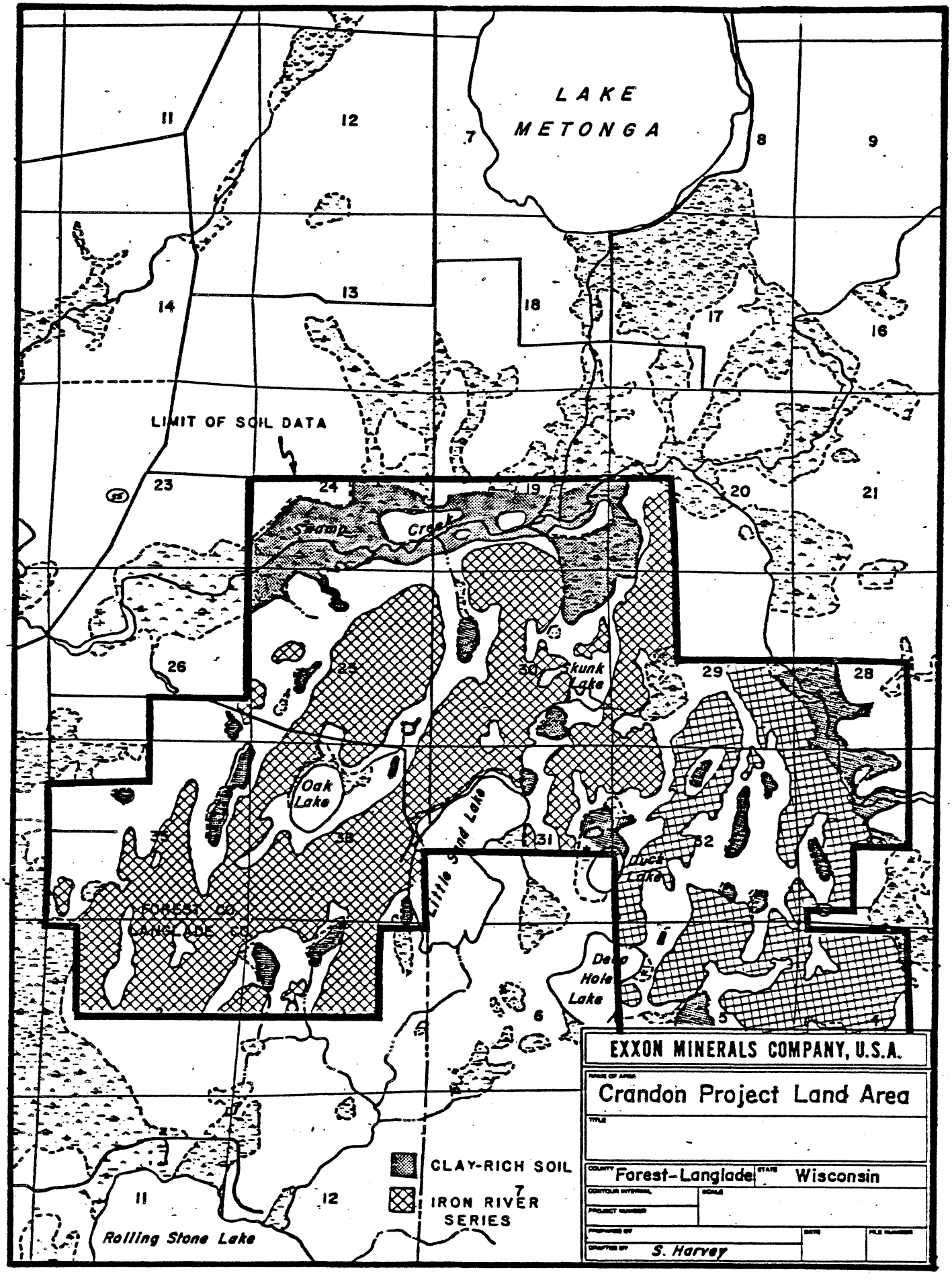
which were deposited during several glacial periods. Adding to the complexity of the drift soil profile was the deposition of fluvial outwash during glacial retreats. As the glacier withdrew, large blocks of ice were left partially or completely buried by outwash material. These blocks melted leaving depressions (kettles) scattered across the outwash plain. Into these flooded depressions were deposited silt and clay that accumulated to form thin impermeable lacustrine deposits. Most lacustrine deposits have overlying perched water tables which, where exposed to the surface, form lakes or wetlands. Therefore, there is a close relationship between many lacustrine clay deposits and wetlands.

The Soils Conservation Service office has mapped soils that overlie the Crandon project site (Figure 6). An overlay to Figure 6 illustrates the association between the clay-rich soils of the Gogebic-Iron River series and wetlands.

Three major soil regions overlie Northern Wisconsin. These are the Gogebic, Withee and the Omega. Engineering test data of these soils is found in Appendix IV. The Gogebic and Omega soils fail to meet minimum criteria established by the EPA. Representative samples of the Withee soils were tested and found to meet all EPA minimum standards. Withee soils occur in the northern and eastern one halves of Forest County as well as much of Central Wisconsin.

Many clay pits have been developed within the Pleistocene Glacial Drift area, but none remain in production today. Most of these pits lie 65 km (40 miles) from the project site, (Figure 1). One of the largest pits, the Shawano pit, contained 5 meters (15 feet) of clay beneath 1 meter (3 feet) of sand and

FIGURE 6



DATA SOURCE: USDA SOIL CONSERVATION SERVICE

topsoil (Buckley, 1901). Other area pits reported clay thicknesses ranging from 1 meter to 4 meters. The clay seams are poorly sorted containing intercalated sands with occasional pebble sized particles.

(b) Pleistocene Lacustrine. Mining of Pleistocene Lacustrine clay-rich soils does not appear to be a viable consideration for the Crandon project. Personal communications with the State Geologist indicate that many of these clay soils are difficult to mechanically compact to an acceptable EPA permeability once they have been disturbed. Some Kewaunee soils have been used successfully to clay line an industrial pond at Escanaba (personal communications, W. Perpich). However, the trucking distance to haul the Kewaunee soils to the Escanaba. site was less than 32 km (20 miles).

This soil region is divided into two soils, the Hibbing and the Kewaunee. These soils were deposited when river channels that drained Lakes Michigan and Superior were blocked by glacial ice. Consequently, the lake levels were elevated, and flooded parts of Northern and Eastern Wisconsin (Figure 2). Thick accumulations of clay were deposited until the present lake drainageways were reopened. The lake elevations were lowered, leaving perched clay beds.

The soil region covers about 1.0 million hectares (2.6 million acres) and contains a distinctive reddish-brown silty clay which is known as the Hibbing soils near Lake Superior and as the Kewaunee soils near Lake Michigan. These glacio-lacustrine deposits contain little or no gravel, although cebbly-sandy lenses do occur. The Hibbing soils contain more clay, but are less calcareous than the Kewaunee (Table I).

Table I
A Comparison of the Hibbing Kewaunee Particle Size

	Hibbing	Kewaunee	Hibbing	Kewaunee
Clay	55*	30-40*	50**	53**
Silt	35	30-40	45	30
Carbonate	10	25-35		
Sand			5	17

* Buckley, 1901
** Hole, 1976

The Kewaunee soils were studied by B. G. Lee (Lee, 1972) who reported montmorillonite as the most abundant clay mineral with lessor amounts of chlorite, vermiculite and kaolinite. Specific surface measurements of the Kewaunee medium sized clay fraction showed that about 36 percent total expanding silicates of which 23 percent is montmorillonite and 13 percent vermiculite. These clays are not as expandable as the Western NA-rich clays. Three areas of lacustrine clay soil, interpreted as part of the Hibbing-Kewaunee region have been mapped in Florence County (Figure 2). No engineering test data results has been located for these Florence County clay soils.

A matrix, Table II, in back pocket, compares engineering, environmental, and transportation concerns for the above described residual and transported soils.

PART II A COMPARISON OF COSTS TO PRODUCE CLAY FROM HERSEY, WISCONSIN VERSUS COMMERCIAL PURCHASE FROM A WESTERN SOURCE

Part II makes a very rough cost comparison between developing a clay deposit near Hersey, Wisconsin versus purchase of commercially produced Western clays. Several assumptions had to be made in order to make this cost comparison and these are discussed in a subsequent section.

The greatest uncertainties in making this evaluation are, (1) the tailings pond design is not finalized, (2) government rules that dictate the amount of clay liner to be used are not promulgated, (3) the Hersey clay deposit has not been delineated, and (4) there is a dearth of engineering test data by which to compare the Hersey clays with the Western States bentonite product.

The cost per ton delivered for the two clay products is close (Table III). However, there are intangible costs at Hersey, which are difficult to quantify with the available data. These intangible costs are directly linked to the needs to satisfy environmental concerns in order that permits be granted for clay mining.

It is recommended that we pursue the purchase of commercially produced Western clay, since there is little indicated cost advantage or assurance of sufficient local product. Furthermore, it is suggested that our engineers determine the volume of high quality betonite that needs to be added to in-place soils beneath the proposed tailings ponds. The selection of in-place materials having the best soils properties, including low permeability, is important. The identification

TABLE III

A Cost Comparison between developing the Hersey Clay versus purchase of clay from a commercial source.

(Cubic Meters - C.M. X 1,000,000)

Possible Clay Source	Cost/Ton	CASE I	CASE II	
		Acreage 240 hectares Thickness 1.5 meters Crandon needs 3.7 c.m. Hersey reserves 0.8 c.m.	Acreage 400 hectares Thickness 1.5 meters Crandon needs 5.2 c.m. Hersey reserves 1.9 c.m.	Acreage 400 hectares Thickness 3 meters Crandon needs 10.4 c.m. Hersey reserves 1.9 c.m.
Hersey Bulk - truck	58.30	$ 30 million	$ 63 million	Sufficient reserves not indicated
NS Bentonite Wyoming Bulk - rail	63.55	$ 21 million	$ 35 million	$ 42 million

The above costs do not include rail siding facilities, transportation of clay on site, nor application and compaction of clay to form the liner.

of the best soil horizon into which bentonite clay could be introduced, mixed, and compacted according to specifications could reduce the amount of clay needed to be purchased.

Assumptions used to make a very preliminary cost comparison:

CASE I

- at least 240 hectares of tailings ponds will be lined using clay.
- a minimum of 1.5 meters of clay liner having an impermeability of 1.0×10^{-7} cm/sec.
- there are 979 kilograms of clay per cubic meter.
- Hersey clay reserves are 0.8 million cubic meters using 3 meters as a minimum mining thickness.

CASE II

- a maximum of 400 hectares needs to be clay lined.
- a minimum of 1.5 meters of clay liner will be government required.
- Hersey clay reserves are 1.9 million cubic meters base on the Wisconsin geological survey estimate (Dasgupta, 1977).
- 240 hectares will need to be clay lined.
- 12 kilograms of bentonite will be required per square meter in order to achieve minimum EPA standards of 10^{-7} transmissivity.

CASE I

Land Acquisition: (200 hectares plus Surface improvements)	$ 1,250,000
Evaluation Project:	
Drilling (80 holes to 8 m) + Mobilization, Costs, Etc.	250,000
Engineering Studies	130,000
Relocate County "NN"	170,000
Bulk Mining of Surface Clay	1,300,000
Capital - Drying and Screening Plant, Conveyors, etc.	4,750,000
Crush, Dry and Screen to -200 mesh	1,100,000
Transport - Stockpile to Crandon (380 km)	12,250,000
Reclamation	2,100,000
	$23,300,000
Contingency :25%	5,825,000
	$29,125,000

Total Hersey production	0.8 million cubic meters
Crandon minimum clay requirements	3.7 million cubic meters
Deficit	2.9 million cubic meters

This means that about $30 million dollars would generate less than 25 percent of the Crandon clay requirement.

CASE II

Land Acquisition (490 hectares for Surface improvements)	$ 3,300,000
Evaluation/Engineering	1,700,000
Road Relocation	170,000
Bulk Surface Mining	3,125,000
Stripping	1,100,000
Capital	5,000,000
Crush, Dry, and Screen	2,775,000
Transport	29,500,000
Reclamation	3,600,000
	$50,270,000
Contingency :25%	12,500,000
	$62,770,000

Total Hersey Production	1.9 million cubic meters
Crandon maximum requirements	5.2 million cubic meters
Deficit	3.3 million cubic meters

An approximate $63 million dollar outlay would provide about 37 percent of the Crandon clay requirement. The most stringent interpretation of federal landfill regulations suggests the installation of 3 meters of clay soil liner. Therefore, to line 400 hectares of tailings pond to the maximum proposed clay soil thickness, would require 12 million cubic meters (16 million cubic yards) of material. The Hersey deposit can only supply 16 percent of this clay requirement.

Engineers from commercial producers indicate that National Standard Bentonite (200 mesh) applied at a rate of 12 kilograms per square meter (2.5 pounds/sq. ft.)

prehydrated with water and mixed with the upper 5 centimeters (2 inches) of sandy loam prior to compaction, will yield a permeability of 10^{-6} cm. Current prices for National Standard Bentonite (NSB) at 200 mesh is as follows:

F.O.B. Bagged Wyoming 200 mesh NSB - English Tons

Truck: $34.10/ton - bagged
Rail: $33.10/ton - bagged
- add $7.50/ton if palletized

Current freight rates for bagged bentonite

Truck: $1.69/100# + 9½% fuel surcharge
Rail: $2.01/100# + 2¼% fuel surcharge

Current price of bulk bentonite, F.O.B., Wyoming

Rail: $24.70/ton (may depend on car availability)
Freight: $1.90/100# + 2¼% fuel surcharge

It is doubtful whether there is 5.2 million cubic meters of available clay at Hersey. To find out will require time, land position, large drilling expenditure, and a costly soils engineering test program. It is recommended that we further pursue a commercial clay source.

Commercial suppliers recommend application of 12 kilograms of NSB in 15 cm (6 inch) lifts. The cost of a 15 cm lift is:

- Truck - Bagged:

Bentonite:	$1,114,047.00
Freight:	1,209,149.00
	$2,323,196.00
Pallets:	245,025.00
Total:	$2,568,221.00

- Rail - Bagged:

Bentonite:	$1,081,377.00
Freight:	1,342,884.00
	$2,424,261.00
Pallets:	245,025.00
Total:	$2,669,286.00

- Rail - Bulk:

Bentonite:	$ 806,949.00
Freight:	1,269,393.00
Total:	$2,076,342.00

Therefore, to purchase clay for a 1.5 meter thick clay soil liner via a bulk rail system will cost:

$2.08 million x 10 = $20.80 million.

Clay purchase cost for a 3 meter clay-soil liner would be $41.60 million for 240 hectares.

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Appendix I. Persons consulted during the compilation of this report.

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Dr. G. La Berge, Assistant Professor, Department of Geology, Oshkosh,

Dr. G. B. Lee, Professor, Department of Soils Science, Madison,

William Ludwig, Engineer, Soils Conservation Services, Rhinelander,

Dr. Michael Mudrey, Geologist, Wisconsin Geological Survey, Madison,

Dr. M. E. Ostrum, Director, Wisconsin Geological Survey, Madison,

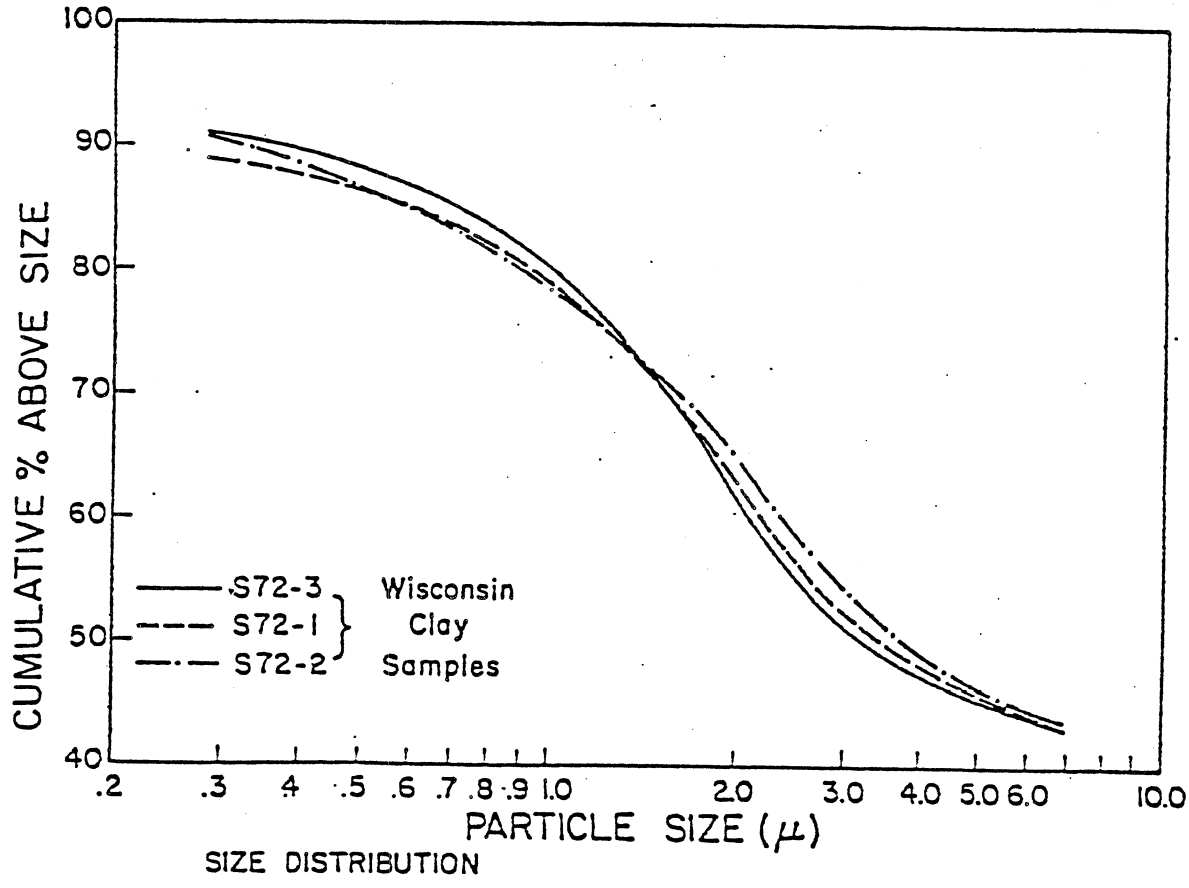
William Perpich, President, Soils Testing Services, Green Bay,

Dr. E. J. Tyler, Assistant Professor, Department of Soils Science, Madison.

APPENDIX II

Chemical Composition and Size Distribution
of
Hersey Clay Samples

SiO_2	52.41
Al_2O_3	34.10
Fe_2O_3	0.15
CaO	0.05
MgO	0.12
Na_2O	0.18
K_2O	0.46
TiO_2	0.80
H_2O	11.89
Total	100.16

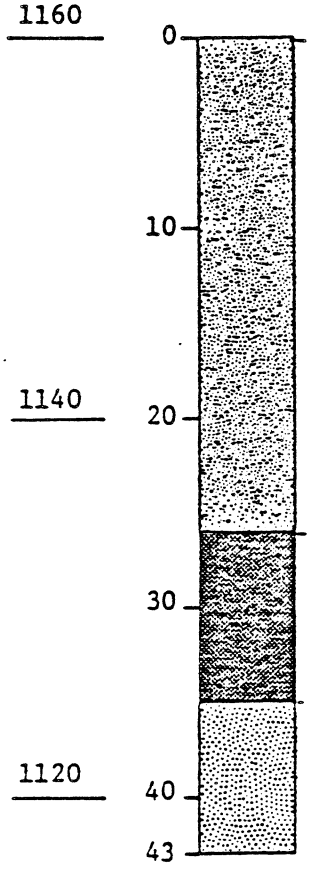


SIZE DISTRIBUTION

APPENDIX III

Auger Drill Logs
Hersey, Wisconsin

HOLE #1



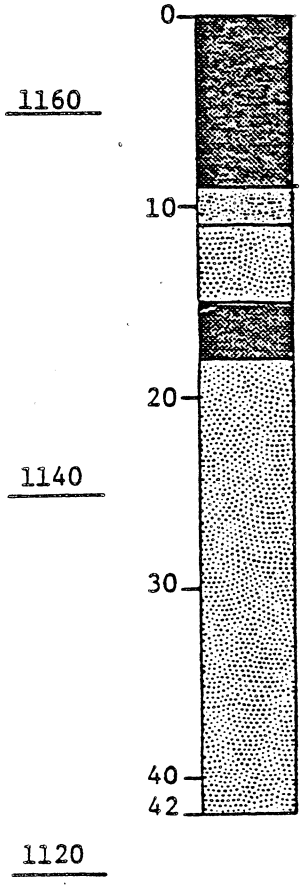
0-26' Clayey Sand Sand with minor clay, very light yellow gray, quartz.

Sand, quartz, with minor clay, light yellow brown, medium- and fine-grained. Clay primarily kaolinite with minor montmorillonite.

26-35' Clay Clay, dark brown gray to light yellow brown; less than 5 percent quartz sand. Clay primarily montmorillonite with minor to no kaolinite.

35-43' Sand Sand, quartz, with trace of clay, yellow brown, medium- and fine-grained. Clay about equal proportions of kaolinite and montmorillonite.

HOLE #2



0-9' Clay Clay, brown mottled and streaked yellow brown, very plastic; 5-10 percent quartz sand; few rock fragments and little white chert; plant fragments 3 to 4 feet. Clay is montmorillonite with very minor kaolinite.

T

9-11' Clayey Sand Clay, brown gray and yellow brown; 30 percent quartz sand. Clay is montmorillonite with minor kaolinite.

11-15' Sand, quartz, light yellow brown; 10 percent clay; scattered fragments of white chert to ½ inch. Clay is montmorillonite with very minor kaolinite.

15-16' Clay Clay, gray mottled yellow brown, plastic, waxy. Clay is montmorillonite with minor kaolinite.

16-18' Sand Clay, gray brown; sandy with chert fragments to ½ inch. One fragment oolitic indicative of Ordovician Prairie du Chien Group dolomites. Clay is kaolinite with very minor to no montmorillonite.

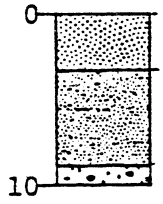
18-42' Sand, light brown gray, medium- and fine-grained; minor white clay; few fragments of oolitic white chert 38-42'. Proportion of kaolinite to montmorillonite estimated to be 7:3.

T Tested for binding taconite pellets.

HOLE #3

1160

1140



0-3' Sand Soil and sand. Sand has variable mineral composition and is medium- and coarse-grained; 6 inch brown clay at base. Likely glacial deposit.

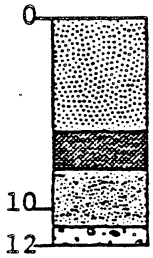
3-8' Clayey Sand Sand, brown, fine- and medium-grained; mostly quartz, clayey, few pebbles. Clay about equal proportions of kaolinite and montmorillonite.

8-9' Gravel or altered bedrock, dolomitic.

1120

HOLE #4

1160



0-1' Sand Soil

1-6' Clay Sand, yellow brown, quartz, fine- to medium-grained. Little clay and few limonite pebbles from 3 feet to 6 feet. Clay about equal proportions of kaolinite and montmorillonite; little illite 4 to 6 feet.

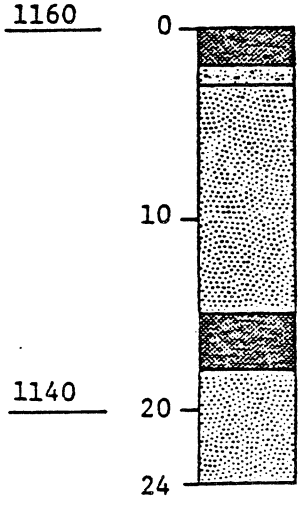
6-8' Clay, yellow brown and light green to white. Yellow brown is slightly streaked with light green and contains s and little very fine sand. Pale green only slightly silty with no sand. Both contain pebbles of limonite and sandy limonite. Clay is montmorillonite and illite in about equal proportions with minor kaolinite.

8-11' Clayey Sand Sand, brown and red brown, medium- and fine-grained, quartx, slightly clayey. Clay is kaolinite and montmorillonite in approximate proportion of 7:3.

11-12' Gravel and sand, slightly clayey, and sand as above. Pebbles of sandstone up to 1 inch. Clay kaolinite and mortmorillonite in about equal amounts.

1140

HOLE #5



0-2' Clay Clay, white, silty. Clay is kaolinite with only slight trace of illite.

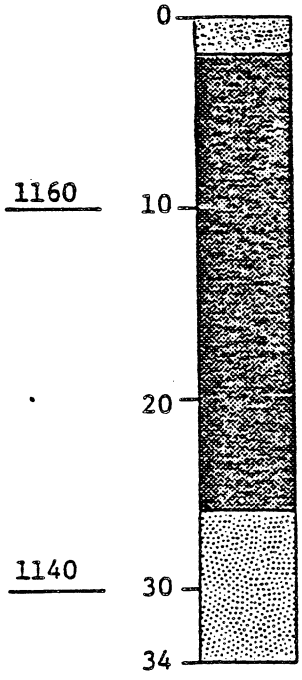
2-3' Clayey Sand Clay and sand, dark gray brown. Clay consists of approximately equal proportions of kaolinite, montmorillonite, and illite.

3-15' Sand Sand, light gray, quartz, fine- and medium-grained; little clay. Clay is kaolinite and montmorillonite in approximate proportion of 6:4.

15-18' Clay Clay, red brown with partings of pale green clay, silty; little quartz sand; few pebbles of limonite in low foot. Trace white clay. Clay about equal proportions of kaolinite and montmorillonite (poor record).

18-24' Sand Sand, red brown, abundant sandstone pebbles in lower 2 feet; very clayey in lower 1 foot. Clay primarily montmorillonite (poor record).

HOLE #6



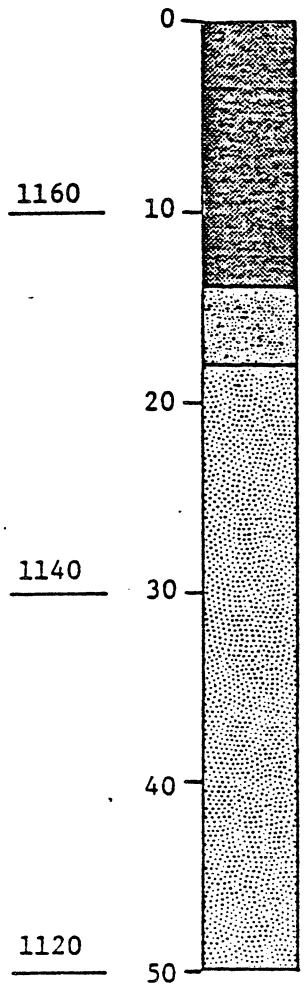
0-2' Sandy Clay Clay and sand, yellow brown, plastic. Clay predominantly montmorillonite with little kaolinite.

2-5' Clay Clay, yellow brown, plastic; little sand. Clay predominantly montmorillonite with little kaolinite.

5-26' Clay, yellow brown and gray, plastic; trace to little sand; plant fragments 17 feet to 18 feet. Clay more than 90 percent montmorillonite.

26-34' Sand Sand, yellowish gray, medium- and fine-grained; little clay. Clay more than 90 percent montmorillonite.

HOLE #7



0-3' Clay Clay, dark brown gray, slightly sandy with pebbles. Clay predominantly montmorillonite with little kaolinite.

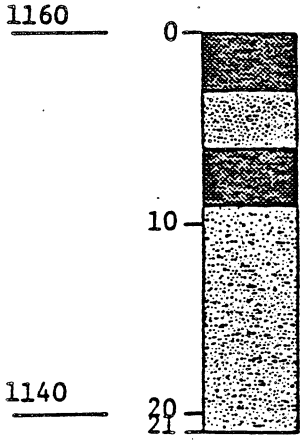
3-9' Clay, light yellow brown and gray, plastic; trace of sand. Clay predominantly montmorillonite with little kaolinite.

9-14' Clayey Sand Clay, dark gray to light yellow brown, plastic; trace of sand. Clay predominantly montmorillonite with little kaolinite.

14-18' Sand, light yellow brown; little clay. Clay predominantly montmorillonite with trace kaolinite.

18-50' Sand Sand, light yellow brown, fine- to medium-grained, quartz. Not sampled.

HOLE #8



0-2' Clay Clay, dark gray brown and yellow brown, plastic; trace of sand. Clay montmorillonite with trace of kaolinite.

2-3' Clay, brown gray, plastic. Clay montmorillonite with trace of kaolinite.

3-6' Clayey Sand Sand and clay, light brown gray, sand is quartz; trace white chert lower foot. Clay montmorillonite with trace of kaolinite.

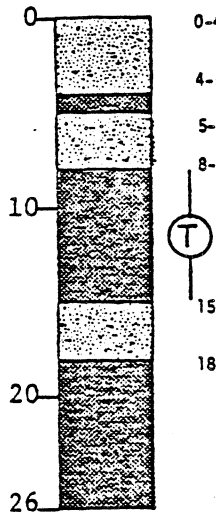
6-9' Clay Clay, light yellow gray, plastic, slightly sandy. Upper foot montmorillonite; lower 2 feet montmorillonite and kaolinite in about equal proportions.

9-14' Clayey Sand Sand and clay, light yellowish gray, sand is quartz. Montmorillonite and kaolinite in about equal proportions; kaolinite locally more abundant.

14-21' Sand and clay; clay white and gray, plastic. Clay kaolinite and montmorillonite in approximate proportion of 5:1.

HOLE #9

1160



1140

0-4' Clayey Sand Sand and clay, brown; few chert pebbles. Clay montmorillonite with little kaolinite.

4-5' Clay Clay, brown, plastic; trace of quartz sand. Clay predominantly montmorillonite with little kaolinite.

5-8' Clayey Sand Sand and clay, brown; sand quartz. Clay predominantly montmorillonite with little kaolinite.

8-15' Clay Clay, gray and brown gray, plastic; trace of quartz sand. Clay montmorillonite with slight trace of kaolinite.

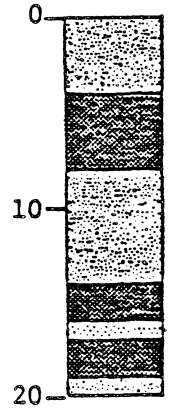
15-18' Clayey Sand Sand, yellow brown and brown gray; trace of clay. Clay is predominantly montmorillonite with trace of kaolinite.

18-26' Clay Clay, gray brown, plastic; little quartz sand; few chert pebbles in upper foot. Clay montmorillonite with trace to little kaolinite.

HOLE #10

1160

1140



1120

0-4' Clayey Sand Sand and clay, brown. Clay montmorillonite with trace of kaolinite.

4-8' Clay Clay, brown mottled light gray, plastic; trace of quartz sand in upper 2 feet. Clay montmorillonite with trace kaolinite.

8-14' Clayey Sand Sand and clay, light yellow gray and brown. Kaolinite and montmorillonite in approximately equal proportions.

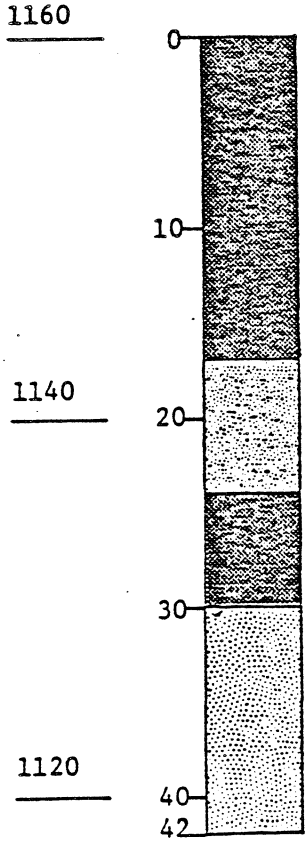
14-16' Clay Clay, gray, plastic. Clay montmorillonite with trace of kaolinite.

16-17' Sand Sand with trace of clay, light yellow brown. Clay montmorillonite with little kaolinite.

17-19' Clay Clay, light yellow brown; little sand. Clay montmorillonite with little illite and trace of kaolinite.

19-20' Clayey Sand Clay and sand, brown and reddish brown; few igneous rock pebbles. Clay mortmorillonite with little illite and kaolinite.

HOLE #11



0-15' Clay Clay, yellow brown with gray streaks, plastic; slightly sandy; few pebbles. Clay montmorillonite with less than 10 percent kaolinite.

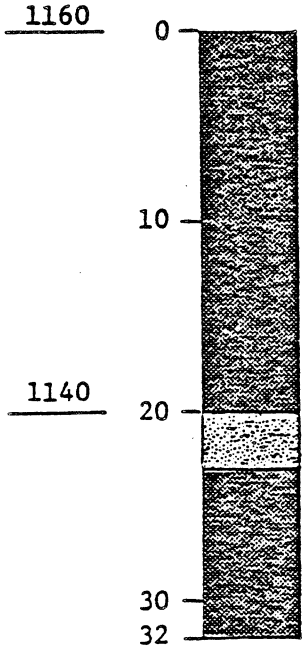
15-17' Clay, gray with some yellow brown, sandy to very sandy in lower 2 feet. Clay montmorillonite with trace of kaolinite.

17-24' Clayey Sand Sand, light yellow brown to yellow gray, medium- to fine-grained, slightly to moderately clayey. Clay montmorillonite with trace of kaolinite.

24-30' Clay Clay, gray, plastic, slightly sandy. Clay montmorillonite with trace of kaolinite.

30-42' Sand Sand, light yellow gray, medium- and fine-grained, quartz; trace of clay. Clay fraction consists of montmorillonite with only slight trace of kaolinite. Lower 6 inches very clayey and contains igneous rock pebbles. Clay in lower 6 inches montmorillonite with little kaolinite.

HOLE #12



0-3' Clay Clay, yellow brown, plastic, silty at top to sandy at base. Clay montmorillonite with trace of kaolinite.

3-7' Clay with abundant gravel consisting of egneous rock pebbles. Clay predominantly montmorillonite with trace of kaolinite.

7-20' Clay, gray streaked with light yellow brown, plastic, slightly sandy in upper foot. Clay montmorillonite with trace to little kaolinite in lower 2 feet.

20-23' Clayey Sand Sand with clay, yellow brown, quartz. Clay montmorillonite with trace of kaolinite.

23-32' Clay Clay, gray and yellow brown with trace red brown; slightly sandy 26 feet to 30 feet; egneous rock pebbles in lower foot. Clay montmorillonite with only slight trace of kaolinite.

APPENDIX IV

Soil Engineering Test Data

Soil name and sample location	Depth in inches	Horizon	Moisture density: Maximum dry density (lbs. per cu. ft.)	Moisture density: Optimum moisture (percent)	Mechanical analysis: Percentage passing sieve: 2"	1"	3/4"	3/8"	#4 4.7 mm.	#10 2.0 mm.	#40 .42 mm.	#200 .074 mm.	Percent smaller than: 0.05 mm.	0.02 mm.	.005 mm.	.002 mm.	Liquid limit	Plasticity index	Classification: AASHO (group index ratings in parenthesis)	Unified
Guenther loamy fine sand	30-38	B2*	-	-	-	-	-	-	-	100	81	28	26	18	10	7	-	NP	A-2-4(0)	SM
Wood Co., Wis.	45-60	C*	113	18	-	-	-	-	-	100	90	57	52	35	17	13	28	5	A-4(4)	ML
Hebron loam	-	B2	-	-	-	-	-	-	100	87	58	37	-	-	26	-	46	25	A-7-6(3)	SC
Racine Co., Wis.																				
Hebron sandy loam	15-36	B2	-	-	-	-	-	-	-	100	95	58	54	46	35	30	31	15	A-6(7)	CL
Ozaukee Co., Wis.	36-48	C	-	-	-	-	-	-	-	100	97	65	55	40	26	18	23	8	A-4(0)	CL
Hebron sandy loam	12-28	B2*	121	12	-	-	-	-	100	95	79	28	28	26	20	18	22			
Racine Co., Wis.	32-42	C*	113	17	-	-	-	-	-	100	99	96	96	95	64	44	36	18	A-6(11)	CL
Hiawatha loamy sand	4-19	B21r	-	-	-	-	-	-	-	100	81	21	20	14	7	5	-	NP	A-2-4(0)	SM
Bayfield Co., Wis.	26-60	C	107	14	-	-	-	-	-	100	93	5	3	2	1	1	-	NP	A-3(0)	SP-SM
Hibbing silt loam	16-27	B2*	114	16	-	-	-	-	-	100	97	73	69	62	46	37	37	22	A-6(13)	CL
Bayfield Co., Wis.	27-34	C*	119	13	-	-	-	-	-	100	96	69	64	55	41	31	33	20	A-6(11)	CL
	13-32	B2	-	-	-	-	-	-	-	100	96	81	79	72	54	37	40	23	A-6(13)	CL
	32-38	C	-	-	-	-	-	-	-	100	82	48	46	43	34	23	29	16	A-6(5)	SC
	16-34	B2	-	-	-	-	-	-	-	100	95	75	73	67	52	39	43	26	A-7-6(16)	CL
	34-40	C	-	-	-	-	-	-	-	100	94	73	71	66	49	35	36	20	A-6(12)	CL
	14-25	B2	-	-	-	-	-	-	-	100	88	60	57	41	17	12	19	4	A-4(5)	CL-ML
Hixton loam	29-34	C	-	-	-	-	-	-	-	100	48	6	5	4	3	3	-	NP	A-1-b(0)	SP-SM
Trempealeau Co., Wis.	13-20	B2*	128	10	-	-	-	-	-	100	73	49	48	37	16	11	21	5	A-4(3)	SC-SM
	26-42	C*	122	9	98	91	89	89	89	89	63	5	5	4	3	3	-	NP	A-3(0)	SP-SM
	21-29	B2	-	-	-	-	-	-	-	100	90	55	52	38	19	15	21	7	A-4(4)	CL-ML
	33-53	C	-	-	-	-	-	-	-	100	79	7	5	4	3	2	-	NP	A-3(0)	SP-SM
Hochheim loam	8-12	B2	-	-	-	-	-	-	-	100	95	74	69	60	43	37	53	31	A-7-6(19)	CH
Calumet Co., Wis.	18-30	C	-	-	100	90	88	81	76	68	60	39	32	21	11	7	-	NP	A-4(1)	SM
Hochheim silt loam	11-15	B2*	110	16	-	99	99	97	96	95	91	68	63	53	38	33	44	25	A-7-6(13)	CL
Fond du Lac Co., Wis.	25-40	C*	140	7	96	91	90	86	82	79	71	49	41	28	14	9	15	3	A-4(3)	SM
	12-15	B2	-	-	-	-	-	-	-	100	99	91	88	71	47	40	56	32	A-7-6(19)	CH
	30-40	C	-	-	-	90	84	69	57	40	19	10	8	5	3	2	-	NP	A-1-a(0)	SP-SM
	8-19	B2	104	19	100	97	97	96	96	95	93	77	74	62	47	39	48	27	A-7-6(17)	CL
	22-42	C	119	13	-	-	-	-	100	98	94	77	74	58	37	25	26	11	A-6(8)	CL
Hochheim silt loam	18-25	B22t	112	15	-	-	-	-	-	100	94	68	65	50	32	26	33	16	A-6(9)	CL
Manitowoc Co., Wis.	42-51	C1	131	9	99	94	92	89	85	82	75	46	39	24	12	8	19	4	A-4(2)	SC-SM

Soil name and sample location	Depth in inches	Horizon	Moisture-density		Mechanical Analysis												Liquid limit	Plasticity index	Classification	
					Percentage passing sieve								Percent smaller than							
			Maximum dry density (lbs. per cu. ft.)	Optimum moisture (percent)	2"	1"	3/4"	3/8"	#4 4.7 mm.	#10 2.0 mm.	#40 .42 mm.	#200 .074 mm.	0.05 mm.	0.02 mm.	.005 mm.	.002 mm.			AASHO (group index ratings in parentheses)	Unified
Hochheim silt loam	9-18	B2	-	-	-	-	-	-	-	100	99	81	77	62	45	38	47	26	A-7-6(15)	CL
Ozaukee Co., Wis.	21-42	C	-	-	-	98	98	96	95	94	92	71	64	40	23	16	20	6	A-4(7)	CL-ML
	7-15	B2	-	-	-	98	97	93	90	86	76	46	43	35	27	25	36	19	A-6(5)	SC
	20-42	C	-	-	100	85	82	74	64	58	46	21	17	9	5	4	-	NP	A-1-b(0)	SM
Hochheim silt loam	10-18	IIB2t	-	-	-	-	-	-	-	100	96	83	80	65	42	35	44	25	A-7-6(15)	CL
Sheboygan Co., Wis.	22-60	IIC	126	9	97	88	86	85	84	75	67	34	28	14	6	5	-	NP	A-2-4(0)	SM
Hochheim silt loam	8-17	B2	106	19	-	-	100	98	96	93	82	57	55	49	38	33	42	21	A-7-6(9)	CL
Waukesha Co., Wis.	21-60	C	140	6	100	93	88	80	74	67	56	35	32	22	11	7	14	2	A-2-4(0)	SM
Hortonville loam	21-27	B2	110	15	-	-	-	-	-	100	96	71	70	59	44	35	37	21	A-6(11)	CL
Brown Co., Wis.	33-40	C	116	13	-	100	99	97	95	95	89	67	65	54	37	25	28	13	A-6(8)	CL
Hortonville silt loam	19-25	B2*	103	21	-	-	-	-	-	100	98	87	85	77	49	35	42	22	A-7-6(13)	CL
Brown Co., Wis.	28-48	C*	119	13	-	98	96	94	93	91	87	78	76	64	31	17	25	8	A-4(8)	CL
Hortonville silt loam	11-25	B21t	-	-	-	-	-	-	-	100	98	78	75	63	47	36	42	22	A-7-6(13)	CL
Outagamie Co., Wis.	25-60	C	123	11	100	95	92	87	83	79	72	54	50	39	23	14	24	9	A-4(4)	CL
Hortonville silt loam	7-17	B2	-	-	-	-	-	-	-	100	96	76	72	61	47	39	45	25	A-7-6(15)	CL
Ozaukee Co., Wis.	20-42	C	-	-	-	99	96	94	92	89	84	65	62	49	31	22	29	14	A-6(8)	CL
	8-19	B2	104	19	100	97	97	96	96	95	93	77	74	62	47	39	48	27	A-7-6(17)	CL
	22-42	C	119	13	-	-	-	-	100	98	94	77	74	58	37	25	27	11	A-6(8)	CL
Hortonville silt loam	22-30	B2	109	17	-	-	-	-	-	100	97	73	71	63	46	33	42	23	A-7-6(13)	CL
Waupaca Co., Wis.	36-60	C	117	13	-	100	99	97	96	95	91	67	65	54	32	22	29	13	A-6(8)	CL
Iron River fine sandy loam	10-18	B21r	-	-	-	-	-	-	-	100	91	54	50	27	10	6	-	NP	A-4(4)	ML
Ashland Co., Wis.	18-24	A'2x	-	-	-	-	-	-	-	100	88	37	32	18	8	5	-	NP	A-4(0)	SM
	32-38	B'2tx	-	-	-	99	98	96	95	93	79	36	32	20	10	8	17	2	A-4(0)	SM
	38-60	IIC	132	7	100	95	93	87	82	76	57	16	13	7	3	2	-	NP	A-2-4(0)	SM
Iron River silt loam	17-28	B*	134	8	97	91	90	89	89	84	73	42	37	22	8	5	-	NP	A-4(1)	SM
Forest Co., Wis.	28-60	C*	135	8	100	90	88	85	85	85	76	39	35	20	9	6	-	NP	A-4(1)	SM
Iron River silt loam	33-47	IIB22t	-	-	100	94	92	86	82	78	62	27	24	15	9	6	-	NP	A-2-4(0)	SM
Sawyer Co., Wis.	64-70	IIC	-	-	-	99	97	92	89	85	70	29	25	15	6	4	-	NP	A-2-4(0)	SM
Jackson silt loam	-	C	-	-	-	-	-	-	-	100	100	98	-	-	23	-	29	10	A-4(8)	CL
Popin Co., Wis.																				
Jump River fine sandy loam	16-28	B2	-	-	-	-	-	-	-	-	100	85	80	44	20	15	28	4	A-4(8)	ML
Rusk Co., Wis.	32-50	C	-	-	-	-	-	-	-	100	99	19	18	8	6	4	-	NP	A-2-4(0)	SM

Soil name and sample location	Depth in inches	Horizon	Moisture-density: Maximum dry density (lbs. per cu. ft.)	Moisture-density: Optimum moisture (percent)	Mechanical Analysis: Percentage passing sieve: 2"	1"	3/4"	3/8"	#4 4.7 mm.	#10 2.0 mm.	#40 .42 mm.	#200 .074 mm.	Percent smaller than: 0.05 mm.	0.02 mm.	.005 mm.	.002 mm.	Liquid limit	Plasticity index	Classification: AASHO (group index ratings in parenthesis)	Classification: Unified
Kewaunee silt loam	8-22	B21t	-	-	-	-	-	-	-	100	97	83	82	76	56	44	49	30	A-7-6(18)	CL
Sheboygan Co., Wis.	36-60	C	-	-	-	-	-	-	-	100	97	85	82	69	46	31	30	15	A-6(10)	CL
Keweenaw loamy sand	4-8	B21ir	-	-	-	-	-	-	-	100	87	22	17	8	3	2	-	NP	A-2-4(0)	SM
Ashland Co., Wis.	37-60+	C	115	11	-	-	-	100	98	96	84	19	12	4	2	1	-	NP	A-2-4(0)	SM
Kibbie fine sandy loam	28-34	B2*	115	15	-	-	-	-	-	-	100	91	84	59	31	25	33	11	A-6(8)	CL
Dodge Co., Wis.	44-68	C*	113	16	-	-	-	-	-	100	99	96	92	61	28	20	31	10	A-4(8)	CL
Kibbie silt loam	15-22	B2	-	-	-	-	-	-	-	100	94	49	44	32	20	16	21	5	A-4(3)	SC-SM
Kenosha Co., Wis.	31-60	C	-	-	-	-	-	-	-	100	98	75	67	37	16	12	19	2	A-4(8)	ML
Kibbie silt loam	14-20	B2	-	-	-	-	-	-	-	-	100	89	87	77	36	25	35	16	A-6(10)	CL
Outagamie Co., Wis.	22-60	C	-	-	-	-	-	-	-	-	-	96	82	25	8	6	-	NP	A-4(8)	ML
	9-23	B22t	-	-	-	-	-	-	-	-	100	87	78	42	18	13	25	4	A-4(8)	ML
	48-60	C2	113	13	-	-	-	-	-	-	-	90	82	30	8	4	20	NP	A-4(8)	ML
Kibbie silt loam	17-20	B2*	111	16	-	-	-	-	-	100	99	53	47	36	27	22	29	11	A-6(4)	CL
Racine Co., Wis.	44-52	C*	120	12	-	-	-	-	-	-	-	79	68	34	11	7	-	NP	A-4(8)	ML
	21-28	B2	-	-	-	-	-	-	-	-	100	27	21	17	16	15	-	NP	A-2-4(0)	SM
	30-54	C	-	-	-	-	-	-	-	-	100	27	13	3	1	1	-	NP	A-2-4(0)	SM
Kibbie silt loam	9-17	B2	-	-	-	-	100	99	98	96	93	64	58	35	18	14	27	7	A-4(6)	CL-ML
Waukesha Co., Wis.	20-60	C	-	-	-	-	-	-	-	-	-	84	70	21	6	5	-	NP	A-4(18)	ML
Kidder sandy loam	11-26	B21t	-	-	-	-	-	-	-	100	94	39	38	31	23	19	23	9	A-4(1)	SC
Green Lake Co., Wis.	34-60	C	135	8	96	95	94	91	89	88	79	35	32	20	11	8	-	NP	A-2-4(0)	SM
Kiva gravelly loam	5-13	B21ir*	-	-	-	98	97	94	92	92	84	27	26	18	9	5	-	NP	A-2-4(0)	SM
Door Co., Wis.	32-36	IIC2*	-	-	96	62	55	43	36	27	14	6	5	3	1	1	-	NP	A-1-a(0)	GM-GW
Knowles silt loam	28-33	IIB24t	-	-	-	-	-	-	-	100	96	84	82	67	47	40	53	30	A-7-6(18)	CH
Green Lake Co., Wis.																				
Knowles silt loam	17-27	B2	-	-	-	-	-	-	-	100	96	72	71	60	45	40	44	24	A-7-6(13)	CL
Washington Co., Wis.	24-32	B2*	98	25	-	-	-	-	-	100	93	71	68	59	45	40	46	25	A-7-6(14)	CL
	18-26	B2	-	-	-	-	-	-	-	100	85	58	54	45	32	27	35	16	A-6(7)	CL
Kolberg silt loam	22-28	B22t	-	-	-	-	-	-	-	100	98	87	83	71	50	37	38	20	A-6(12)	CL
Door Co., Wis.	33-38	IIC	-	-	-	99	96	92	89	84	79	71	64	42	24	17	21	6	A-4(7)	CL-ML
Lafont silt loam	-	B2	-	-	-	-	-	-	100	94	81	46	-	-	11	-	17	2	A-4(2)	SM
Oneida Co., Wis.	-	C	-	-	-	-	-	-	100	83	69	29	-	-	8	-	12	1	A-2-4(0)	SM

Soil name and sample location	Depth in inches	Horizon	Moisture-density		Mechanical Analysis												Liquid limit	Plasticity index	Classification	
			Maximum dry density (lbs. per cu. ft.)	Optimum moisture (percent)	Percentage passing sieve								Percent smaller than						AASHO (group index ratings in parenthesis)	Unified
					2"	1"	3/4"	3/8"	#4 4.7 mm.	#10 2.0 mm.	#40 .42 mm.	#200 .074 mm.	0.05 mm.	0.02 mm.	.005 mm.	.002 mm.				
Karlin sandy loam	5-21	B21r	-	-	-	-	-	-	100	96	75	23	22	13	6	4	-	NP	A-2-4(0)	SM
Bayfield Co., Wis.	30-42	C1	109	12	-	-	100	99	96	91	69	2	2	2	1	1	-	NP	A-3(0)	SP
	42-53	C2	104	16	-	-	-	-	-	100	96	2	2	2	1	1	-	NP	A-3(0)	SP
Kennan loam	-	B2	-	-	-	-	-	-	100	91	71	30	-	-	12	-	18	3	A-2-4(1)	SM
Marathon Co., Wis.	-	C	-	-	-	-	-	-	100	86	64	20	-	-	8	-	-	NP	A-2-4(0)	SM
Kennan loam	13-27	IIB2t	-	-	-	98	98	94	91	85	67	24	23	19	15	13	23	8	A-2-4(0)	SC
Portage Co., Wis.	32-60	IIC	-	-	-	99	96	93	89	83	62	14	12	8	6	4	-	NP	A-2-4(0)	SM
Kennan silt loam	19-38	B*	132	8	-	99	99	96	92	87	68	20	18	14	8	6	-	NP	A-2-4(0)	SM
Portage Co., Wis.	52-60	C*	131	8	-	99	97	95	92	88	67	19	16	13	7	6	-	NP	A-2-4(0)	SM
	19-31	B	-	-	-	-	-	-	-	100	85	41	37	25	15	12	21	6	A-4(1)	SC-SM
	45-60	C	-	-	-	100	97	92	87	81	56	15	13	11	7	6	-	NP	A-2-4(0)	SM
Kennan silt loam	31-40	B	-	-	-	100	98	94	89	81	60	20	18	14	8	6	-	NP	A-2-4(0)	SM
Shawano Co., Wis.	44-50	C	-	-	-	98	96	91	85	85	62	18	15	11	7	5	-	NP	A-2-4(0)	SM
Kert silt loam	14-22	IIA&B	-	-	-	-	-	-	-	100	98	29	27	20	12	9	-	NP	A-2-4(0)	SM
Wood Co., Wis.	40-52	VC2	107	20	-	-	-	-	-	100	98	92	91	84	64	50	55	30	A-7-6(19)	CH
Kewaunee silt loam	11-20	B2t	-	-	-	-	-	-	-	100	97	68	66	60	46	35	41	25	A-7-6(13)	CL
Brown Co., Wis.	48-54	C	-	-	-	-	-	-	-	100	96	73	71	60	44	30	31	17	A-6(11)	CL
Kewaunee silt loam	11-20	B2	-	-	-	-	-	-	-	100	99	95	95	93	79	65	72	39	A-7-5(20)	CH
Calumet Co., Wis.	39-48	C	-	-	-	100	99	98	97	97	96	92	92	91	77	62	52	31	A-7-6(18)	CH
	12-23	B2	-	-	-	-	-	-	-	100	99	90	88	83	68	58	54	30	A-7-6(19)	CH
	34-55	C	-	-	-	97	96	95	93	91	88	79	77	68	51	37	35	18	A-6(11)	CL
Kewaunee silt loam	10-21	B2*	98	20	-	-	-	100	99	99	97	88	87	86	74	62	54	28	A-7-6(18)	CH
Fond du Lac Co., Wis.	28-47	C*	115	16	-	99	98	95	92	89	84	74	71	64	52	41	44	24	A-7-6(14)	CL
Kewaunee silt loam	21-30	C1	-	-	-	-	-	-	-	100	96	77	73	65	48	40	44	28	A-7-6(16)	CL
Manitowoc Co., Wis.	30-45	C2	-	-	-	-	-	-	-	100	94	78	74	64	48	36	34	20	A-6(12)	CL
	10-18	B2	-	-	-	-	-	-	-	100	98	76	74	67	53	45	47	29	A-7-6(17)	CL
	18-26	C	-	-	-	-	-	-	-	100	96	80	76	66	49	36	37	22	A-6(13)	CL
Kewaunee silt loam	10-22	B2	99	23	-	-	-	-	-	100	99	90	89	85	67	57	58	33	A-7-6(20)	CH
Milwaukee, Wis.	22-48	C	115	15	-	100	99	98	97	95	90	76	73	63	43	31	22	15	A-6(10)	CL
Kewaunee silt loam	7-20	B2	-	-	-	-	-	-	-	100	98	87	85	75	57	48	52	30	A-7-6(18)	CH
Ozaukee Co., Wis.	23-42	C	-	-	-	100	99	98	97	95	92	81	79	67	48	36	38	20	A-6(12)	CL

Soil name and sample location	Depth in inches	Horizon	Moisture-density		Mechanical Analysis												Liquid limit	Plasticity index	Classification	
			Maximum dry density (lbs. per cu. ft.)	Optimum moisture (percent)	Percentage passing sieve								Percent smaller than						AASHO (group index ratings in parenthesis)	Unified
					2"	1"	3/4"	3/8"	#4 4.7 mm.	#10 2.0 mm.	#40 .42 mm.	#200 .074 mm.	0.05 mm.	0.02 mm.	.005 mm.	.002 mm.				
Morley silt loam	15-23	B2	-	-	-	-	-	-	-	100	94	77	74	67	50	41	52	33	A-7-6(14)	CH
Kenosha Co., Wis.	30-54	C	-	-	-	100	98	95	93	92	86	71	64	44	26	17	21	8	A-4(7)	CL
	9-17	B2	-	-	-	-	-	-	-	100	97	83	81	74	54	43	52	31	A-7-6(18)	CH
	37-48	C	-	-	-	-	100	96	93	89	80	59	53	38	19	10	21	7	A-4(5)	CL-ML
	11-20	B2	-	-	-	-	-	-	-	100	99	92	92	91	70	60	60	35	A-7-6(11)	CH
	36-50	C	-	-	-	-	-	-	-	100	98	89	89	84	52	36	32	14	A-6(10)	CL
	10-24	B2	-	-	-	-	-	-	-	100	97	86	85	79	56	44	48	25	A-7-6(16)	CL
	36-48	C	-	-	-	-	-	-	-	100	97	81	75	49	21	15	20	3	A-4(8)	ML
Norden silt loam	18-23	B21	110	15	-	-	-	-	-	100	99	91	82	50	28	21	27	8	A-4(8)	CL
Dunn Co., Wis.	23-28	B22	114	16	-	-	-	-	-	100	99	59	52	40	32	27	34	17	A-6*8)	CL
	14-28	B21*	107	17	-	-	-	-	-	-	-	99	95	60	30	26	35	14	A-6(10)	CL
	34-40	B22*	115	15	-	-	-	-	-	100	99	80	72	46	27	22	30	12	A-6(9)	CL
Norden silt loam	16-20	B2	105	18	-	-	-	-	-	-	-	96	92	60	32	28	42	20	A-7-6(12)	CL
Pepin Co., Wis.	38-50	C	115	12	94	87	83	79	76	73	73	15	12	11	7	7	-	NP	A-2-4(0)	SM
Octagon silt loam	13-16	B22t	105	19	-	-	-	-	-	100	98	84	84	69	42	35	51	34	A-7-6(18)	CH
Dane Co., Wis.	33-60	C1	13[illegible]	8	-	96	94	90	87	84	75	28	25	14	7	6	-	NP	A-2-4(0)	SM
Okee loamy fine sand	10-25	B1*	-	-		99	99	96	93	91	82	12	10	7	5	3	-	NP	A-2-4(0)	SP-SM
Green Lake Co., Wis.	31-45	IIB22t*	-	-	-	-	-	-	-	100	92	29	27	21	16	14	-	NP	A-2-4(0)	SM
	45-60	IIC*	134	7		98	97	94	91	89	81	29	23	16	10	7	-	NP	A-2-4(0)	SM
	28-35	IIB21t	-	-	-	-	-	-	-	100	99	51	42	28	16	12	20	4	A-4(3)	CL-ML
Okee loamy fine sand, fine loamy variant	55-60	IIC	139	6	100	93	91	86	83	80	72	31	25	15	6	4	-	NP	A-2-4(0)	SM
Green Lake Co., Wis.																				
Omega sand	20-40	C	-	-	-	-	-	-	-	100	99	12	-	-	-	-	-	NP	A-2-4(0)	SP-SM
Washburn Co., Wis.																				
Onamia sandy loam	16-22	B2	-	-	93	85	83	82	81	81	70	46	45	34	20	16	30	13	A-6(3)	SC
Chippewa Co., Wis.	29-42	C	-	-	94	77	70	55	45	32	13	4	3	2	1	1	-	NP	A-1-a(0)	GP
Onaway loam	15-20	B2*	115	14	-	97	96	94	93	91	86	58	54	45	33	27	34	18	A-6(8)	CL
Door Co., Wis.	30-54	C*	104	8	-	97	96	91	87	82	74	43	34	20	7	5	-	NP	A-4(2)	SM
	20-26	B'2t	-	-	-	-	-	-	-	100	94	50	45	37	28	24	24	9	A-4(3)	CL
	30-60	C	132	8	-	-	100	96	94	89	81	39	33	22	13	10	15	NP	A-4(1)	SM
Onaway loam	14-21	B2	-	-	-	98	97	96	96	94	89	46	42	32	23	18	26	12	A-6(3)	SC
Marinette Co., Wis.	35-45	C	-	-	-	98	96	92	89	84	74	30	24	14	8	5	-	NP	A-2-4(0)	SM

Soil name and sample location	Depth in inches	Horizon	Moisture-density		Mechanical Analysis												Liquid limit	Plasticity index	Classification	
			Maximum dry density (lbs. per cu. ft.)	Optimum moisture (percent)	Percentage passing sieve								Percent smaller than						AASHO (group index ratings in parenthesis)	Unified
					2"	1"	3/4"	3/8"	#4 4.7 mm.	#10 2.0 mm.	#40 .42 mm.	#200 .074 mm.	0.05 mm.	0.02 mm.	.005 mm.	.002 mm.				
Washburn sandy loam	3-7	B2hir	-	-	-	94	94	92	91	89	76	33	29	17	7	4	-	NP	A-2-4(0)	SM
Ashland Co., Wis.	19-29	A&Bx	-	-	-	98	98	96	92	88	69	27	24	13	5	3	-	NP	A-2-4(0)	SM
	29-42	IIBtx	-	-	-	99	97	94	91	86	69	22	18	10	4	2	-	NP	A-2-4(0)	SM
	61-72	IIIC2	126	9	100	98	97	94	91	88	69	21	17	9	4	2	-	NP	A-2-4(0)	SM
Washburn sandy loam	21-41	B	-	-	98	93	90	88	88	88	73	18	14	11	6	4	-	NP	A-2-4(0)	SM
Vilas Co., Wis.	41-60	C	-	-	100	94	92	87	83	77	63	14	12	8	4	2	-	NP	A-2-4(0)	SM
	19-39	B	-	-	100	93	91	86	85	85	67	14	12	8	5	4	-	NP	A-2-4(0)	SM
	39-60	C	-	-	-	-	-	-	-	100	81	18	14	11	6	7	-	NP	A-2-4(0)	SM
Washburn sandy loam	35-44	B2t	131	9	-	100	99	96	94	92	75	23	21	14	10	8	-	NP	A-2-4(0)	SM
Washburn Co., Wis.	44-56	C	131	8	-	100	99	98	95	93	74	24	22	12	8	6	-	NP	A-2-4(0)	SM
Waukegan silt loam	-	B2	-	-	-	-	-	-	-	100	90	77	-	-	25	-	32	10	A-4(8)	CL
St. Croix Co., Wis.																				
Waymor silt loam	9-19	B2t	-	-	-	-	-	-	-	100	98	75	73	66	51	42	43	24	A-7-6(19)	CL
Sheboygan Co., Wis.	29-60	C	125	11	-	-	100	98	96	93	89	71	66	50	29	21	25	10	A-6(7)	CL
	9-21	B2t	-	-	-	-	-	-	-	100	99	88	86	79	59	47	46	25	A-7-6(15)	CL
	31-60	C	124	11	-	100	99	97	96	93	89	69	64	49	29	20	24	10	A-4(7)	CL
Westville loam	30-44	B2	-	-	-	99	99	98	97	96	88	47	45	36	23	19	29	15	A-6(4)	SC
Green Co., Wis.	56-60	C	-	-	-	97	95	90	86	82	74	36	34	22	11	7	-	NP	A-4(0)	SM
	22-58	B2	-	-	-	96	92	85	79	73	60	34	32	27	19	16	43	19	A-2-7(2)	SC
	58-70	B3	-	-	100	97	96	90	84	77	58	19	17	13	9	7	-	NP	A-2-4(0)	SM
	24-40	B2*	111	17	-	-	-	-	-	100	94	64	62	52	37	31	35	17	A-6(9)	CL
	52-60	C*	132	8	-	92	90	85	80	77	68	38	36	22	9	7	-	NP	A-4(1)	SM
Whalan silt loam	32-38	B2	-	-	-	-	-	-	-	100	95	41	40	34	27	23	28	16	A-6(9)	SC
Green Co., Wis.	17-35	B2*	124	12	-	-	-	-	-	100	94	58	56	46	28	23	25	11	A-6(8)	CL
	15-29	B2	-	-	-	-	-	-	-	100	98	68	65	53	31	26	34	18	A-6(10)	CL
Whalan silt loam	20-30	IIB23t	-	-	100	86	82	80	79	79	72	46	41	28	18	16	30	14	A-6(3)	SC
St. Croix Co., Wis.	30-34	IIIB3t	-	-	-	-	-	-	-	100	89	62	61	56	38	28	39	21	A-6(10)	CL
Withee silt loam	18-30	B2	-	-	-	-	-	-	-	100	94	78	76	60	36	30	41	24	A-7-6(14)	CL
Marathon Co., Wis.	37-50	C	-	-	-	-	-	-	-	100	91	68	66	54	38	32	40	25	A-6(13)	CL
Withee silt loam	25-31	B2*	117	15	-	-	-	-	-	100	94	74	69	54	37	30	39	24	A-6(14)	CL
Wood Co., Wis.	37-50	C*	119	14	-	-	-	-	-	100	96	66	62	50	38	30	36	23	A-6(11)	CL
	23-30	B2	-	-	-	-	-	-	-	100	93	67	63	46	27	23	32	16	A-6(8)	CL
	37-60	C	-	-	-	-	-	-	-	100	91	63	60	50	36	32	39	25	A-6(12)	CL

A PRELIMINARY STUDY OF REQUIREMENTS FOR PLANT GROWTH ON SOILS FROM THE CRANDON PROJECT AREA

MINE WASTE RECLAMATION LTD.
565 MASSEY ROAD,
GUELPH, ONTARIO, N1H 6R1, CANADA.

JUNE 1982

SUBMITTED TO EXXON MINERALS COMPANY,
CRANDON PROJECT, RHINELANDER, WISCONSIN.

1.0 INTRODUCTION

A preliminary study of the requirements for plant growth on sub-soil material from the Crandon Project site was undertaken by Mine Waste Reclamation Ltd. The study was limited to one sample obtained during earlier geotechnical investigations. The main objective was to determine if Crandon Project area sub-soils contained any serious impediments to plant growth. The extremely limited nature of the investigation was recognized at the outset. Hence the result of the study was to be considered indicative only and not to provide a definitive answer.

2.0 MATERIALS AND METHODS

2.1 Sample source

The sub-soil sample used in the study was originally obtained from the Crandon Project area, test pit No. 22, adjacent to boring G41-P24 (Table 1). It had been studied by Golder Associates as part of the geotechnical investigation being undertaken by them.

- sample size

2.2 Sample treatment

At Guelph, the contents of each container were passed through an horticultural sieve to remove soil fractions larger than one inch diameter. Each container contents were then mixed to ensure uniformity. An approximately one-pound

sub-sample was taken from each container and submitted to the Soil Testing Laboratory, Ontario Ministry of Agriculture and Food for a standard agricultural soil test analysis (pH, total salts, phosphorous, potassium, calcium and magnesium).

It must be emphasized that such tests have been developed for normal agricultural soils in Ontario having well defined characteristics and that the analytical techniques used are not necessarily suited to sub-soil and non-soil materials lacking these characteristics. Hence the soil test data obtained must be interpreted with caution and considered indicative only, not absolute.

The two samples were placed in plastic containers 6" x 2.5" diameter. Selected rates of agricultural limestone were added and mixed in with the sample materials. The procedure was repeated for fertilizer addition. The agricultural limestone used was a dolomitic type, having a neutralizing value of 100 per cent and an agricultural index measuring 56. Commerical agricultural fertilizers were used, which had previously been ground in order to achieve a uniform distribution throughout the sample materials (Table 2).

The F2 fertilizer treatment is representative of a standard formula used by the Ontario Ministry of Transportation and

Communications when seeding after highway construction. The F3 fertilizer treatment was selected for use on the basis of experience gained by Mine Waste Reclamation Ltd. when seeding soils exposed during construction activity.

After mixing of agricultural limestone and fertilizer, the contents of each plastic container was equally split between three smaller plastic containers. Three seeds mixtures were used to assess plant growth on the non-amended and amended sub-soil samples; birdsfoot trefoil + creeping red fescue, crown-vetch + creeping red fescue, and an all-grass mixture (Table 2). Following seeding of the two legume-grass mixtures the sub-soil surfaces were dusted with commercial legume inoculant. All three seeds mixtures were then mulched using a proprietary straw-cotton-paper based mulch. Both birdsfoot trefoil and crown-vetch were seeded at rates equivalent to 22.4 kg/ha (20 lb/ac), creeping red fescue at 34 kg/ha (30 lb/ac) and the all-grass mixture at 90 kg/ha (80 lb/ac).

The containers were divided into three blocks, each block containing one container of each sample x every treatment combination. The three blocks were placed in an environmentally-controlled growth room having a 12-hr. photoperiod, ambient temperature 72 $\pm$ 5^{O}F, relative humidity 60 $\pm$ 5 per cent. Containers were watered once every 48 hours by watering from above with a conventional garden hose attachment.

At harvest time, (54 days after seeding), plant material was clipped at soil level, dried, weighed and an appropriate table constructed.

3.0 RESULTS

3.1 Soil Testing analysis

Both sub-soil samples exhibited a moderately acid reaction (Table 3). The total salts value for either sample was extremely low. According to soil test interpretative values developed for Ontario, the sub-soil(s) showed medium requirements for phosphorous fertilization with a phosphate requirement of 20 kg/ha P_2O_5 (18 lb/ac). Similarly interpreted, the sub-soil was low in potassium and would require fertilization with 80 kg/ha potash (71 lb/ac). The values obtained for magnesium and calcium indicated neither a deficiency or an excess present.

3.2 Plant growth

Acceptable levels of plant growth were obtained on the two sub-soil samples where birdsfoot trefoil-creeping red fescue and crownvetch-creeping red fescue were seeded; growth of the all grass seeds mixture was poor and it showed typical symptoms of nitrogen deficiency. There was no advantage to correcting the moderately acid reaction of the samples by adding agricultural limestone (Table 4).

Both legume-based seeds mixtures responded to the addition of fertilizer. However, no practically significant differences were obtained between the fertilizer treatment of 448 kg/ha 5-20-20 and that of 841 kg/ha 5-20-20 + 560 kg/ha 0-46-0. On sample 1 there was a response of the all grass seeds mixture between no fertilizer and the lesser fertilizer treatment (F2) but not between the no fertilizer and larger fertilizer application (F3).

There was a definite pattern across all treatments for plant growth to be less on sub-soil sample 2 than on sub-soil sample 1. No satisfactory explanation can be offered for this unexpected discrepancy; unexpected in the fact that both samples were sub-samples of one larger sample.

The greater dry weight of the birdsfoot trefoil-creeping red fescue mixture over that of the crownvetch-creeping red fescue mixture should not necessarily be taken as an indication of superiority of the former. The difference in plant dry weight is a reflection of the known lower seedling vigour and growth rate of crownvetch compared to that of birdsfoot trefoil. Crown-vetch seedlings were healthy and exhibited no symptoms of nutrient deficiency or disease on either sub-soil sample. However, there were fewer seedlings per replicate (visual observation) which again is reflective of the lower germination rate of crownvetch compared to that of birdsfoot trefoil.

The plant growth data obtained here should not be used solely as the basis for seeds mixture selection for subsequent field seedings. Other factors such as required end-use of areas seeded in the field must also be taken into consideration.

TABLE 1

Origin of Soil Sample

Sample Material:	Data from container label
	Crandon project area, test pit No. 22, adjacent to boring G41-P24.
	Collector's sample No. GT-1, sampled March 15, 1981.
Sample source:	Golder Associates, 3151 Wharton Way, Mississauga, Ont.
	Sub-samples from main container (2 x 5-gal. pails) collected by E. M. Watkin, Mine Waste Reclamation Ltd., December 29, 1981.
Sub-sample identification:	Two sub-samples were labelled 1 and 2 (random) and both used as separate samples in plant growth test.

TABLE 2

1. Treatment of Sample:

Main treatment: Agricultural limestone applied at 0 and 5.6 tonnes/ha (0 and 2.5 tons/acre).

Sub-treatments:

F1 No fertilizer added

F2 5-20-20 @ 448 kg/ha (400 lb/ac)

F3 5-20-20 @ 841 kg/ha (400 lb/ac) + 0-46-0 @ 560 kg/ha(500 lb/ac)

2. Species assay mixtures:

Seeds mixture:

S1 - Birdsfoot trefoil cv. Leo and Creeping red fescue cv. Reptans.

S2 - Crownvetch cv. Penngift and Creeping red fescue cv. Reptans.

S3 - Pickseed "low maintenance" mixture; a commercially produced blend of various grasses only and in which exact details of component species, varieties thereof, and proportions thereof, is not public information.

3. Replication:

Each sample x agricultural limestone x fertilizer treatment x species mixture combination was replicated three times.

4. Seeding date: February 2, 1982; Harvest date: March 30, 1982.

TABLE 2 (cont'd)

Treatment Number Identification

		Trefoil red fescue		Crownvetch red fescue		All grass mixture	
		Agricultural limestone t/ha*					
Sample No. 1	Fertilizer treatment	0	5.6	0	5.6	0	5.6
	F1	1-3	4-6	19-21	22-24	37-39	40-42
	F2	7-9	10-12	25-27	28-30	43-45	46-48
	F3	13-15	16-18	31-33	34-36	49-51	52-54
Sample No. 2	F1	55-57	58-60	73-75	76-78	91-93	94-96
	F2	61-63	64-66	79-81	82-84	97-99	100-102
	F3	67-69	70-72	85-87	88-90	103-105	106-108

*tonnes per hectare

TABLE 3

Soil Test Values for Sub Soil Samples from Crandon Project Site

	pH	Total Salts micromhos/cm	Phosphorous ppm	Potassium ppm	Magnesium ppm	Calcium ppm
Sample 1	6.2	100	12	60	144	725
Sample 2	5.9	100	12	50	139	700

Notes:

1. Analysis by Soil Testing Laboratory, Ontario Ministry of Agriculture and Food, Guelph, Canada.
2. pH by paste technique.
3. Available phosphorous by 0.5N sodium bicarbonate extraction.
4. Available potassium by 1N ammonium acetate extraction.
5. Total calcium and magnesium by A.A.

TABLE 4

Plant growth on soil sample from Test Pit No. 22 from proposed tailings area - Crandon Project
Average grams dry weight of three replicates after 56 days

Sample	Fertilizer treatment	Trefoil Red fescue		Crownvetch Red fescue		All grass mixture	
		Agricultural limestone t/ha*					
		0	5.6	0	5.6	0	5.6
Sample 1	F1 None	0.71	0.66	0.35	0.35	0.27	0.29
	F2 5-20-20 @ 448 kg/ha	0.88	0.86	0.48	0.49	0.36	0.36
	F3 5-20-20 @ 841 kg/ha +0-46-0 @ 560 kg/ha	0.84	0.88	0.44	0.39	0.27	0.31
	Ave.	0.81	0.80	0.42	0.41	0.30	0.32
Sample 2	F1 None	0.42	0.50	0.23	0.24	0.24	0.29
	F2 5-20-20 @ 448 kg/ha	0.72	0.85	0.32	0.33	0.26	0.32
	F3 5-20-20 @ 841 kg/ha +0-46-0 @ 560 kg/ha	0.93	0.84	0.35	0.31	0.28	0.20
	Ave.	0.69	0.73	0.30	0.29	0.26	0.27

*tonnes per hectare

WASTE DISPOSAL FACILITY RECLAMATION CAP

CRANDON PROJECT
WATER BALANCE ANALYSIS

EXXON MINERALS COMPANY
RHINELANDER, WISCONSIN

NOVEMBER 1982

OWEN AYRES & ASSOC INC

Waste Disposal Facility Reclamation Cap

Crandon Project

Water Balance Analysis

Prepared for

Exxon Minerals Company

By

OWEN AYRES & ASSOCIATES INC

November 1982

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Executive Summary

Waste Disposal Facility Reclamation Cap

Crandon Project

Water Balance Analysis

EXECUTIVE SUMMARY

Water balance of the three alternate cover systems was analyzed for the Crandon Project's Waste Disposal Facility Reclamation Cap. The water balance calculations for these cover systems were based on a black spruce forest as the reclaimed surface condition with various subsurface layering alternatives. The water balance method in combination with Darcy flow calculations provided a monthly distribution of total precipitation into storage and outflow components which include: overland surface water runoff, evapotranspiration, soil moisture change, and percolation from the cover system. The effect of dry, normal, and wet years on evapotranspiration and percolation quantities was analyzed for each of the three alternatives.

Alternative 1, a two layered cover system, consisted of a 914 mm (36 inch) till layer underlain by a 152 mm (6 inch) bentonite modified soil layer. The bentonite modified soil layer limited percolation to the underlying tailings deposit. Based on the yearly variation in precipitation, the total yearly percolation to the tailings was estimated at 15.2 to 112.8 mm (0.60 to 4.44 inches), with a normal year at 39.6 mm (1.56 inches). This system resulted in periods of saturated soil conditions during wet years and periods of minimal soil moisture during dry years. This wide fluctuation in soil moisture condition could adversely affect development of the black spruce forest cover.

Alternatives 2 and 3 were both three layered cover systems with an additional 203 mm (8 inch) processed drainage layer immediately above the bentonite modified soil layer. Alternative 3 consisted of a 711 mm (28 inch) upper till layer instead of the 914 mm (36 inch) till layer used in Alternative 2. The inclusion of the drainage layer for these alternatives greatly reduced the vertical percolation into the tailings deposit which ranged from 15.2 to 18.8 mm (0.60 to 0.74 inches) for the extreme wet and dry annual conditions. These alternatives also eliminate the saturated soil condition which occurs during wet years. Alternative 2 provided the greatest depth for storage of soil moisture which is beneficial for the black spruce forest cover during dry years.

Alternative 2 cover system is recommended as the best design. This system reduced percolation to the tailings, provided more storage of soil moisture for the black spruce forest than Alternative 3, eliminated potential difficulties resulting from saturated soil conditions, and had a larger buffer zone which decreases the probability of the black spruce root system penetrating the drainage or bentonite modified soil layers.

Main Report

Waste Disposal Facility Reclamation Cap

Crandon Project

Water Balance Analysis

MAIN REPORT

WASTE DISPOSAL FACILITY RECLAMATION CAP
WATER BALANCE ANALYSIS

1.0 Introduction

This report includes a discussion of the water balance flow calculations required to analyze various cover systems for the Crandon Project tailings ponds. The project site is located approximately 10 km (6.2 miles) south of Crandon, Wisconsin and includes a proposed four cell tailings pond for disposal of fine-grained tailings generated from the mining of the ore body (Golder Associates, 1982). Three cover systems were investigated for an average, dry, and wet year to determine their effectiveness under a reforested condition. The advantages of the alternate systems were determined by the quantity of percolation reaching the tailings deposit and the systems' effect on moisture availability to the forest in the upper zone.

2.0 Water Balance Procedure

2.1 General

The water balance method was used to provide a monthly accounting of water movement within the alternate cover systems. This methodology is applied to a control volume situation where inflow less outflow provides a resultant storage change. For this application, inflow to the system is represented by monthly precipitation; outflow is expressed as overland runoff, evapotranspiration, and percolation from the system; and the change in storage is the change to the system soil moisture content. The cover system will be composed of an upper layer of till varying in depth from 711 to 914 mm (28 to 36 inches) and a 152 mm (6 inch) lower layer of bentonite modified soil. The upper layer will act as the storage area for available soil moisture with the lower layer functioning as a confining cap which limits percolation to the underlying tailings. Two of the alternatives include a drainage layer which divides the till and bentonite modified soil layers. This drainage layer will be 203 mm (8 inches) thick with a processed gradation varying from a No. 40 U.S. standard sieve size up to 50.8 mm (2.0 inches) grain size. The hydraulic conductivities of the varying layers are shown below (Golder Associates, 1981, 1982).

Material	K (m/s)
Till layer	1×10^{-6}
Drainage layer	5×10^{-3}
Bentonite modified soil layer	5×10^{-10}
Tailings deposit	5×10^{-8}

The data presented above indicate a differential hydraulic conductivity between the bentonite modified soil layer and underlying tailings. The delivery rate of water from the bentonite modified soil layer is less than the capacity of the tailings deposit to transmit percolation; therefore, the tailings will not be saturated at this interface and will not affect the water balance within the cover system. The control volume limits for this investigation were from the ground surface to the bottom of the confining bentonite modified soil layer.

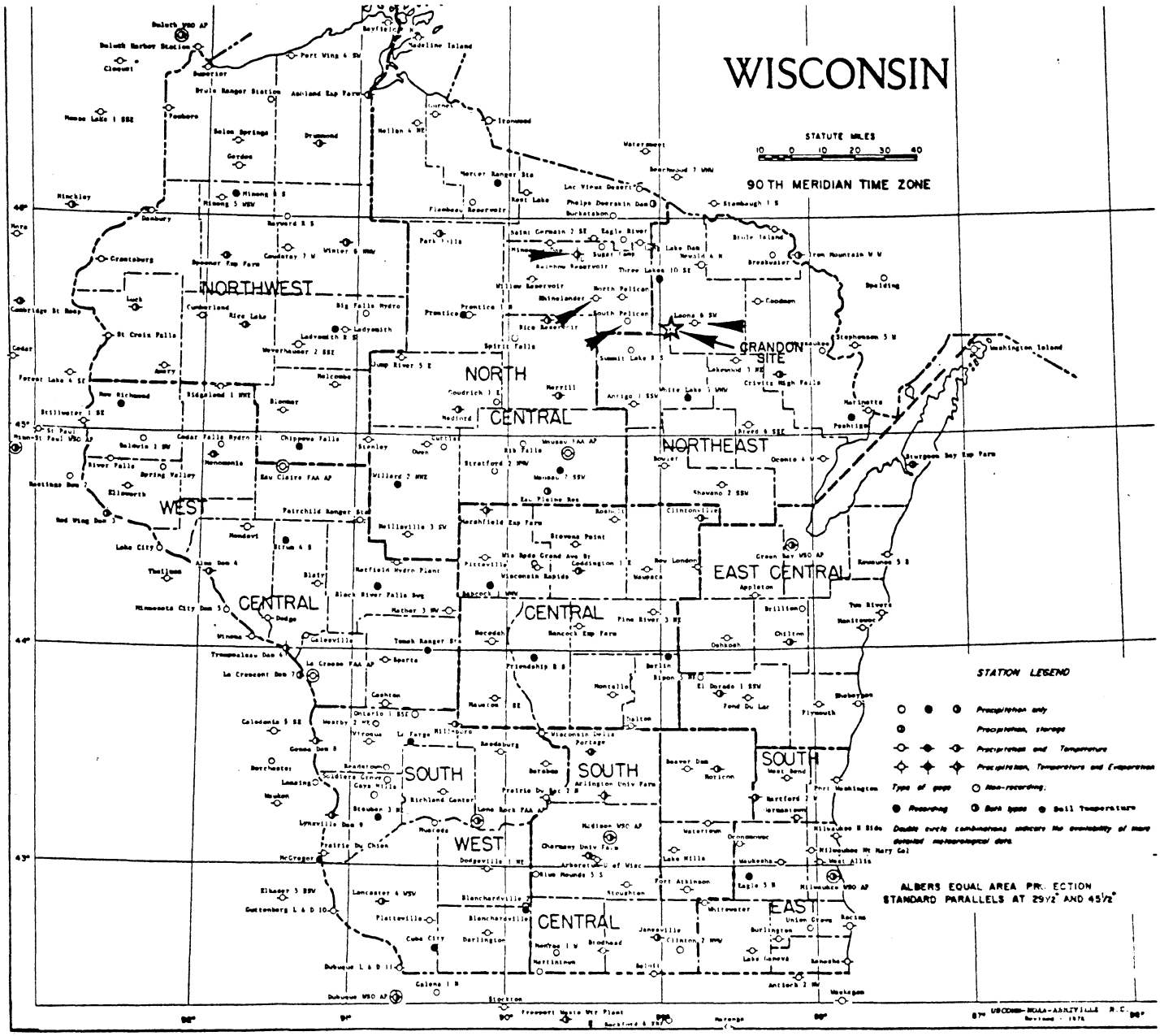
It was assumed in the analysis that a capillary break will be provided below the bentonite modified soil layer. This break will prevent the development of capillary pore pressures which could draw water from the bentonite modified soil layer into the underlying tailings deposit. The moisture content of the bentonite modified soil layer has an effect on air (oxygen) diffusion into the tailings. Saturation of this layer provides a barrier for oxygen diffusion, with a decrease in the degree of saturation resulting in an exponential increase of oxygen diffusion. Consideration of oxygen diffusion through the bentonite modified soil is not a part of this analysis.

2.2 Precipitation

Precipitation records from four recording stations (Figure 1) were compared for the period November 1977 to December 1978 (Dames and Moore, 1981 and NOAA, 1940-1981) to determine the variations between these recording sites. Three of the gages are National Oceanic and Atmospheric

FIGURE 1

Climatological Stations*



* Extracted from NOAA, 1940-1981.

Administration (NOAA) gages at Rhinelander, Laona, and South Pelican with the fourth being records from Nicolet College in Rhinelander, Wisconsin. Comparison of these gages shows good agreement between the three NOAA gages, however there was a significant difference between records at the Rhinelander NOAA station and the Nicolet College data. The amount of rainfall was generally higher at the Rhinelander NOAA station during this specific period. The Rhinelander NOAA station was used in the analysis for precipitation data since it is a supported NOAA station, provides the highest rainfall for this period, and has published long-term averages. These monthly averages, for 1941-1970, are tabulated in Table 1.

2.3 Overland Runoff

Overland surface water runoff for these calculations is expressed as a percentage of the monthly precipitation. Published data for overland runoff vary depending on the soil type and surface slope. Previous studies have indicated a runoff percentage of 5 percent for sandy soils at 2 percent slope (Journal of the Environmental Engineering Division, 1977). The Soil Conservation Service (SCS) runoff curve number procedure for estimating runoff quantities was used to verify this percentage. Based on soil data provided for Forest County, Wisconsin (USDA, SCS), the surface soils at the project site are divided evenly between hydrologic group B and C soils. Assuming a forested condition, the resultant runoff curve numbers vary from 67 to 83 for Antecedent Moisture Conditions (AMC) II and III, respectively.

TABLE 1
Rhinelander Average Monthly Precipitation

Month	Average Precipitation* (mm)	(inches)
January	25.4	1.00
February	18.8	0.74
March	38.1	1.50
April	56.1	2.21
May	89.7	3.53
June	112.3	4.42
July	93.2	3.67
August	98.8	3.89
September	96.5	3.80
October	58.2	2.29
November	46.7	1.84
December	29.0	1.14
YEARLY	762.8	30.03

* Averages for 1941-1970.

These runoff curve numbers were used to compute the daily runoff from May 1978 to October 1981 using daily precipitation records from NOAA (NOAA, 1978-1981) and procedures outlined by the SCS (USDA, SCS, 1969). A plot of the percent runoff versus the total monthly precipitation indicates runoff coefficients of up to 0.02 and 0.05 for AMC II and III respectively. These data in combination with published data (Journal of the Environmental Engineering Division, 1977) yielded a monthly coefficient of 0.05 during months when the ground surface is unfrozen. Given frozen ground conditions, it was assumed that all precipitation becomes runoff. To determine the duration of frozen conditions, soil temperature records for 1965-1981 (NOAA, 1940-1981) at Rainbow Reservoir (Figure 1) were reviewed for different depths. Based on this review, it was assumed the soil is frozen from approximately November 20 through March 10. These coefficients were used to determine the amount of runoff in the water balance calculations.

2.4 Evapotranspiration

Evapotranspiration is a two component parameter which combines evaporation, due to temperature and air movement, with transpiration, which is dependent on the type of vegetation and its degree of activity. When considering this parameter, most literature is based on the potential evapotranspiration (PET) that will occur given adequate moisture availability. Procedures to compute PET vary from considering only climatic parameters to including consumptive use coefficients for different types of vegetation. It was assumed that the reclamation of the tailings pond will include reforestation with predominantly black spruce (may include mixture with

other species). Black spruce will be used due to its shallow root system which will not penetrate the bentonite modified soil cap. A study by the Forestry Science Laboratory of the U.S. Forest Service (Verry, in press) on black spruce in Minnesota shows that the PET computed by the Thornthwaite method (Thornthwaite, 1955) provides a good prediction of actual evapotranspiration measured at the Marcell Experimental Forest. Additional research at the Marcell Experimental Forest (Verry, 1976) indicated that 20 to 30 percent of all precipitation will be intercepted by a black spruce forest and will never reach the ground. This interception, which is a portion of the evapotranspiration value, is not available for plant uptake or overland flow.

The Thornthwaite method of determining PET was adopted for these water balance calculations. This procedure requires a monthly adjustment factor for latitude (Thornthwaite, 1955) and the mean monthly temperature. Monthly average temperatures for the Rhinelander NOAA gage were selected after reviewing data for NOAA stations at Rhinelander and Laona, Nicolet College and monitoring station 1 near the project site (Dames and Moore, 1981) for the period April 1977 to December 1978. The long-term averages for the Rhinelander NOAA gage for 1941-1970 are presented below:

Month	Temperatures (°C)
January	-10.9
February	- 9.4
March	- 3.4
April	5.6
May	12.2
June	23.1
July	19.8
August	18.8
September	13.8
October	8.6
November	- 0.6
December	- 7.9

2.5 Soil Moisture

Changes in soil moisture in the two separate layers were considered in the water balance calculation. In the upper layer (till layer) the soil moisture is subject to evapotranspiration through the black spruce forest. Calculations in the lower layer (bentonite modified soil layer) were based on the assumption that the overlying root system will not penetrate this layer and therefore the layer does not provide moisture for evapotranspiration. Parameters required to define soil moisture changes in each layer include the initial soil moisture and available soil moisture for percolation and/or evapotranspiration, which are determined by the materials porosity and field capacity. Field capacity is the amount of water remaining in the soil after gravitational water has drained (Linsley, 1958).

The upper till layer can be classified as a loamy sand based on previous grain size analyses (Golder Associates, 1981) and the U.S. Department of Agriculture textural classification triangle (Chow, 1964). The porosity for in-place conditions was calculated as 0.37 for the till layer, which is well within accepted limits for this material. The field capacity of the material was estimated as 10 percent by dry weight, based on geotechnical investigations (Golder Associates, 1981). The available moisture for plant growth is the difference between the soils field capacity and the wilt point moisture. The wilt point moisture represents the moisture bound by tension to the soil particles by a force equal to the osmotic pressure exerted by plants (Linsley, 1958). At this pressure (15 atmospheres) (Linsley, 1958) the plants can no longer extract moisture from

the soil. The available moisture for loamy sand is 0.08 mm per mm of soil depth (Walsh, 1972; Linsley, 1958; and U.S. Forest Service, 1969).

Similar calculations for the lower bentonite modified soil layer provide the saturation, field capacity, and available moisture given a porosity of 0.25 for this layer. The moisture holding capabilities of these two layers are summarized below:

Layer	Saturation (mm/mm)	Field Capacity (mm/mm)	Available Moisture (mm/mm)
Till	0.37	.17	.08
Bentonite modified soil	0.25	.18	*

* Available moisture not considered since roots will not penetrate bentonite modified soil layer.

2.6 Water Balance Calculations

The water balance methodology, using previously described data, was computerized to facilitate the analysis of various alternatives. The computer model was initially used for an average year without a confining bentonite modified soil layer to simulate a normal year under existing conditions. This analysis verified the selected input parameters, as previously defined. The following paragraphs detail the operation of this program and its predictions for a normal year.

For this computation the till layer was assumed to be 914 mm (36 inches) in depth which will provide the only storage zone for moisture changes. For this depth, the soil profile has a saturation content of 338.2 mm (13.32 inches), field capacity of 155.4 mm (6.12 inches), and available moisture of 73.1 mm (2.88 inches), giving a wilt point of 82.3 mm (3.24 inches). The program operations deal only with available moisture and disregards wilt point moisture since it is unavailable to evapotranspiration. The soil properties were adjusted to provide a resultant saturation of 255.9 mm (10.08 inches) and field capacity of 73.1 mm (2.88 inches). Table 2 shows the results of a normal year given these parameters. All calculations and output are represented in inches of water for the water balance program.

The average precipitation (Row 1) is from the long-term records for Rhinelander with surface water runoff (Row 2) being computed as 5 percent of the monthly precipitation during unfrozen conditions. Row 3 is the difference between precipitation and runoff and reflects the moisture available for infiltration into the soil profile. Row 4 is based on the water balance at the end of the previous month and cannot exceed the soil field capacity under these conditions. Total available moisture (Row 5) is the summation of Rows 3 and 4 and indicates moisture availability for evapotranspiration and percolation. Row 6, the potential evapotranspiration (PET) was determined by the Thornthwaite method. The actual evapotranspiration for this analysis equals the lesser of Rows 5 and 6. The remaining available moisture for percolation and soil moisture storage is given in Row 8 and is calculated as Row 5 less Row 7. Negative values in Row 9 indicate that the liner moisture balance is not used in

TABLE 2
Water Balance Analysis
Normal Conditions

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	5.61	12.22	23.06	19.83	18.83	13.83	8.61	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 1 - CRANDON SITE

	PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
ROW 1	AVERAGE PRECIPITATION	1.00	.74	1.50	2.21	3.53	4.42	3.67	3.89	3.80	2.29	1.84	1.14	30.03
ROW 2	SURFACE RUNOFF	1.00	.74	.45	.11	.18	.22	.18	.19	.19	.11	.55	1.14	5.07
ROW 3	MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.05	2.10	3.35	4.20	3.49	3.70	3.61	2.18	1.29	.00	24.96
ROW 4	INITIAL SOIL MOISTURE	2.88	2.88	2.88	2.88	2.88	2.88	1.13	.00	.00	.89	1.60	2.88	
ROW 5	TOTAL AVAILABLE MOISTURE	2.88	2.88	3.93	4.98	6.23	7.08	4.62	3.70	3.61	3.07	2.89	2.88	
ROW 6	POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72
ROW 7	ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	4.62	3.70	2.72	1.46	.00	.00	22.48
ROW 8	REMAINING AVAILABLE MOISTURE	2.88	2.88	3.93	3.88	3.30	1.13	.00	.00	.89	1.60	2.89	2.88	
ROW 9	LINER MOISTURE	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	
ROW 10	PERCOLATION	.00	.00	1.05	1.00	.42	.00	.00	.00	.00	.00	.01	.00	2.47

this example. The percolation out of the root zone is given in Row 10 as Row 8 less the soils field capacity. When Row 8 is less than field capacity, no percolation can occur and the value becomes zero. The difference between Rows 8 and 10 provides the initial soil moisture (Row 4) for the following month.

Given the average precipitation of 762.8 mm (30.03 inches), the corresponding runoff was computed as 128.8 mm (5.07 inches), actual evapotranspiration of 571.0 mm (22.48 inches), and percolation of 62.8 mm (2.47 inches). Soil moisture is at field capacity from December through June (Table 2). In July the evapotranspiration exceeds the available infiltration and plant uptake begins to decrease the available soil moisture. During August and September, the soil moisture is depleted (wilt point moisture remains in the soil profile) and the only moisture available for evapotranspiration is that portion infiltrating into the upper layer after surface runoff and plant interception. The portion of the rainfall intercepted by the black spruce will be returned directly to the atmosphere. These data were compared to the results of a water balance analysis by the U.S. Geological Survey (USGS) (Atlas HA-321, 1968) using rainfall data and actual surface water runoff records for this portion of Wisconsin and the results are presented below:

Parameter	Water Balance Computations (mm)	(inches)	USGS Study (mm)	(inches)
Average precipitation	762.8	30.03	762.0	30.0
Runoff	128.8	5.07	203.2*	8.0*
Evapotranspiration	571.0	22.48	558.8	22.0
Percolation	62.8	2.47	*	*

* USGS study combines runoff and percolation.

Values presented above show a good comparison between the natural condition water balance and the USGS investigations. The computed evapotranspiration of 571.0 mm (22.48 inches) is comparable to reported values for black spruce (Verry, in press; and Verry, communications, 1982). Based on this verification, the water balance model was used to analyze three alternate cover systems for the proposed tailings pond.

3.0 Results and Discussion

3.1 Alternative No. 1

The Alternative 1 cover system consists of 914 mm (36 inches) of till over 152 mm (6 inches) of bentonite modified soil mixture. The till surface and layer interface will both be constructed at a 2 percent grade. The low hydraulic conductivity of the bentonite modified soil layer will inhibit percolation, resulting in moisture contents in the upper layer in excess of the material's field capacity. The maximum rate of perolation through the bentonite modified soil layer, assuming saturation to the soil surface, is 3.5×10^{-9} m^3/s per m^2 or 9.4 mm/month (0.37 inch per month). Since the bentonite modified soil layer is not affected by plant uptake, the moisture content in this zone will not go below the field capacity. During months when all available moisture in the upper zone is lost through evapotranspiration, the bentonite modified soil layer will continue to provide a minimum percolation to the tailings resulting in a desaturation of this layer. With a gradient of unity, the minimum percolation from this layer is 5×10^{-10} m^3/sec per m^2 or 1.3 mm/month (.05 inch per month). The model development assumes that this percolation will be replaced to the liner layer by moisture in the upper layer if its content is above field capacity or by moisture which otherwise would be a portion of evapotranspiration. In the event that moisture is not available from these sources, the liner layer moisture content decreases as desaturation of the upper fringe occurs. Calculations assume that this fringe will be resaturated in following months

due to capillary potential of this layer. These assumptions result in a conservative estimate of the total percolation quantity. In actuality, the tension in the till layer, once moisture content drops below field capacity, will retain the moisture in the upper regions of the layer for evapotranspiration, resulting in desaturation of the bentonite modified soil layer to its field capacity. Table 3 displays the results of the water balance during an average year for the Alternative 1 cover system.

Table 3 shows that the percolation is decreased to 39.6 mm (1.56 inches) and evapotranspiration is increased to 594.1 mm (23.39 inches) given the bentonite modified soil layer constraints. The soil moisture now exceeds the field capacity for April-June, providing a storage of moisture for evapotranspiration to follow in July and August. The soil moisture only reaches the wilt point for the month of August under these conditions. The percolation rates for March-May reach the maximum allowable, given the cap constraints. However, a review of the initial soil moisture for these months clearly shows the profile is not fully saturated to 255.9 mm (10.08 inches). The maximum percolation rate assumes a fully saturated condition; therefore, the overall percolation for these months is conservatively high. Calculation of the percolation rate could be refined by determining the percent of total depth saturated for each month and revising the corresponding Darcy flow calculations for that period. This refinement was not made to the analysis procedures.

TABLE 3
Water Balance Analysis
Alternative 1 - Normal Year

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	5.61	12.22	23.06	19.83	18.83	13.83	8.61	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 2 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.00	.74	1.50	2.21	3.53	4.42	3.67	3.89	3.80	2.29	1.84	1.14	30.03
SURFACE RUNOFF	1.00	.74	.45	.11	.18	.22	.18	.19	.19	.11	.55	1.14	5.07
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.05	2.10	3.35	4.20	3.49	3.70	3.61	2.18	1.29	.00	24.96
INITIAL SOIL MOISTURE	2.88	2.88	2.88	3.36	3.99	4.04	2.29	.68	.00	.89	1.60	2.89	
TOTAL AVAILABLE MOISTURE	2.88	2.88	3.93	5.46	7.34	8.23	5.78	4.37	3.61	3.07	2.89	2.89	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.89	5.05	4.32	2.67	1.41	.00	.00	23.39
REMAINING AVAILABLE MOISTURE	2.88	2.88	3.73	4.36	4.41	2.29	.68	.00	.89	1.60	2.89	2.89	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	1.40	
PERCOLATION	.05	.05	.37	.37	.37	.05	.05	.05	.05	.05	.05	.05	1.56

3.1.1 Yearly Variations

To define the extreme yearly variations at the Rhinelander recording station, the analysis considers 42 years of precipitation data (NOAA, 1940-1981). During this time frame the driest year was 1976 with 447.8 mm (17.63 inches) of precipitation. The greatest amount of rainfall, 1,014 mm (39.92 inches), occurred in 1978 and was preceded by 990.9 mm (39.01 inches) in 1977. A water balance analysis for Alternative 1 includes the actual monthly precipitation and temperature records for these extreme years.

Computations for the dry year of 1976 indicate that the greatest impact will be on the following season. The end of December soil moisture for 1976 is used as the initial January moisture for a normal year following 1976. Tables 4 and 5 provide the data from these analyses.

Water balance for the dry year yields a significant drop in the amount of evapotranspiration occurring and yet provides the same overall quantity of percolation. The percolation is again a conservative estimate since saturation does not occur during March-May when the maximum value is reached. Evapotranspiration is calculated as 360.4 mm (14.19 inches) for 1976 with interception being approximately 101.6 mm (4.0 inches) of this total value. The remaining soil moisture in December of 5.1 mm (0.2 inch) is carried to the following year as the initial soil moisture in Table 5. Table 5 shows that the next year has essentially zero soil moisture in January which decreases the actual evapotranspiration to

TABLE 4
Water Balance Analysis
Alternative 1 - Dry Year 1976

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	6.89	11.06	18.67	20.72	18.67	12.94	4.50	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.50	2.78	4.82	5.43	4.51	2.65	.81	.00	.00	22.49

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 2 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.70	.98	2.54	2.63	1.55	1.71	1.53	3.43	.40	.43	.28	.45	17.63
SURFACE RUNOFF	1.70	.98	.76	.13	.08	.09	.08	.17	.02	.02	.08	.45	4.56
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.78	2.50	1.47	1.62	1.45	3.26	.38	.41	.20	.00	13.07
INITIAL SOIL MOISTURE	2.88	2.88	2.88	4.09	4.71	3.04	.00	.00	.00	.00	.00	.20	
TOTAL AVAILABLE MOISTURE	2.88	2.88	4.66	6.59	6.19	4.66	1.45	3.26	.38	.41	.20	.20	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.50	2.78	4.82	5.43	4.51	2.65	.81	.00	.00	22.49
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	1.50	2.78	4.61	1.40	3.21	.33	.36	.00	.00	14.19
REMAINING AVAILABLE MOISTURE	2.88	2.88	4.46	5.08	3.41	.00	.00	.00	.00	.00	.20	.20	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	1.40	
PERCOLATION	.05	.05	.37	.37	.37	.05	.05	.05	.05	.05	.05	.05	1.56

TABLE 5
Water Balance Analysis
Alternative 1 - Normal Year Following 1976

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	5.61	12.22	23.06	19.83	18.83	13.83	8.61	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 2 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.00	.74	1.50	2.21	3.53	4.42	3.67	3.89	3.80	2.29	1.84	1.14	30.03
SURFACE RUNOFF	1.00	.74	.45	.11	.18	.22	.18	.19	.19	.11	.55	1.14	5.07
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.05	2.10	3.35	4.20	3.49	3.70	3.61	2.18	1.29	.00	24.96
INITIAL SOIL MOISTURE	.20	.20	.20	1.25	2.25	2.67	.92	.00	.00	.89	1.60	2.89	
TOTAL AVAILABLE MOISTURE	.20	.20	1.25	3.35	5.60	6.86	4.41	3.70	3.61	3.07	2.89	2.89	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	.80	2.89	5.89	4.36	3.65	2.67	1.41	.00	.00	21.67
REMAINING AVAILABLE MOISTURE	.20	.20	1.25	2.25	2.67	.92	.00	.00	.89	1.60	2.89	2.89	
LINER MOISTURE	1.35	1.30	1.25	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	1.40	
PERCOLATION	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.60

550.4 mm (21.67 inches) and percolation to a minimum value of 15.2 mm (0.6 inch). By the end of the year soil moisture has increased to its normal level for the following season.

To analyze the full impact of the wet year, it is necessary to consider the impact of 1977 on the initial moisture level for 1978. Tables 6 and 7 summarize the results of the wet year computations. Assuming a normal year prior to 1977, the results of the water balance are indicated in Table 6, which provides a total percolation for this year of 88.4 mm (3.48 inches). The initial soil moisture for January 1978 is the remaining available moisture less the maximum percolation for December 1977, or 213.1 mm (8.39 inches) as shown in Table 7. The wet year of 1978 provides 306.3 mm (12.06 inches) of runoff, 561.3 mm (22.10 inches) of actual evapotranspiration, and a maximum percolation of 112.8 mm (4.44 inches). Review of the initial soil moisture for August-December shows 4 months of total saturation (255.9 mm [10.08 inches]) which significantly increases the total runoff quantity. The remaining available moisture in December is also near the saturation level which indicates that the impact of this event will also carry to the following year.

Table 8 provides an overall review of the water balance calculations for Alternative 1 given various yearly conditions.

TABLE 6
Water Balance Analysis
Alternative 1 - Wet Year 1977

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.83	7.83	17.33	16.50	20.89	15.78	13.33	7.11	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.14	1.66	4.36	4.19	5.44	3.74	2.68	1.24	.00	.00	23.45

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 2 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	.58	.44	3.28	3.81	2.87	4.21	2.97	7.01	5.41	2.49	4.09	1.85	39.01
SURFACE RUNOFF	.58	.44	.98	.19	.14	.21	.15	.35	.27	.12	1.23	1.85	6.52
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	2.30	3.62	2.73	4.00	2.82	6.66	5.14	2.37	2.86	.00	32.49
INITIAL SOIL MOISTURE	2.88	2.88	2.88	4.47	6.06	4.05	3.49	.87	3.42	5.51	6.27	8.76	
TOTAL AVAILABLE MOISTURE	2.88	2.88	5.18	8.09	8.78	8.05	6.31	7.53	8.56	7.88	9.13	8.76	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.14	1.66	4.36	4.19	5.44	3.74	2.68	1.24	.00	.00	23.45
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.14	1.66	4.36	4.19	5.39	3.74	2.68	1.24	.00	.00	23.40
REMAINING AVAILABLE MOISTURE	2.88	2.88	4.84	6.43	4.42	3.86	.87	3.79	5.88	6.64	9.13	8.76	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	
PERCOLATION	.05	.05	.37	.37	.37	.37	.05	.37	.37	.37	.37	.37	3.48

TABLE 7
Water Balance Analysis
Alternative 1 - Wet Year 1978

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	3.22	13.28	16.33	18.11	18.89	15.22	7.06	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	.71	3.39	4.23	4.75	4.58	3.15	1.30	.00	.00	22.10

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 2 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	.55	.38	.15	2.75	4.66	4.48	7.70	9.15	5.70	1.42	1.54	1.44	39.92
SURFACE RUNOFF	.55	.38	.05	.14	.23	.22	1.75	4.20	2.18	.07	.85	1.44	12.06
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	.11	2.61	4.43	4.26	5.95	4.95	3.52	1.35	.69	.00	27.86
INITIAL SOIL MOISTURE	8.39	8.02	7.65	7.38	8.92	9.59	9.25	10.08	10.08	10.08	9.76	10.08	
TOTAL AVAILABLE MOISTURE	8.39	8.02	7.75	10.00	13.35	13.85	15.20	15.03	13.60	11.43	10.45	10.08	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	.71	3.39	4.23	4.75	4.58	3.15	1.30	.00	.00	22.10
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	.71	3.39	4.23	4.75	4.58	3.15	1.30	.00	.00	22.10
REMAINING AVAILABLE MOISTURE	8.39	8.02	7.75	9.29	9.96	9.62	10.45	10.45	10.45	10.13	10.45	10.08	
LINER MOISTURE	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	
PERCOLATION	.37	.37	.37	.37	.37	.37	.37	.37	.37	.37	.37	.37	4.44

TABLE 8
Water Balance Summary
Alternative 1

Yearly Condition	Precipitation (mm)	Precipitation (inches)	Runoff (mm)	Runoff (inches)	Evapotranspiration (mm)	Evapotranspiration (inches)	Percolation (mm)	Percolation (inches)
Normal Year	762.8	30.03	128.8	5.07	594.1	23.39	39.6	1.56
Dry Year 1976	447.8	17.63	115.8	4.56	360.4	14.19	39.6	1.56
Normal Year Following 1976	762.8	30.03	128.8	5.07	550.4	21.67	15.2	0.60
Wet Year 1977	990.9	39.01	165.6	6.52	594.4	23.40	88.4	3.48
Wet Year 1978	1014.0	39.92	306.3	12.06	561.3	22.10	112.8	4.44

The data summarized in Table 8 provide information on the impact of yearly fluctuations on the overall water balance. The dry year of 1976 resulted in a significant drop in evapotranspiration and a decrease in the soil moisture content. This soil moisture depletion carries to the following year where the percolation is decreased as soil moisture is replaced with evapotranspiration back to its normal range. The wet year of 1977 results in increased percolation and moisture content, yielding even higher percolation in 1978. Actual runoff does not vary significantly until soil saturation occurs in wet year 1978. All percolation values given in Table 8 are conservatively high based on the limiting assumptions as previously discussed. This analysis does not address whether a saturated or wilt point soil condition will be most detrimental to a black spruce forest.

3.2 Alternative No. 2

3.2.1 Drainage Layer Water Movement

The cover system designated as Alternative 2 is the same as the previous system except that a drainage layer is included between the till and bentonite modified soil layers. The overall cover system will function as a layered system with varying hydraulic conductivities, porosities, and moisture holding capabilities. Darcy flow calculations for the system indicate that the drainage layer cannot be fully saturated due to the substantial difference in hydraulic conductivities of the bentonite modified soil layer (5×10^{-10} m/s) and the drainage layer (5×10^{-3} m/s). Since the till-drainage layer interface is not saturated, the overlying till layer will function as an unsaturated media, independent of the two lower layers. Given a hydraulic gradient of one, the till layer can deliver up to 1.0×10^{-6} m^3/s per m^2 or 2,678 mm/month (105 inches per month) which is approximately equal to the maximum hourly infiltration rate expected (Chow, 1964). Based on these considerations, the delivery rate to the drainage layer will be limited to the moisture not used in the upper till layer for evapotranspiration or replenishing soil moisture. In principle, this system reverts to a natural condition as far as the water balance calculations are concerned for the upper 914 mm (36 inch) till layer.

Previous calculations shown in Table 5, for normal conditions, provide a maximum percolation rate of 26.7 mm (1.05 inches) for March. This yields a maximum delivery rate 1.029×10^{-8} m^3/s per m^2 to the drainage

layer for the normal year. Review of proposed grading plans for the cover system shows a maximum drainage layer length of approximately 550 m (1,805 feet). Conditions within the drainage layer and its interaction with the underlying bentonite modified soil layer were analyzed by dividing this system into 11 individual elements, each 50 m (164 feet) in length. Given a constant delivery rate, the inflow, outflow and corresponding saturation depth were calculated for each element to determine the steady-state outflow components perpendicular and parallel to the drainage-bentonite modified soil layer interface. Outflow, parallel to the interface, for one element becomes a portion of the combined inflow for the next element. For discussion purposes, the flow perpendicular to the layers will be referred to as vertical percolation, with horizontal percolation describing the parallel flow component within the drainage layer. A schematic representation of this procedure is presented in Figure 2.

Vertical outflow summation for all elements compared to the total horizontal outflow from the last element provides an estimate of the drainage layer efficiency. For these conditions with a 1 m (3.28 feet) wide strip, the total inflow, vertical and horizontal outflow, percent leakage to the bentonite modified soil and monthly percolation are summarized for two delivery rates in Table 9. The delivery rates shown are for the maximum anticipated moisture available to the drainage layer for a normal and wet year.

Figure 2
Drainage Layer
Flow Element Analysis

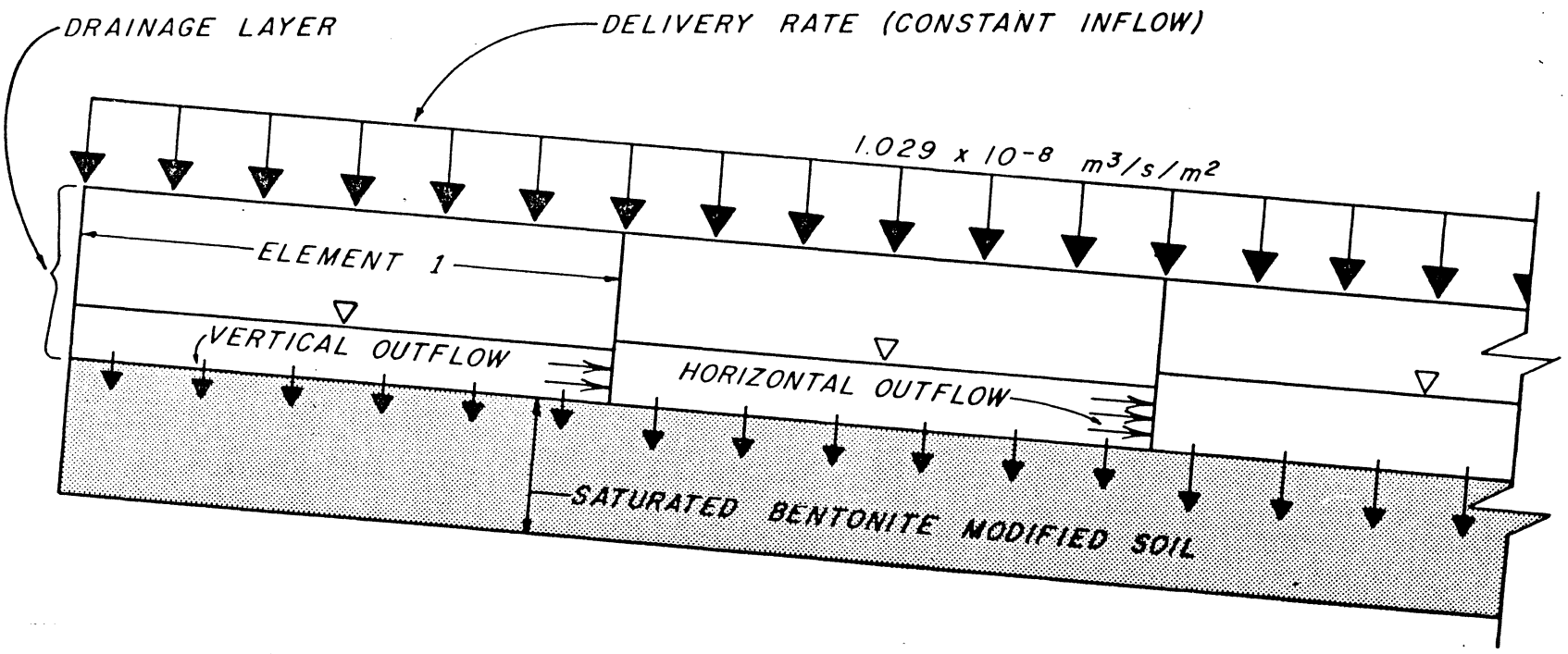


TABLE 9
Drainage Layer Efficiency

Components	Delivery Rates	
	Normal Year	Wet Year
	1.029×10^{-8} $m^3/s/m^2$ or 26.7 mm/month (1.05 in./month)	4.03×10^{-8} $m^3/s/m^2$ or 104.4 mm/month (4.11 in./month)
Total Inflow	5.66×10^{-6} m^3/s	22.11×10^{-6} m^3/s
Horizontal Outflow	5.33×10^{-6} m^3/s	21.62×10^{-6} m^3/s
Vertical Outflow	0.33×10^{-6} m^3/s	0.49×10^{-6} m^3/s
Percent Leakage	5.8%	2.2%
Vertical Percolation	1.6 mm/month (.06 in./month)	2.4 mm/month (.09 in./month)

As the delivery rate increases the percent leakage will decrease (Table 9). The high efficiency of this drainage layer is due to seven orders of magnitude difference in the hydraulic conductivity of the two layers. For the maximum month in a wet year (104.4 mm/month delivery rate), the full depth of the drainage layer will be saturated within the last computation element. The minimum vertical percolation from the bentonite modified soil layer was previously calculated as 1.3 mm/month (.05 inches per month). Comparing this value with the maximum value as shown in Table 9 provides a narrow range for the vertical percolation component. A plot of the delivery rate and vertical percolation component results in a linear relationship which is applied to the combined percolation from the water balance computation to determine the individual horizontal and vertical components.

Computations by the modified Wong Model (University of Wisconsin-Extension, 1981) for percent leakage into a liner provide a verification of the element method discussed above. Results of the Wong Model indicate the time for the trailing edge of the percolation volume to exit the end of the drainage layer is shorter than the time required for vertical percolation into the bentonite modified soil layer. Comparisons of the percent leakage, shown below, provide verification of both methodologies. Values generated by the element method were utilized for further analyses.

Procedure	% Leakage for Max. Month Normal Year	Wet Year
Element Analysis	5.8	2.2
Wong Model	6.2	2.7

3.2.2 Water Balance Computations

Water balance computations for Alternative 2 assume that plant uptake is limited to the upper 914 mm (36 inch) till layer. The analysis does not limit vertical percolation through the upper layer due to the efficiency of the drainage layer. Minimum percolation from the bentonite modified soil layer is included and only has a vertical component. The water balance calculations were made without consideration of separate percolation components, which were determined manually as previously described. Tables 10-14 display the water balance results for Alternative 2 given these constraints.

Tables 10-14 show substantially greater total percolation than similar analyses for Alternative 1 because the drainage layer allows the till layer to stay at field capacity. The higher percolation shown in Table 11 for Dry Year 1976 versus Table 10 is a result of the placement of 1976 precipitation at the beginning of the year, prior to soil moisture content reduction. In general, Alternative 2 does not allow soil moisture increases above field capacity. This results in reduced evapotranspiration and increased percolation from values generated for Alternative 1.

The total percolation shown in Tables 10-14 is divided into its separate components, as previously described. The results of the separation are displayed in Table 15. The actual vertical percolation to the tailings is greatly reduced from Alternative 1. This percolation shows a narrow range of values of 15.2 mm (0.60 inch) to a maximum of 18.8 mm (0.74 inch), due to the efficiency of the drainage layer.

TABLE 10
Water Balance Analysis
Alternative 2 - Normal Year

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUI	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	5.61	12.22	23.06	19.83	18.83	13.83	8.61	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 3 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.00	.74	1.50	2.21	3.53	4.42	3.67	3.89	3.80	2.29	1.84	1.14	30.03
SURFACE RUNOFF	1.00	.74	.45	.11	.18	.22	.18	.19	.19	.11	.55	1.14	5.07
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.05	2.10	3.35	4.20	3.49	3.70	3.61	2.18	1.29	.00	24.96
INITIAL SOIL MOISTURE	2.88	2.88	2.88	2.88	2.88	2.88	1.13	.00	.00	.89	1.60	2.89	
TOTAL AVAILABLE MOISTURE	2.88	2.88	3.93	4.98	6.23	7.08	4.62	3.70	3.61	3.07	2.89	2.89	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.89	4.57	3.65	2.67	1.41	.00	.00	22.23
REMAINING AVAILABLE MOISTURE	2.88	2.88	3.73	3.88	3.30	1.13	.00	.00	.89	1.60	2.89	2.89	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	1.40	
PERCOLATION	.05	.05	.85	1.00	.42	.05	.05	.05	.05	.05	.05	.05	2.72

TABLE 11
Water Balance Analysis
Alternative 2 - Dry Year 1976

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	6.89	11.06	18.67	20.72	18.67	12.94	4.50	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.50	2.78	4.82	5.43	4.51	2.65	.81	.00	.00	22.49

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 3 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.70	.98	2.54	2.63	1.55	1.71	1.53	3.43	.40	.43	.28	.45	17.63
SURFACE RUNOFF	1.70	.98	.76	.13	.08	.09	.08	.17	.02	.02	.08	.45	4.56
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.78	2.50	1.47	1.62	1.45	3.26	.38	.41	.20	.00	13.07
INITIAL SOIL MOISTURE	2.88	2.88	2.88	2.88	2.88	1.57	.00	.00	.00	.00	.00	.20	
TOTAL AVAILABLE MOISTURE	2.88	2.88	4.66	5.38	4.35	3.20	1.45	3.26	.38	.41	.20	.20	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.50	2.78	4.82	5.43	4.51	2.65	.81	.00	.00	22.49
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	1.50	2.73	3.15	1.40	3.21	.33	.36	.00	.00	12.68
REMAINING AVAILABLE MOISTURE	2.88	2.88	4.46	3.88	1.57	.00	.00	.00	.00	.00	.20	.20	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	1.40	
PERCOLATION	.05	.05	1.58	1.00	.05	.05	.05	.05	.05	.05	.05	.05	3.07

TABLE 12
Water Balance Analysis
Alternative 2 - Normal Year Following 1976

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	5.61	12.22	23.06	19.83	18.83	13.83	8.61	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 3 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.00	.74	1.50	2.21	3.53	4.42	3.67	3.89	3.80	2.29	1.84	1.14	30.03
SURFACE RUNOFF	1.00	.74	.45	.11	.18	.22	.18	.19	.19	.11	.55	1.14	5.07
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.05	2.10	3.35	4.20	3.49	3.70	3.61	2.18	1.29	.00	24.96
INITIAL SOIL MOISTURE	.20	.20	.20	1.25	2.25	2.67	.92	.00	.00	.89	1.60	2.89	
TOTAL AVAILABLE MOISTURE	.20	.20	1.25	3.35	5.60	6.86	4.41	3.70	3.61	3.07	2.89	2.89	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	.80	2.89	5.89	4.36	3.65	2.67	1.41	.00	.00	21.67
REMAINING AVAILABLE MOISTURE	.20	.20	1.25	2.25	2.67	.92	.00	.00	.89	1.60	2.89	2.89	
LINER MOISTURE	1.35	1.30	1.25	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	1.40	
PERCOLATION	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.60

TABLE 13
Water Balance Analysis
Alternative 2 - Wet Year 1977

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.83	7.83	17.33	16.50	20.89	15.78	13.33	7.11	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.14	1.66	4.36	4.19	5.44	3.74	2.68	1.24	.00	.00	23.45

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 3 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	.58	.44	3.28	3.81	2.87	4.21	2.97	7.01	5.41	2.49	4.09	1.85	39.01
SURFACE RUNOFF	.58	.44	.98	.19	.14	.21	.15	.35	.27	.12	1.23	1.85	6.52
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	2.30	3.62	2.73	4.00	2.82	6.66	5.14	2.37	2.86	.00	32.49
INITIAL SOIL MOISTURE	2.88	2.88	2.88	2.88	2.88	1.24	1.05	.00	2.92	2.88	2.88	2.88	
TOTAL AVAILABLE MOISTURE	2.88	2.88	5.18	6.50	5.61	5.24	3.87	6.66	8.06	5.25	5.74	2.88	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.14	1.66	4.36	4.19	5.44	3.74	2.68	1.24	.00	.00	23.45
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.14	1.66	4.31	4.14	3.82	3.69	2.68	1.24	.00	.00	21.68
REMAINING AVAILABLE MOISTURE	2.88	2.88	4.84	4.84	1.24	1.05	.00	2.92	5.39	4.00	5.74	2.88	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	
PERCOLATION	.05	.05	1.96	1.96	.05	.05	.05	.05	2.51	1.12	2.86	.05	10.76

TABLE 14
Water Balance Analysis
Alternative 2 - Wet Year 1978

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	3.22	13.28	16.33	18.11	18.89	15.22	7.06	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	.71	3.39	4.23	4.75	4.58	3.15	1.30	.00	.00	22.10

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 3 - CRANDON SITE
WET YEAR 1978

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	.55	.38	.15	2.75	4.66	4.48	7.70	9.15	5.70	1.42	1.54	1.44	39.92
SURFACE RUNOFF	.55	.38	.05	.14	.23	.22	.38	.46	.28	.07	.46	1.44	4.67
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	.11	2.61	4.43	4.26	7.31	8.69	5.41	1.35	1.08	.00	35.25
INITIAL SOIL MOISTURE	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.88	
TOTAL AVAILABLE MOISTURE	2.88	2.88	2.99	5.49	7.31	7.14	10.19	11.57	8.30	4.23	3.96	2.88	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	.71	3.39	4.23	4.75	4.58	3.15	1.30	.00	.00	22.10
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	.71	3.39	4.21	4.75	4.58	3.15	1.30	.00	.00	22.07
REMAINING AVAILABLE MOISTURE	2.88	2.88	2.88	4.69	3.92	2.88	5.45	6.99	5.15	2.88	3.96	2.88	
LINER MOISTURE	1.40	1.35	1.41	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	
PERCOLATION	.05	.05	.05	1.81	1.04	.05	2.57	4.11	2.27	.05	1.08	.05	13.18

TABLE 15
Water Balance Summary
Alternative 2

Yearly Variation	Precipitation (mm)	Precipitation (inches)	Runoff (mm)	Runoff (inches.)	Evapotranspiration (mm)	Evapotranspiration (inches)	Vertical Percolation (mm)	Vertical Percolation (inches)	Horizontal Percolation (mm)	Horizontal Percolation (inches)
Normal Year	762.8	30.03	128.8	5.07	564.6	22.23	16.0	0.63	53.1	2.09
Dry Year 1976	447.8	17.63	115.8	4.56	322.1	12.68	16.0	0.63	62.0	2.44
Normal Year Following 1976	762.8	30.03	128.8	5.07	550.4	21.67	15.2	0.60	0.0	0.00
Wet Year 1977	990.9	39.01	165.6	6.52	550.7	21.68	18.3	0.72	255.0	10.04
Wet Year 1978	1014.0	39.92	118.6	4.67	560.6	22.07	18.8	0.74	316.0	12.44

3.3 Alternative No. 3

3.3.1 Water Balance Analysis

The Alternative 3 cover system is the same as Alternative 2 except the upper till layer is reduced to 711 mm (28 inches). This reduces the overall moisture holding capabilities to a field capacity of 56.9 mm (2.24 inches) in this layer. The underdrain water movement and functioning of the drainage layer is the same as discussed for Alternative 2.

Tables 16-20 display the water balance results for the various yearly conditions. Table 21 provides a summary for these events and divides the percolation quantity into its individual components. Table 21 shows that Alternative 3 provides less evapotranspiration and more percolation than Alternative 2 because there is less moisture storage capability in the upper till layer. Vertical percolation for these two alternatives is similar.

TABLE 16
Water Balance Analysis
Alternative 3 - Normal Year

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	5.61	12.22	23.06	19.83	18.83	13.83	8.61	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 4 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.00	.74	1.50	2.21	3.53	4.42	3.67	3.89	3.80	2.29	1.84	1.14	30.03
SURFACE RUNOFF	1.00	.74	.45	.11	.18	.22	.18	.19	.19	.11	.55	1.14	5.07
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.05	2.10	3.35	4.20	3.49	3.70	3.61	2.18	1.29	.00	24.96
INITIAL SOIL MOISTURE	2.24	2.24	2.24	2.24	2.24	2.24	.49	.00	.00	.89	1.60	2.24	
TOTAL AVAILABLE MOISTURE	2.24	2.24	3.29	4.34	5.59	6.44	3.98	3.70	3.61	3.07	2.89	2.24	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.89	3.93	3.65	2.67	1.41	.00	.00	21.59
REMAINING AVAILABLE MOISTURE	2.24	2.24	3.09	3.24	2.66	.49	.00	.00	.89	1.60	2.89	2.24	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	
PERCOLATION	.05	.05	.85	1.00	.42	.05	.05	.05	.05	.05	.65	.05	3.31

TABLE 17
Water Balance Analysis
Alternative 3 - Wet Year 1976

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	6.89	11.06	18.67	20.72	18.67	12.94	4.50	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.50	2.78	4.82	5.43	4.51	2.65	.81	.00	.00	22.49

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 4 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.70	.98	2.54	2.63	1.55	1.71	1.53	3.43	.40	.43	.28	.45	17.63
SURFACE RUNOFF	1.70	.98	.76	.13	.08	.09	.08	.17	.02	.02	.08	.45	4.56
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.78	2.50	1.47	1.62	1.45	3.26	.38	.41	.20	.00	13.07
INITIAL SOIL MOISTURE	2.24	2.24	2.24	2.24	2.24	.93	.00	.00	.00	.00	.00	.20	
TOTAL AVAILABLE MOISTURE	2.24	2.24	4.02	4.74	3.71	2.56	1.45	3.26	.38	.41	.20	.20	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.50	2.78	4.82	5.43	4.51	2.65	.81	.00	.00	22.49
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	1.50	2.73	2.51	1.40	3.21	.33	.36	.00	.00	12.04
REMAINING AVAILABLE MOISTURE	2.24	2.24	3.82	3.24	.93	.00	.00	.00	.00	.00	.20	.20	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	1.40	
PERCOLATION	.05	.05	1.58	1.00	.05	.05	.05	.05	.05	.05	.05	.05	3.07

TABLE 18
Water Balance Analysis
Alternative 3 - Normal Year Following 1976

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	5.61	12.22	23.06	19.83	18.83	13.83	8.61	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 4 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	1.00	.74	1.50	2.21	3.53	4.42	3.67	3.89	3.80	2.29	1.84	1.14	30.03
SURFACE RUNOFF	1.00	.74	.45	.11	.18	.22	.18	.19	.19	.11	.55	1.14	5.07
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	1.05	2.10	3.35	4.20	3.49	3.70	3.61	2.18	1.29	.00	24.96
INITIAL SOIL MOISTURE	.20	.20	.20	1.25	2.25	2.24	.49	.00	.00	.89	1.60	2.24	
TOTAL AVAILABLE MOISTURE	.20	.20	1.25	3.35	5.60	6.44	3.98	3.70	3.61	3.07	2.89	2.24	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	1.10	2.94	5.94	5.10	4.45	2.72	1.46	.00	.00	23.72
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	.80	2.94	5.89	3.93	3.65	2.67	1.41	.00	.00	21.29
REMAINING AVAILABLE MOISTURE	.20	.20	1.25	2.25	2.67	.49	.00	.00	.89	1.60	2.89	2.24	
LINER MOISTURE	1.35	1.30	1.25	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	
PERCOLATION	.05	.05	.05	.05	.43	.05	.05	.05	.05	.05	.65	.05	1.57

TABLE 19
Water Balance Analysis
Alternative 3 - Wet Year 1977

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUI	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.83	7.83	17.33	16.50	20.89	15.78	13.33	7.11	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.14	1.66	4.36	4.19	5.44	3.74	2.68	1.24	.00	.00	23.45

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 3 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	.58	.44	3.28	3.81	2.87	4.21	2.97	7.01	5.41	2.49	4.09	1.85	39.01
SURFACE RUNOFF	.58	.44	.98	.19	.14	.21	.15	.35	.27	.12	1.23	1.85	6.52
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	2.30	3.62	2.73	4.00	2.82	6.66	5.14	2.37	2.86	.00	32.49
INITIAL SOIL MOISTURE	2.24	2.24	2.24	2.24	2.24	.60	.41	.00	2.24	2.24	2.24	2.24	
TOTAL AVAILABLE MOISTURE	2.24	2.24	4.54	5.86	4.97	4.60	3.23	6.66	7.38	4.61	5.10	2.24	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.14	1.66	4.36	4.19	5.44	3.74	2.68	1.24	.00	.00	23.45
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.14	1.66	4.31	4.14	3.18	3.74	2.68	1.24	.00	.00	21.09
REMAINING AVAILABLE MOISTURE	2.24	2.24	4.20	4.20	.60	.41	.00	2.92	4.70	3.36	5.10	2.24	
LINER MOISTURE	1.35	1.30	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	
PERCOLATION	.05	.05	1.96	1.96	.05	.05	.05	.68	2.46	1.12	2.86	.05	11.35

TABLE 20
Water Balance Analysis
Alternative 3 - Wet Year 1978

POTENTIAL EVAPOTRANSPIRATION DATA
(THORNTHWAITE AND MATHER METHOD)

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
TEMPERATURE ABOVE FREEZING	.00	.00	.00	3.22	13.28	16.33	18.11	18.89	15.22	7.06	.00	.00	
ADJUSTING FACTOR FOR LATITUDE	.79	.81	1.02	1.13	1.29	1.30	1.32	1.21	1.04	.94	.79	.75	
POTENTIAL EVAPOTRANSPIRATION	.00	.00	.00	.71	3.39	4.23	4.75	4.58	3.15	1.30	.00	.00	22.10

(CAP CONSTRAINT ON MONTHLY PERCOLATION RATE USED)

EXXON MINERALS - WATER BUDGET ANALYSIS NO. 4 - CRANDON SITE

PARAMETERS	JAN	FEB	MAR	APL	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
AVERAGE PRECIPITATION	.55	.38	.15	2.75	4.66	4.48	7.70	9.15	5.70	1.42	1.54	1.44	39.92
SURFACE RUNOFF	.55	.38	.05	.14	.23	.22	.38	.46	.28	.07	.46	1.44	4.67
MOISTURE AVAILABLE FOR INFILTRATION	.00	.00	.11	2.61	4.43	4.26	7.31	8.69	5.41	1.35	1.08	.00	35.25
INITIAL SOIL MOISTURE	2.24	2.24	2.24	2.24	2.24	2.24	2.24	2.24	2.24	2.24	2.24	2.24	
TOTAL AVAILABLE MOISTURE	2.24	2.24	2.34	4.85	6.67	6.50	9.56	10.93	7.65	3.59	3.32	2.24	
POTENTIAL EVAPO-TRANSPIRATION	.00	.00	.00	.71	3.39	4.23	4.75	4.58	3.15	1.30	.00	.00	22.10
ACTUAL EVAPO-TRANSPIRATION	.00	.00	.00	.71	3.39	4.21	4.75	4.58	3.15	1.30	.00	.00	22.07
REMAINING AVAILABLE MOISTURE	2.24	2.24	2.24	4.05	3.28	2.24	4.81	6.35	4.51	2.24	3.32	2.24	
LINER MOISTURE	1.40	1.35	1.41	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.45	
PERCOLATION	.05	.05	.05	1.81	1.04	.05	2.57	4.11	2.27	.05	1.08	.05	13.18

TABLE 21
Water Balance Summary
Alternative 3

Yearly Variation	Precipitation (mm)	Precipitation (inches)	Runoff (mm)	Runoff (inches.)	Evapotranspiration (mm)	Evapotranspiration (inches)	Vertical Percolation (mm)	Vertical Percolation (inches)	Horizontal Percolation (mm)	Horizontal Percolation (inches)
Normal Year	762.8	30.03	128.8	5.07	548.4	21.59	16.3	0.64	67.8	2.67
Dry Year 1976	447.8	17.63	115.8	4.56	305.8	12.04	16.0	0.63	62.0	2.44
Normal Year Following 1976	762.8	30.03	128.8	5.07	540.8	21.29	15.7	0.62	24.1	0.95
Wet Year 1977	990.9	39.01	165.6	6.52	535.7	21.09	18.5	0.73	269.7	10.62
Wet Year 1978	1014.0	39.92	118.6	4.67	560.6	22.07	18.8	0.74	316.0	12.44

4.0 Summary and Conclusions

The preceding discussion details the development and utilization of a water balance model for three alternate cover systems. The analysis considers a normal year plus the extreme dry and wet years identified from 42 years of records at Rhinelander, Wisconsin.

Calculations for Alternative 1 show that the bentonite modified soil layer restricts vertical percolation causing fluctuations in the soil moisture level in the upper till layer to values in excess of the field capacity. For the wettest year on record, the upper till layer becomes completely saturated for nearly 4 months resulting in a significant increase to the total runoff quantity. In the normal year, the soil moisture reaches the wilt point for 1 month, and in the dry year of 1976 for 5 months. During these periods, the only moisture available for plant uptake is that portion entering the soil profile from infiltration. Total percolation to the tailings is conservatively estimated as 15.2 to 112.8 mm (0.60 to 4.44 inches) with the normal year at 39.6 mm (1.56 inches). This percolation is normally concentrated in the March-May period when infiltration from snowmelt and rainfall exceeds the limited evapotranspiration occurring at this time.

Alternatives 2 and 3 employ a drainage layer above the bentonite modified soil layer which will convey all flow delivered from the above till layer. Use of this drainage layer allows the upper till layer to function independent from conditions existing in the lower layers. Under these systems, the soil moisture in the till layer will not exceed field

capacity, with the layer operating essentially as a free draining, natural condition. Analysis of the drainage-bentonite modified soil interface indicates that only 2 to 6 percent of the flow delivered to the drainage layer will ever reach the tailings deposit. Considering the extreme years, the actual percolation to the tailings varies from 15.2 to 18.8 mm (0.60 to 0.74 inches) annually. With these cover systems, the wet years will no longer result in adverse conditions. Potential difficulties with this system would occur during dry periods since the soil can no longer store moisture above field capacity. Calculations show that soil moisture in a normal year will reach the wilt point in August and September and will affect 5 months during the dry year of 1976. This condition would occur for any natural black spruce forest provided their root systems are in similar till material.

Table 22 provides an overview of all three alternatives. Runoff calculations were the same for all conditions, except Alternative 1 for wet year 1978 which resulted in a period of complete soil saturation. Alternative 1 provides the greatest evapotranspiration with the least occurring for Alternative 3. This difference is due to the ability of the upper till layer to retain excess moisture for plant uptake in later months. Comparison of the percolation in Alternative 1 with the vertical percolation in Alternatives 2 and 3 shows the efficiency of the drainage layer in reducing percolation to the tailings. The horizontal percolation values for Alternatives 2 and 3 will exit the system via the drainage layer and will be combined with overland runoff around the site perimeter.

TABLE 22
Water Balance Summary

Yearly Variation	Precipitation (mm)	Precipitation (inches)	Evapotranspiration* (mm)	Evapotranspiration* (inches)	Vertical Percolation (mm)	Vertical Percolation (inches)	Horizontal Percolation (mm)	Horizontal Percolation (inches)
Alternative 1								
Normal Year	762.8	30.03	594.1	23.39	39.6	1.56	--	--
Dry Year 1976	447.8	17.63	360.4	14.19	39.6	1.56	--	--
Normal Year Following 1976	762.8	30.03	550.4	21.67	15.2	0.60	--	--
Wet Year 1977	990.9	39.01	594.4	23.40	88.4	3.48	--	--
Wet Year 1978	1014.0	39.92	561.3	22.10	112.8	4.44	--	--
Alternative 2								
Normal Year	762.8	30.03	564.6	22.23	16.0	0.63	53.1	2.09
Dry Year 1976	447.8	17.63	322.1	12.68	16.0	0.63	62.0	2.44
Normal Year Following 1976	762.8	30.03	550.4	21.67	15.2	0.60	--	--
Wet Year 1977	990.9	39.01	550.7	21.68	18.3	0.72	255.0	10.04
Wet Year 1978	1014.0	39.92	560.6	22.07	18.8	0.74	316.0	12.44
Alternative 3								
Normal Year	762.8	30.03	548.4	21.59	16.3	0.64	67.8	2.67
Dry Year 1976	447.8	17.63	305.8	12.04	16.0	0.63	62.0	2.44
Normal Year Following 1976	762.8	30.03	540.8	21.29	15.7	0.62	24.1	0.95
Wet Year 1977	990.9	39.01	535.7	21.09	18.5	0.73	269.7	10.62
Wet Year 1978	1014.0	39.92	560.6	22.07	18.8	0.74	316.0	12.44

* Evapotranspiration includes plant interception estimated as 20 to 30 percent of total precipitation.

Alternative 1 has the advantage of providing 10 to 15 percent more moisture for evapotranspiration during the dry period of 1976. Also, Alternative 1 does not require the processing and placement of a drainage layer required for the other alternatives. The disadvantages include a higher rate of percolation to the tailings deposit and occurrence of complete saturation under extreme wet conditions which could be detrimental to the black spruce forest.

Alternatives 2 and 3 will significantly decrease percolation to the tailings given the differential hydraulic conductivity between the drainage layer and bentonite modified soil layer. Alternative 2 provides a greater till depth, 914 versus 711 mm (36 versus 28 inches), for Alternative 3 and supplies approximately 5 percent more moisture for evapotranspiration during dry periods at the cost of placing additional till material.

Alternative 2 cover system provides the best solution, disregarding the additional processing and placement costs. This system reduces the percolation to tailings, provides greater evapotranspiration for the black spruce than Alternative 3, and has a larger buffer zone which decreases the probability of the black spruce root system penetrating the drainage or bentonite modified soil layers.

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Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

Report on

UNDERDRAIN REVIEW
CRANDON PROJECT
WASTE DISPOSAL SYSTEM
PROJECT REPORT 3.2

Submitted to:

Exxon Minerals Company
Post Office Box 813
Rhinelander, Wisconsin 54501

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March, 1982

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CONSULTING GEOTECHNICAL AND MINING ENGINEERS

March 8, 1982

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Exxon Minerals Company
P. O. Box 813
Rhinelander, Wisconsin 54501

Attention: Mr. C. E. Fowler

Re: Exxon Crandon Project
Waste Disposal System
Crandon Wisconsin

Gentlemen:

Enclosed for your review and comment is the final draft of Project Report 3.2, Underdrain System Review. This report describes the alternative underdrain systems reviewed and presents our recommendations for an underdrain system, should Exxon elect to use one.

Please call us if you have any questions.

Very truly yours,

GOLDER ASSOCIATES

Gary H. Collison

Gary H. Collison, P.E.
Associates

JFC:GHC:dap

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1.0 INTRODUCTION

This report presents Golder Associates' review of the feasibility of using an underdrain system above a tailings disposal pond liner for the Crandon Project to reduce the amount of seepage through the liner during operation. This study focuses on the design of the underdrain system to act as a filter system to inhibit migration of tailings into the drain, the hydraulic capacity of the underdrain to accomodate estimated seepage volumes, and methods to collect and remove the seepage from the underdrain system. The concepts presented are directed toward designs for the Crandon Project but some details are omitted since they will have to be specific to a given slurry tailings pond.

The primary purpose of an underdrain system is to collect seepage for removal from the disposal system. To do so, the underdrain must be designed to inhibit migration of solid material into the underdrain but allow sufficient hydraulic capacity for the volume of seepage flow. Drainage systems to meet these requirements are commonplace in many water retention dams to prevent piping of the foundation or embankment materials. These drainage systems are designed as soil filters and the design criteria are well known.

An underdrain system may range from a series of collection pipes surrounded by filter material to a complete blanket of filter and/or drain material across a disposal facility. The design of an underdrain system is dependent on the grain size of the disposed waste, in this case the tailings. An underdrain system may be comprised of one, two, or even three layers of material, each filtering the previous layer to allow drainage but not migration of fines.

An underdrain system will reduce the pressure head of pond water acting against a liner beneath the drain. The amount of head reduction depends on the permeability of the tailings and the hydraulic characteristics of the underdrain system. Reducing the head will reduce the driving force of the seepage on the liner and hence reduce the amount of seepage through the liner.

2.0 FILTER DESIGN

2.1 Filter Design Criteria

Several somewhat different sets of criteria exist for the design of filters based on theoretical work and practical applications(Ref. 1,2,3,4). The following are well known criteria proposed by the USBR(Ref. 1) and the U.S. Army Corps of Engineers(Ref. 4):

1) $\frac{D_{15}\ \text{filter}}{D_{85}\ \text{base}} \leq 5$ or less

2) $\frac{D_{15}\ \text{filter}}{D_{15}\ \text{base}} \leq 5$ to 40

3) $\frac{D_{50}\ \text{filter}}{D_{50}\ \text{base}} \leq 25$

In these expressions, D_{15} is the size at which 15% of the total particles are finer by weight, D_{85} is the size at which 85% of the total particles are finer by weight, and D_{50} is the size at which 50% of the total particles are finer by weight. When more than one filter layer is required, the same criteria are followed with the finer filter considered the "base" for selection of the gradation of the coarser filter.

The intent of criterion 1 is to assure that migration of fines from the base material into the filter material will not occur. For a filter system this criterion is of primary importance since migration of fines will cause clogging of the filter. Criterion 2 is to insure sufficient permeability in filters and drains to prevent the build-up of large seepage forces; this is basically a check on the hydraulic capability of the drain in meeting dis-

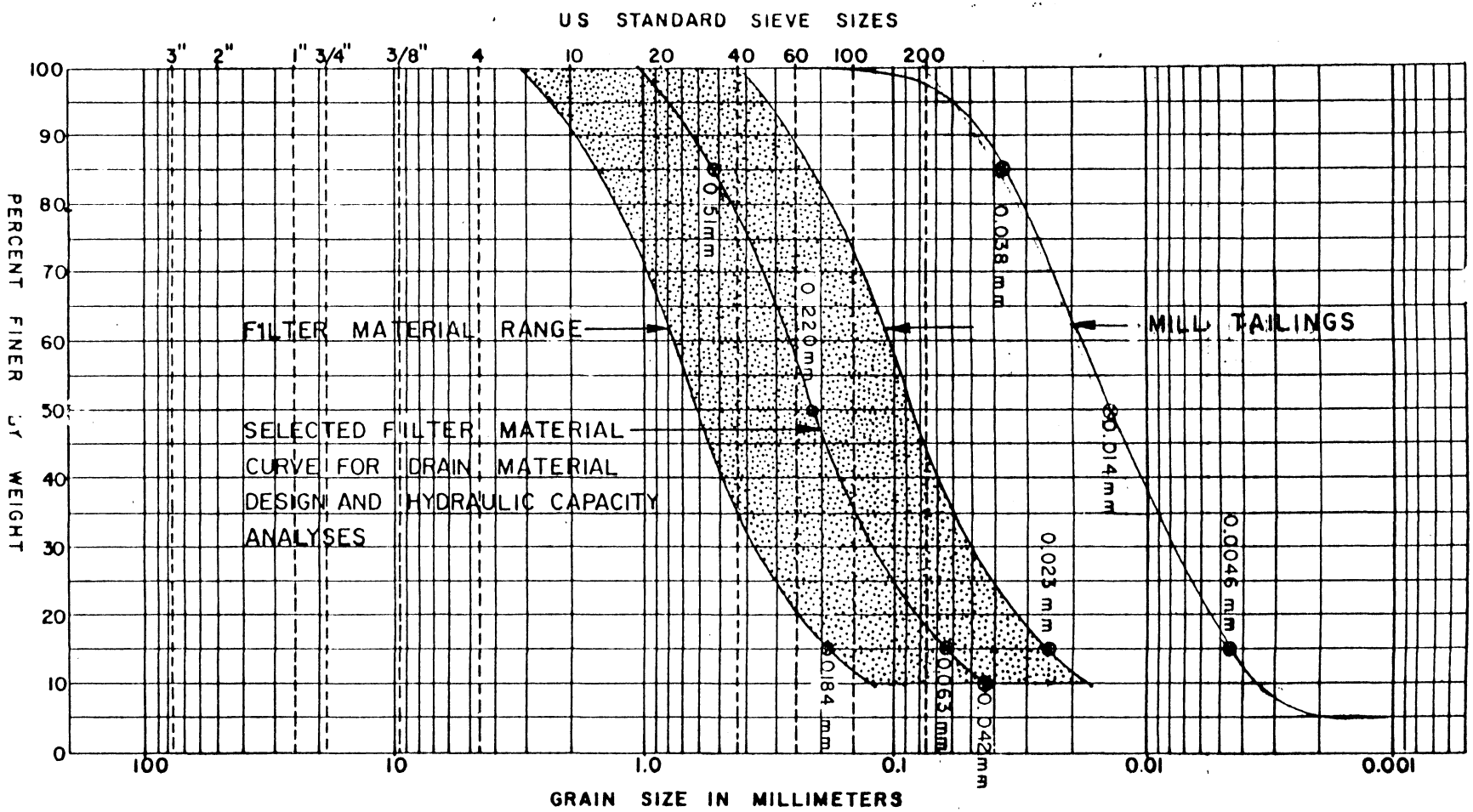
charge needs. Commonly, drains are designed with less than 5% of the material passing the #200 U.S. Standard Sieve (0.074 mm) but this is dependent on the grain size of the material being drained. Criterion 3 is to assure that the grain size curve of the filter is roughly parallel to that of the base material so that there is filtering action throughout the range of grain sizes.

2.2 Materials Design

For the Crandon Project application, the base material for underdrain design is the tailings. The size gradation for the tailings, determined from laboratory tests on a sample supplied by Exxon through Lakefield Research of Crandon, is shown on Figure 2.1. Laboratory permeability tests on tailings materials indicate a hydraulic conductivity of about 5×10^{-8} m/s (1.6×10^{-7} ft./sec.) to be a reasonable estimate for design of the tailings facilities (Ref. 6).

The range of filter material gradations which meets Criteria 1 and 2 discussed in Section 2.1. is also shown on Figure 2.1. The coarser end of the band is slightly greater than 25 times the D_{50} of the tailings but the band is roughly parallel to the tailings grain size curve. Because of the very fine grained nature of the tailings, much of the appropriate filter material could have more than 5% finer than the #200 sieve. Because of the fine grain sizes of this filter, a second more permeable layer will be needed to provide sufficient hydraulic capacity for the underdrain system. The details of the hydraulic capacity analyses are presented in Section 4 of this report.

GRAIN SIZE DISTRIBUTION
FILTER MATERIAL DESIGN



COBBLES	GRAVEL		SAND			FINES*	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

D_{15} FILTER $\leq$ 5(D_{85} TAILINGS) = 5(0.038) = 0.19 mm

D_{15} FILTER $\geq$ 5(D_{15} TAILINGS) = 5(0.0046) = 0.023 mm

D_{15} FILTER $\leq$ 40(D_{15} TAILINGS) = 40(0.0046) = 0.184 mm

D_{50} FILTER $\leq$ 25(D_{50} TAILINGS) = 25(0.014) = 0.35 mm

FIGURE 2.1

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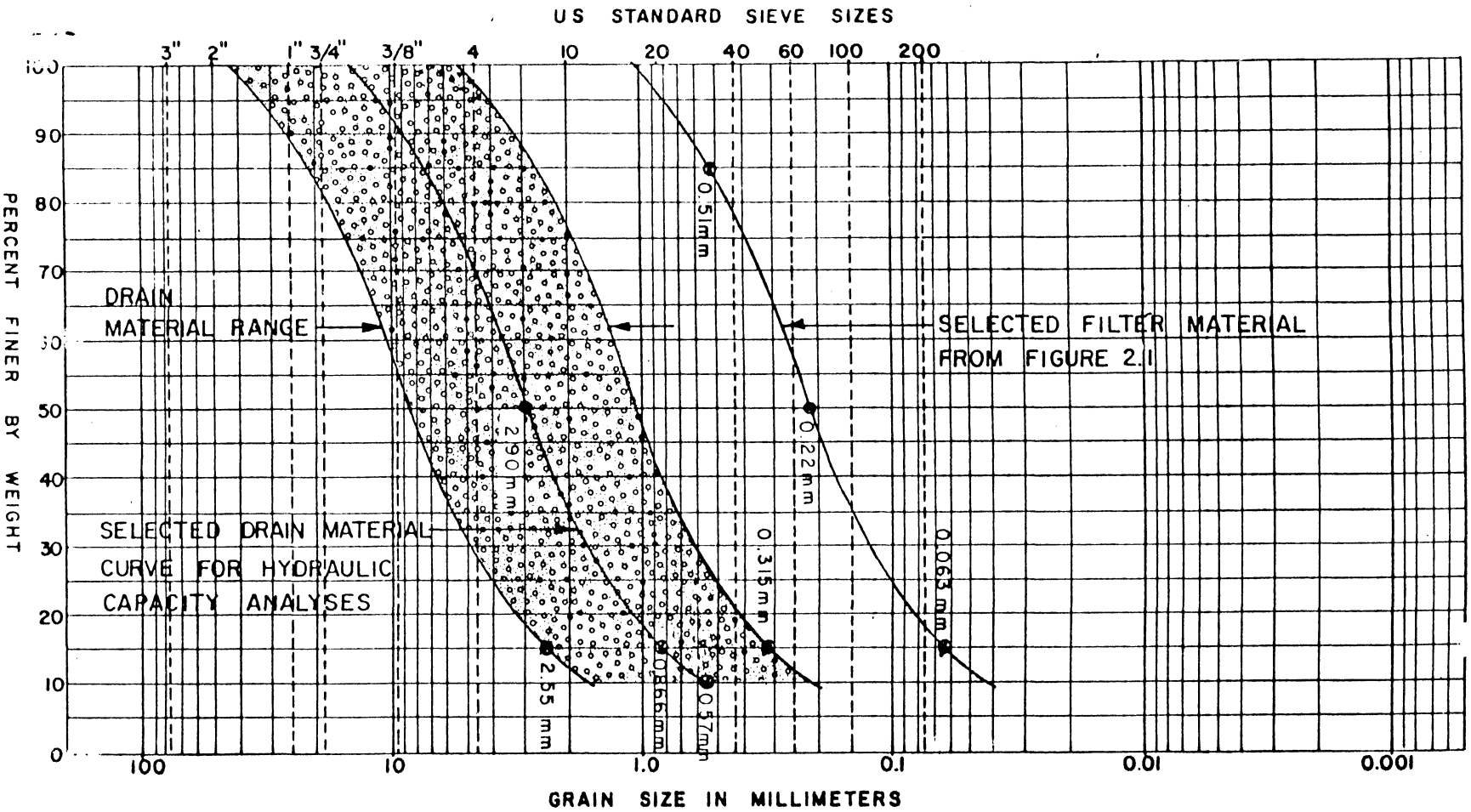
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Design of the second material, the drain layer, is based on the gradation of the filter material. However, the range of the gradation of the filter material is wide, and applying the design criteria to both ends of the filter band would yield two bands of drain material. To illustrate the feasibility of the underdrain system, a filter gradation curve has been selected for the design basis of the drain material and it is shown on Figure 2.1 and Figure 2.2. The curve selected is located such that 75% of the filter band at the D_{15} size is coarser, thus having lower hydraulic conductivity than the average gradation of the filter material and hence provides a somewhat conservative estimate of filter thickness. The drain material band determined from the selected filter gradation curve is shown on Figure 2.2. Also shown on Figure 2.2 is the drain gradation curve used in hydraulic capacity estimates. It has been selected such that 75% of this drain band is coarser to provide a somewhat lower than average hydraulic conductivity and hence a more conservative drain layer thickness estimate.

In comparing the filter material gradation band to the gradation curves of till from Site 41, shown on Figure 2.3, it is apparent that the native till will nearly satisfy Criterion 1 for a filter to the tailings. Although less satisfactory for Criterion 2 and 3, these are less important in an underdrain situation than when applied to design of filters for dam embankments. Also, the drain material gradation band shown on Figure 2.2 will essentially satisfy Criterion 1 and 3 using the till as the base material. Thus, there are enough fines in the till to prevent migration of the tailings, and the drain is fine enough to prevent migration of the till. Use of till as filter material could be considered if an extra thickness allowance were

GRAIN SIZE DISTRIBUTION

DRAIN MATERIAL DESIGN



COBBLES	GRAVEL		SAND			FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

D_{15} DRAIN $\leq 5(D_{85}$ FILTER$) = 5(0.51) = 2.55$ mm

D_{15} DRAIN $\geq 5(D_{15}$ FILTER$) = 5(0.063) = 0.315$ mm

D_{15} DRAIN $\leq 40(D_{15}$ FILTER$) = 40(0.063) = 2.52$ mm

D_{50} DRAIN $\leq 25(D_{50}$ FILTER$) = 25(0.22) = 5.5$ mm

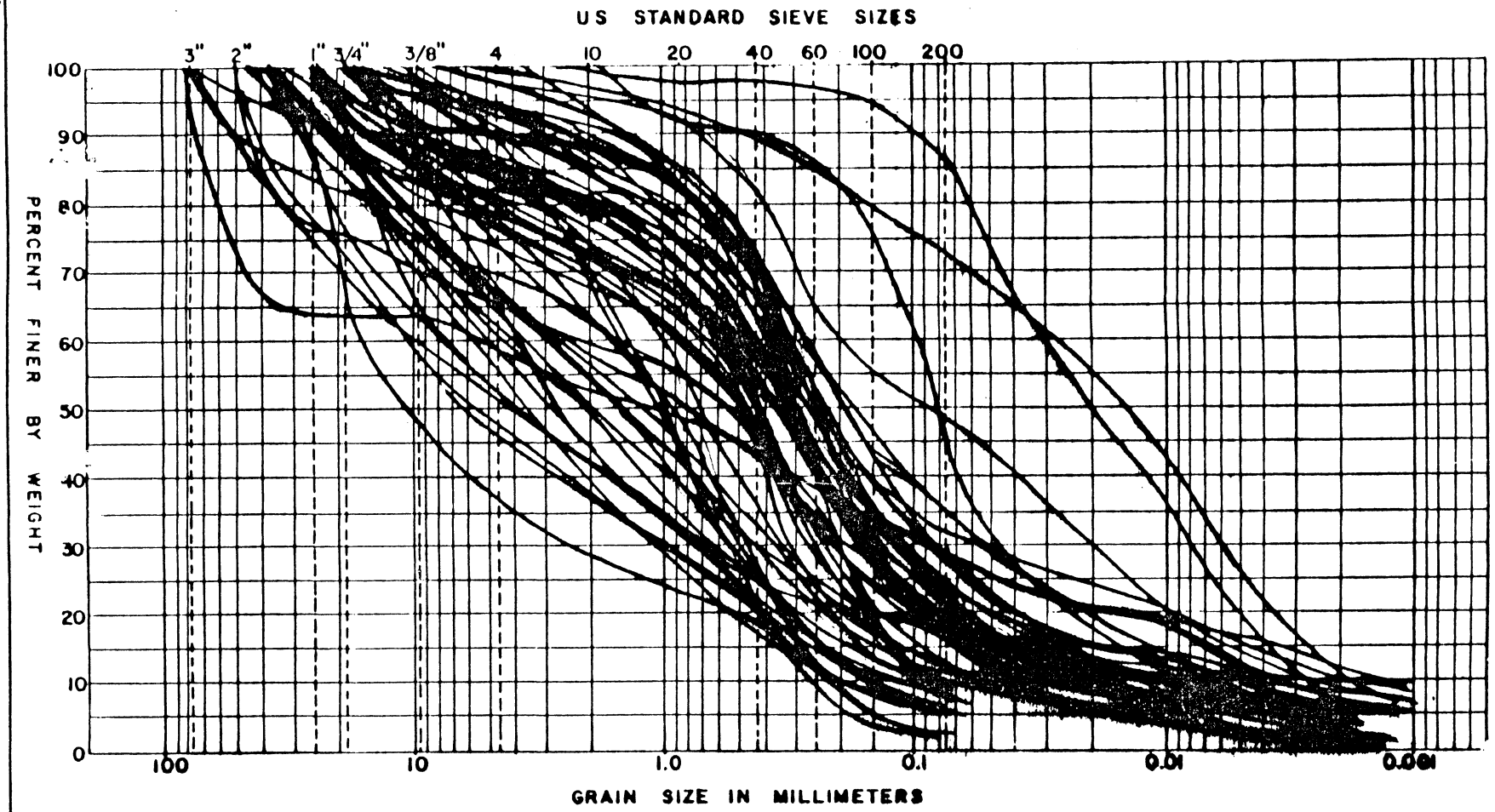
FIGURE 2.2

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GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO.	ELEV. OR DEPTH	W_n	W_L	W_p	I_p	DESCRIPTION OR CLASSIFICATION
-	-	-	-	-	-	Samples of Till from Site 41 (Reference 8)

FIGURE 2.3

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made in the drain material to assure that any small amount of fines from the finer grain sized till materials passing into the drain would not clog the drain.

3.0 UNDERDRAIN SYSTEM

3.1 Hydraulic Capacity

The hydraulic capacity of an underdrain system is a measure of the amount of water which the system can transmit for removal. For this project, the concept of the underdrain system is to reduce seepage from the waste disposal pond by reducing the head on the liner beneath the underdrain. An effective underdrain system is one in which the amount of water that can be transmitted by the underdrain is greater than the seepage through the tailings and one which reduces the head acting on the liner to a practical minimum.

Hydraulic capacities of the underdrain systems considered were designed to achieve atmospheric pressure at the bottom of the tailings and honor continuity; outflow through the underdrain must be equal to inflow to the underdrain plus the change in storage within the underdrain. The change in storage in this application was considered a second order effect and taken as zero.

It is presently estimated that the volume of slurry pumped into a pond will be about 900 gallons per minute (5.7×10^{-2} m^3/s) with 690 gallons per minute (4.3×10^{-2} m^3/s) of the slurry being water. This will be a constant rate for most of the life of a pond (lesser during start-up operations). The amount of water contained at any one time in the pore spaces of the settled tailings is about 240 gallons per minute of tailings slurry input (1.5×10^{-2} m^3/s) based on a 50 percent solids (by weight) slurry with an estimated tailings void ratio of 1.1, an estimated tailings dry density of 95 pounds per cubic foot (1522 kg/m^3), and an average tailings specific gravity of

3.22(Ref. 6). Thus, about 450 gallons of water per minute (2.8×10^{-2} m^3/s) will be available for recycle. Of this, about 340 gallons per minute (2.1×10^{-2} m^3/s) could pass through the underdrain system (see Figure 3.1) with about 110 gallons per minute (0.69×10^{-2} m^3/s) becoming ponded water to be decanted.

In addition to the water entering the pond with the tailings, precipitation falling on the pond will also be retained. The average annual precipitation for the area is 30.77 inches (782 mm) and the estimated lake evaporation rate is 23.36 inches (593 mm). Thus, the estimated precipitation retained in a ponded area is 7.41 inches (188 mm). For a 100 acre (40 ha) pond this is equivalent to an average annual amount of about 38 gallons per minute (2.4×10^{-3} m^3/s).

As presently envisioned, the tailings ponds will include separate underdrain and decant systems. The underdrain system will be used to drain and evacuate water seeping through the tailings into the underdrain and water seeping into the underdrain system in areas where free water is ponded. In the ponded water area, the very fine tailings particles enter in suspension and slowly settle out, leaving clarified water at the top. This clarified water will be removed by the decant system. A simple decant system such as a barge mounted pump could be used.

It is not desirable to have the free pond water flow directly into the underdrain system from both a volume and turbidity standpoint. The less water passing through the underdrain, the less is the potential for contamination of underdrain. In order to minimize the volume of water, and to clarify turbid water trying to seep into the underdrain

a retarding layer over the filter may be needed. This added retarding layer could be glacial till, waste rock (depending on its gradation), or tailings spigotted down the side slopes of the pond. Either method will suffice and the final selection is somewhat dependent on the shape and operational details of the tailings pond. If glacial till is employed as the filter, it need only be placed thicker to act as both the filter and retarding layer.

3.2 Blanket Underdrain Analysis

Flow estimates of water through the tailings and hence through a blanket underdrain system were made using Darcy's equation:

$$Q=kiA$$

where:

Q = flow (L^3/T)
k = hydraulic conductivity (L/T)
i = hydraulic gradient (L/L)
A = area (L^2)

A more detailed explanation of seepage estimating procedures is presented in Reference 5.

The hydraulic capacity of blanket underdrain systems were assessed in terms of cross-section area (which can be reduced to thickness) for various gradients with the hydraulic conductivities estimated from the filter and drain material bands previously discussed.

3.3 Parallel Pipe Underdrain Analysis

Parallel pipe underdrain systems were not specifically analyzed for the quantity of flow which could be transmitted. The small quantities of flow estimated from the tailings can easily be handled with small diameter pipes, in the 4 to 6 inch (102 mm to 152 mm) range so that the specific size of pipes is a detail left to the final de-

sign. The head reduction on the liner for this type of system is related to the spacing between the pipes. Drain spacing and head relationship is defined by the following equation (Ref. 7):

$$W^2 = \frac{4k\ (b^2-a^2)}{q_d}$$

where
- W = drain spacing (L)
- a = height of drain base above the liner (L)
- b = maximum head on liner between drain (L)
- k = hydraulic conductivity of tailings (L/T)
- q_d = seepage through the tailings per unit area (L/T)

3.4 Combination Underdrain System Analysis

Consideration was also given to a parallel pipe underdrain system connected by a thin blanket of filter material; a combined system. This was done to examine whether a decrease in volume of blanket underdrain material and the number of parallel pipes could achieve a more economic system than could be attained by either system alone. All underdrain systems for this project were analyzed as having at least one collector pipe to channel the water to a discharge point. For purposes of discussion, any other pipe not directly connected to the discharge point is termed a pipe drain.

For combined underdrain systems, the analyses used varying thicknesses of filter material between pipe drains and varying filter bed slopes. Pipe drain spacing could then be determined by adding twice the length of the blanket, as limited by the required hydraulic capacity for a given filter bed thickness, to the pipe drain spacing determined for parallel pipe drains considered alone (see Figure 3.3). The pipe drain size was considered a detail, as previously discussed, and not calculated.

3.5 Analyses Parameters

To compare effectiveness of various underdrain systems, conceptual designs and analyses were applied to a square, nominal 100 acre (40 ha) pond with a maximum tailings depth of 70 feet (21.3 m). A sketch of the pond with dimensions used in the analyses is shown on Figure 3.1.a.

Seepage flow into the blanket underdrain system was estimated separately for the pond bottom area and pond side slope areas. These seepage flow estimates are presented on Figure 3.1.b.

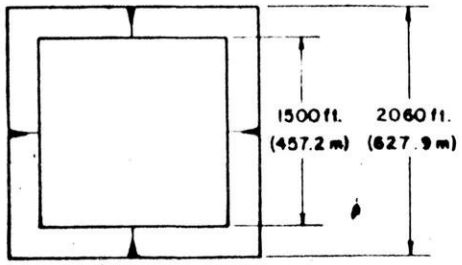
Hydraulic conductivity estimates for the filter material and drain material are given on Figure 3.1.c.

3.6 Underdrain System Analyses

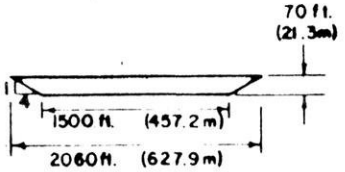
The various bottom underdrain schemes analyzed for this study included: 1) blanket systems with and without bottom slopes and with either a single end or full perimeter collection pipe, 2) pipe drain system with variable pipe spacings, and 3) combination pipe drain and blanket drain systems with varying slopes between the pipes and varying thicknesses of filter material between the pipes. These analyses for the pond bottom systems are presented on Figures 3.2 and 3.3.

Analyses of the underdrain configuration on the pond side slopes included: 1) a single layer filter material blanket, 2) a single layer filter material blanket with parallel pipe drains, and 3) a two layer filter and drain material blanket system. These analyses are presented on Figure 3.4.

a)



PLAN VIEW



CROSS SECTION

TOTAL AREA = 97.4 acres = 39.4 hectares
SLOPE AREA = 2.05×10^6 ft^2 (1.9×10^5 m^2)
BOTTOM AREA = 2.25×10^6 ft.^{2} (2.09×10^5 m^2)

b) SEEPAGE THROUGH THE TAILINGS BY DARCY'S LAW

$Q = kiA$

ASSUMING:

a. 3 ft.(0.9 m) of WATER OVER TAILINGS
b. MAXIMUM AREA OF FLOW
c. AVERAGE GRADIENT FOR MAXIMUM AREA

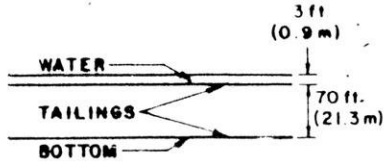
WITH:

Q = FLOW (L^3/T)
k = HYDRAULIC CONDUCTIVITY (L/T) OF TAILINGS
= 5.0×10^{-8} m/s (1.6×10^{-7} ft./sec) (REFERENCE 6)
i = HYDRAULIC GRADIENT (L/L)
A = AREA (L^2)

POND BOTTOM:

MAXIMUM DEPTH, TAILINGS = 70ft. (21.3m)
AVERAGE DEPTH, WATER = 3ft. (0.9 m)

$i = \frac{70 + 3}{70} = 1.04$ say $i = 1.1$

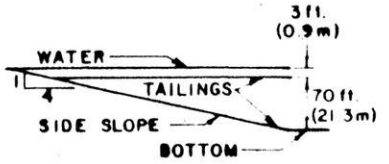


$Q = kiA$
$= (1.6 \times 10^{-7}$ ft./sec)$(1.1)(2.25 \times 10^6$ ft.$^2)$
$= 0.40$ cfs (180 gpm)$(1.13 \times 10^{-2}$ m^3/s)

POND SLOPES:

AVERAGE DEPTH, TAILINGS = 35 ft. (10.7m)
AVERAGE DEPTH, WATER = 3 ft. (0.9 m)

$i = \frac{35 + 3}{35} = 1.08$ say 1.1



$Q = kiA$
$= (1.6 \times 10^{-7}$ ft./sec)$(1.1)(2.05 \times 10^6$ ft.$^2)$
$= 0.36$ cfs (160 gpm)$(1.02 \times 10^{-2}$ m^3/s)

c) HYDRAULIC CONDUCTIVITY OF FILTER AND DRAIN MATERIALS

HAZEN'S APPROXIMATION: k (cm/sec) = $[D_{10}$ (mm)$]^2$

FILTER: $k_f = (0.042)^2$
$= (1.8 \times 10^{-3}$ cm/sec)
say 1.0×10^{-5} m/s (3.3×10^{-5} ft./sec), rounded to the nearest half order of magnitude

DRAIN: $k_D = (0.57)^2$
$= (3.2 \times 10^{-1}$ cm/sec)
say 5.0×10^{-3} m/s (1.6×10^{-2} ft./sec), rounded to the nearest half order of magnitude

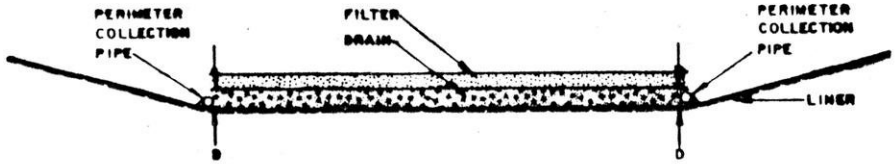
JOB NO. 786085	SCALE NOT TO SCALE	CONCEPTUAL ANALYSES PARAMETERS
DRAWN SKB	DATE 1-11-82	
CHECKED	DWG. NO. —	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 3.1

1500 ft. (457.2 m)
FIGURES A THROUGH D

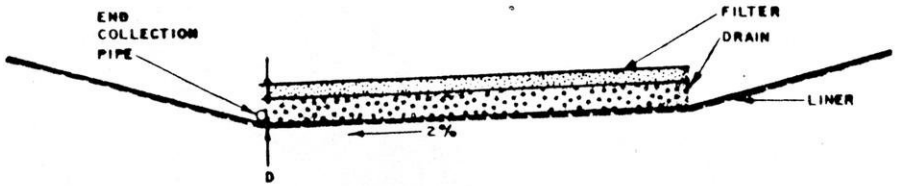
A.) HOR. BOTTOM UNDERDRAIN WITH COLLECTION AT ONE END



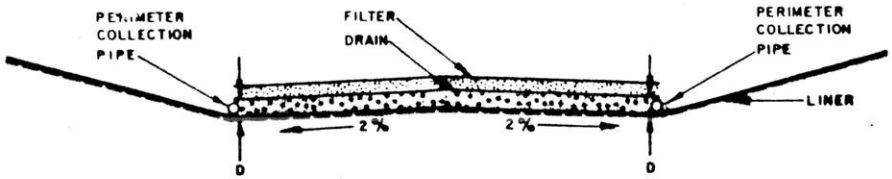
B.) HOR. BOTTOM UNDERDRAIN WITH PERIMETER COLLECTION



C.) SLOPING UNDERDRAIN WITH COLLECTION AT ONE END



D.) SLOPING UNDERDRAIN WITH PERIMETER COLLECTION



EQUATIONS: 1. $Q = kiA$
2. $Q_{in} = Q_{out}$

Q = flow
k = hydraulic conductivity
i = gradient
A = area

So: $Q_{in} = kiA$ (from the tailings, see Fig. 3.1b)
$= 0.40$ cfs

With: $Q_{in} = Q_{out}$ (through drain cross section)

A) $0.40 = kiA$
$0.40 = (1.6 \times 10^{-2})(D/1500)(15000)$
So: D = 5.0 ft. (1.5 m)

$k = 1.6 \times 10^{-2}$ ft/sec.
A = D (thickness) x 1500 ft.
i = D/1500

B) $0.40 = kiA$
$0.40 = (1.6 \times 10^{-2})(D/375)(6000D)$
So: D = 1.6 ft. (0.5 m)

$k = 1.6 \times 10^{-2}$ ft/sec.
A = D(thickness) (4 x 1500) ft.
i = D/L with L = average flow distance
$= (1500)^2 / 1500 \times 4$
= 375 ft. (114.3 m)

C) $0.40 = kiA$
$0.40 = (1.6 \times 10^{-2})(0.02)(1500\,D)$
So: D = 0.8 ft. (0.2 m)

$k = 1.6 \times 10^{-2}$ ft/sec.
A = D (thickness) x 1500
i = 0.02 (bottom slope)

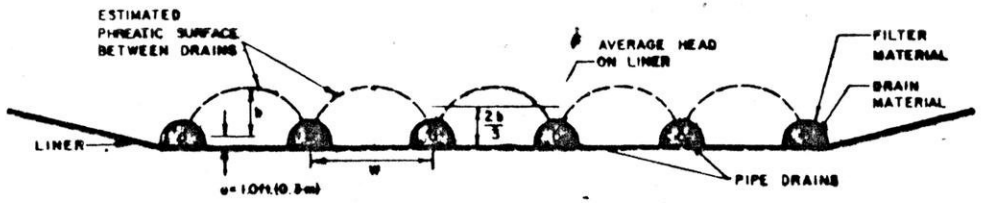
D) $0.40 = kiA$
$0.40 = (1.6 \times 10^{-2})(0.02)(6000\,D)$
So: D = 0.2 ft. (0.06m)

$k = 1.6 \times 10^{-2}$ ft/sec.
A = D (thickness) (4 x 1500)
i = 0.02 (bottom slope)

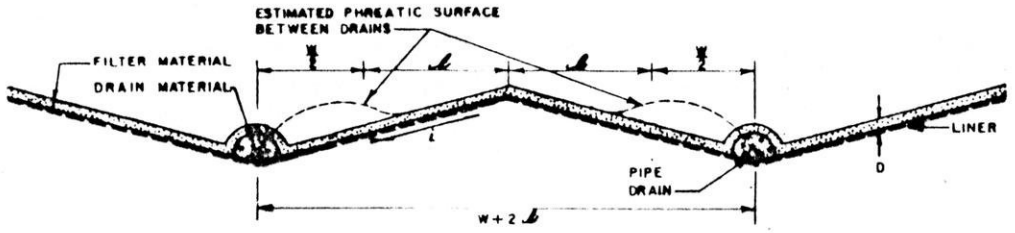
NOTE:
Hydraulic capacity analysis for side slopes shown on FIGURE 3.4

JOB NO. 786089	SCALE NOT TO SCALE	BOTTOM BLANKET THICKNESS FOR REQUIRED HYDRAULIC CAPACITY
DRAWN SKB	DATE 7-29-81	
CHECKED JFC	DWG. NO. ——	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 3.2

A) PIPE DRAIN OR LINE DRAIN SYSTEM



B) COMBINED PIPE UNDERDRAIN AND FILTER BLANKET SYSTEM



EQUATIONS: Q in = Q out

$Q = kiA$

$q = ki$

Q = flow
q = flow per unit area
k = hydraulic conductivity
i = gradient
A = area

A.) DRAIN SPACING, W, IS ESTIMATED BY:

$$W^2 = \frac{4k(h^2 - e^2)}{q_d}$$

$k = 1.6 \times 10^{-7}$ ft/sec. (tailings)

q_d = inflow from tailings per unit area
$= ki$
$= (1.6 \times 10^{-7})(1.1)$
$= 1.8 \times 10^{-7}$ cfs / ft^2

eg: MAXIMUM PHREATIC SURFACE AT SURFACE OF POND, b = 70 ft

$$W = \left[\frac{4(1.6 \times 10^{-7} \times 70^2 - 1^2)}{1.80 \times 10^{-7}}\right]^{\frac{1}{2}}$$

$= 132.0$ft (40.2 m)

So:

b	(ft)	2.0	6.0	10.0	22.0	28.0	41.0	55.0	70.0	83.0
	(m)	0.6	1.8	3.0	6.7	8.5	12.5	16.8	21.3	25.3
W	(ft)	3.3	11.2	18.8	41.4	52.8	77.3	103.7	132.0	156.5
	(m)	1.0	3.4	5.7	12.6	16.1	23.6	31.6	40.2	47.7

B.) ESTIMATED ADDITIONAL SPACE BETWEEN PIPE DRAINS WITH A 2.0 ft. THICK BLANKET OF FILTER MATERIAL OVER THE BOTTOM.

$$Q_{in} = Q_{out}$$

$$k_I i_I A_I = k_0 i_0 A_0$$

eg: D = 2 ft. and i = 25%

$$(1.6 \times 10^{-7})(1.1)(\ell) = (3.3 \times 10^{-5})(0.25)(2)$$

$$\ell = 93.8 \text{ft. } (28.6 \text{m})$$

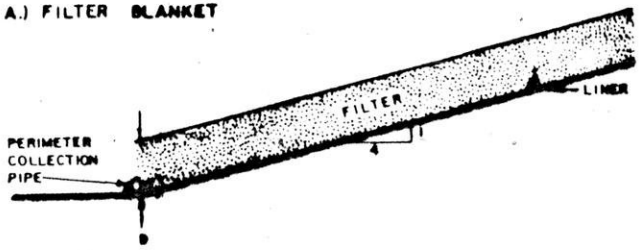
$k_I = 1.6 \times 10^{-7}$ ft/sec
$k_0 = 3.3 \times 10^{-5}$ ft/sec
$i_I = 1.1$
i_0 = bottom slope
$A_I = 1' \times \ell$
$A_0 = 1' \times D$

So:

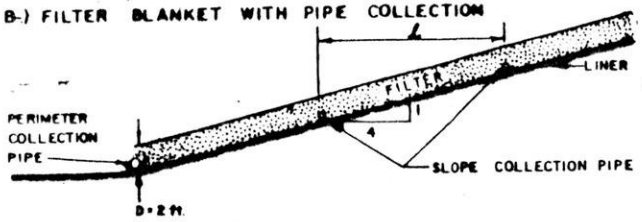
D (ft)	D (m)	i	ℓ (ft)	ℓ (m)
2.0	0.6	0.05	18.8	5.7
		0.10	37.5	11.4
		0.25	93.8	28.6
3.0	0.9	0.05	28.1	8.6
		0.10	56.3	17.2
		0.25	140.6	42.9
4.0	1.2	0.05	37.5	11.4
		0.10	75.0	22.9
		0.25	187.5	57.2
5.0	1.5	0.05	46.9	14.3
		0.10	93.8	28.6
		0.25	234.4	71.4

JOB NO. 786085	SCALE NOT TO SCALE	PIPE DRAIN SPACING FOR REQUIRED HYDRAULIC CAPACITY	
DRAWN SKB	DATE 7-20-81		
CHECKED	DWG. NO.		
Golder Associates		EXXON MINERALS COMPANY	FIGURE 3.3

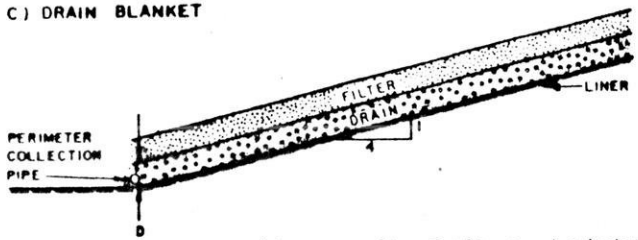
A.) FILTER BLANKET



B-) FILTER BLANKET WITH PIPE COLLECTION



C) DRAIN BLANKET



EQUATIONS: 1. $Q = kiA$ with: Q = flow; k = hydraulic conductivity
2. $Q_{in} = Q_{out}$ i = gradient; A = area

So: $Q_{in} = kiA$ (from the tailings, see Fig. 3.1b)
$= 0.36$ cfs

With: $Q_{in} = Q_{out}$ (through the drain cross section)

A) $0.36 = kiA$
$0.36 = (3.3 \times 10^{-5})(0.25)(6000\ D)$
So: $D = 7.3$ ft. (2.2 m)

$k = 3.3 \times 10^{-5}$ ft/sec.
$i = 0.25$ (side slope)
$A = D \times (4 \times 1500)$

B) for: $D = 2.0$ ft. (see combined pipe drain and filter blanket calculations for pond bottom, FIGURE 3.3b)
$= 0.6$ m
$\ell = 93.8$ ft (28.6m)

C) $0.36 = kiA$
$0.36 = (1.6 \times 10^{-2})(0.25)(6000D)$
So: $D = 0.015$ ft. (0.0045m)

$k = 1.6 \times 10^{-2}$ ft/sec.
$i = 0.25$ (side slope)
$A = D \times (4 \times 1500)$

NOTE: Flow through filter assumed vertical so its thickness is determined by construction practicality.

JOB NO. 786089	SCALE NOT TO SCALE	SLOPE BLANKET THICKNESS FOR REQUIRED HYDRAULIC CAPACITY
DRAWN SKB	DATE 7-27-81	
CHECKED JFC	DWG. NO. ———	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 3.4

March, 1982

As shown by the analyses, sloping the pond bottom reduces the thickness of the underdrain system by providing a gradient in addition to that provided by the water flow depth. A minimum 2% slope is considered suitable for a blanket drain system for ponds of the size anticipated for the Crandon Project. Steeper slopes were considered for the combined system.

For analyses purposes, the thickness of the blanket underdrains and blanket portions of combined and side slope underdrains were computed for the section of maximum flow. For blanket underdrains this is at the collection pipe location. For the side slope blanket underdrains it is assumed that there is a collection pipe at the base of the slope. For the combined pipe and blanket underdrains, the distance between drains was calculated based on flow depth limited by the selected thickness of blanket and bottom slope.

4.0 POND SEEPAGE ESTIMATES

Seepage estimates through a liner were made using the nominal 100 acre (40 ha) pond with the dimensions which were shown in Figure 3.1.a. Steady state seepage for operating ponds was estimated using Darcy's equation:

$$Q_s = kiA$$

with

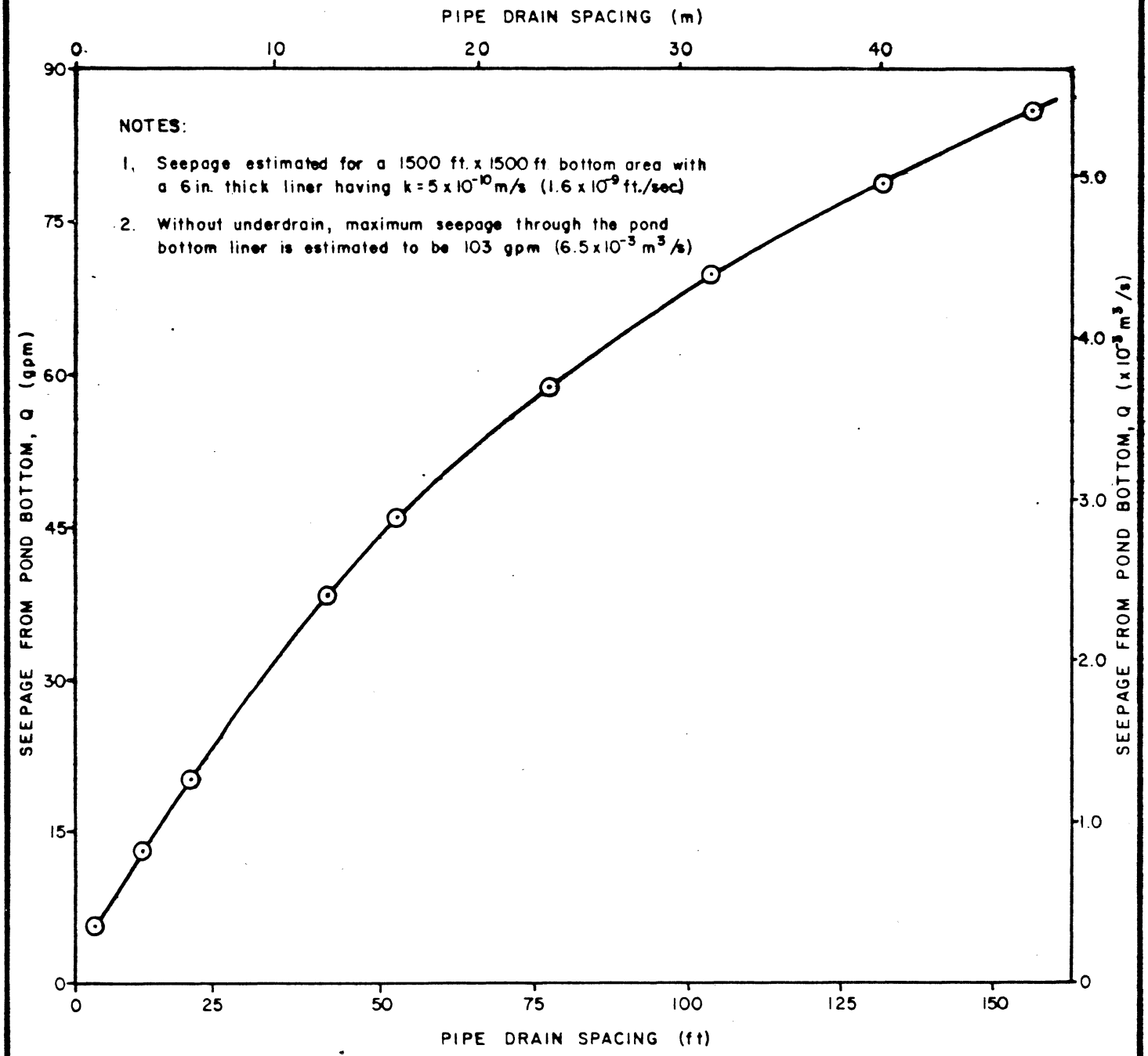
Q_s = the seepage through the liner (L^3/T)
k = the hydraulic conductivity of the liner (L/T)
A = the bottom or slope liner area (L^2)
i = the hydraulic gradient on the liner (L/L)

The hydraulic gradient with blanket drains was estimated using the head on the liner as being equal to the required hydraulic thickness of the drain. For pipe drain systems, 2/3 of the maximum head between the drains was used as an average head across the liner. The hydraulic gradient, i, was calculated as:

$$i = \frac{\text{head + liner thickness}}{\text{liner thickness}}$$

With only pipe drains, the estimated seepage from the pond bottom varies with pipe drain spacing. For the bottom portion of the nominal 100 acre (40 ha) pond, the estimated seepage for varying drain spacing is shown on Figure 4.1. As can be seen from this graph, very close pipe drain spacing is required to significantly reduce the estimated seepage.

Table 4.1 presents seepage estimates for the nominal 100 acre (40 ha) pond with various bottom and side slope



JOB NO. 786085	SCALE AS SHOWN	PIPE DRAIN SPACING vs SEEPAGE FROM POND BOTTOM
DRAWN SKB	DATE 1-6-81	
CHECKED	DWG NO. —	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 4.1

GAF DRAFTING MEDIA

TABLE 4.1
ESTIMATED SEEPAGE THROUGH LINER FOR VARIOUS UNDERDRAIN CONFIGURATIONS

Underdrain Configuration	Seepage Through	Hydraulic Thickness ft. (m)	Gradient	Seepage		
				(10^{-2} cfs)	(gpm)	(10^{-3} m^3/s)
Fig. 3.2 (A): Hor. bottom blanket w/one end collection	Bottom	5.0 (1.5)	11.0	4.0	18	1.1
Fig. 3.2 (B): Hor. bottom blanket w/perim. collection	Bottom	1.6 (0.5)	4.2	1.5	6.8	0.43
Fig. 3.2 (C): Bottom blanket @ 2% slope, one end collection	Bottom	0.8 (0.2)	2.6	0.94	4.2	0.27
Fig. 3.2 (D): Bottom blanket @ 2% slope, perim. collection	Bottom	0.2 (0.06)	1.4	0.50	2.2	0.14
Fig. 3.3 (A): Pipe drain @ 132 ft. (40.2 m) centers	Bottom	46.7 (14.2)	1.0	18*	81*	5.1*
Fig. 3.3 (B): Pipe drain @ 320 ft. (97.5 m) centers, bottom blanket @ 4.0 hor.:1.0 ver. slope	Bottom	2.0 (0.6)	5.0	1.8	8.1	0.51
Fig. 3.4 (A): Filter blanket	Slope	7.3 (2.2)	15.6	5.1	23	1.4
Fig. 3.4 (B): Filter blanket w/pipe collection	Slope	2.0 (0.6)	5.0	1.6	7.4	0.46
Fig. 3.4 (C): Filter and drain blankets	Slope	0.015 (0.0045)	1.0	0.33	1.5	0.09
Liner without underdrain	Bottom	70.0 (21.3)	1.1	23*	103*	6.5*
Liner without underdrain	Slope	35.0 (10.7)	1.1	15*	67*	4.3*

Notes:
Bottom Area = 2.25×10^6 ft.^{2} (2.09×10^5 m^2), Slope Area = 2.05×10^6 ft.^{2} (1.90×10^5 m^2)
Liner Thickness = 6 inches (152 mm), Liner Permeability = 1.6×10^{-9} ft./sec. (5.0×10^{-10} m/s)

*Seepage, $Q = \dfrac{H_t + H_l}{(H_t/k_t) + (H_l/K_l)}$ iA, where, t and l denote tailings and liner respectively, and H denotes head.

736085

underdrain configurations previously shown on Figures 3.2, 3.3, and 3.4. For comparison purposes, the pipe drain and combined drain systems are shown in Table 4.1 with the pipe drains spaced at 132 foot (40.2 m) centers. Also included in Table 4.1 are seepage estimates for the lined pond bottom and side slopes without an underdrain for comparison.

5.0 CONCLUSIONS

5.1 Seepage Reduction

The underdrain analyses show that dramatic reductions in tailings pond seepage during operation can be obtained with the installation of an underdrain system. The effect of the underdrain system is to reduce the head of water acting on the liner and hence reduce this gradient across the liner. For a given pond size and liner type, the gradient controls the seepage. Changes in gradient can be attained by either making the liner thicker or reducing the head on the liner. To reduce the gradient between one and two orders of magnitude the thickness of the liner would have to be increased by 10 to 100 times; an increase from say 6 inches (152 mm) to between 5 and 50 feet (1.5 m to 15 m). A similar reduction of about one to two orders of magnitude in gradient can be achieved with a blanket underdrain system.

5.2 Two Layer Blanket Underdrain

The conceptual analyses for a nominal 100 acre (40 ha) pond for various underdrain systems indicate that a two layer blanket underdrain is the most feasible system. A single layer of filter material does not have sufficient hydraulic capacity to transmit the seepage using a reasonable material thickness, say less than 5 feet (1.52 m). A coarser material with sufficient hydraulic capacity does not have enough fines to prevent migration of tailings into the drain, which would then clog the drain. Therefore, a coarse bottom drain layer with a filter material layer between it and the tailings appears to be the most functional system. An additional layer of material may be needed to retard short circuiting of large volumes of clarified or unclarified ponded water directly into the drain layer.

5.3 Bottom Slope

The thickness of the drain material can be reduced by sloping the underdrain system. Because of the relatively large aerial extents of the ponds and their relatively shallow depths, steep slopes are not practical. Very flat slopes, 1% or less, are extremely difficult to control over such a large area. A two percent bottom slope is considered a minimum, and it provides an adequate margin for excess hydraulic capacity of the drain system. Somewhat steeper slopes, say 5 to 10 percent may be employed depending on the final pond configuration.

5.4 Pipe Drain and Combined Drain Alternatives

A series of parallel or branching pipe line drains do not seem practical. Pipe drains alone would have to be spaced at about 20 foot (6.1 m) centers to reduce the seepage flow to even 1/5 of the estimated seepage without an underdrain across the same liner. These drains would require a large amount of pipe and drain/filter materials; approximately 22 miles (35 km) of pipe for a nominal 100 acre (40 ha) pond.

Modificiation of the pipe drain system by connecting the drains with filter material (a blanket) would allow widening the drain spacing, but with the added expense of substantial bottom grading. Significant seepage reduction could be attained with a 2 foot (0.61 m) thick layer of filter material including drain pipes spaced at approximately 300 foot (91.4 m) centers and with the filter blanket placed at 4 horizontal to 1 vertical slopes between the drains. Cutting a pond bottom into such a series of ridges would be a more difficult grading job than preparing a flat bottom slope.

If a layer of coarse drain material was added below the filter material in a combined system, it would reduce the underdrain thickness and slopes to the extent that only one pipe down the center would be needed. This, however, is essentially the two layer blanket drain system.

5.5 Inside Embankment Slopes

Alternatives were investigated for draining the 4 horizontal to 1 vertical side slopes. Filter material alone would have to be placed in a fairly thick layer, about 7 feet (2.1 m), to achieve sufficient hydraulic capacity. A modified pipe drain system with pipes placed parallel to the crest on about 90 foot (27.4 m) centers with filter material 2 feet (0.6 m) thick along the entire slope could be used. But, without a similar system on the bottom, there is little reason to install such a complicated system on the slopes. Again, a two layer blanket underdrain system seems most feasible.

5.6 Collection System

Significant reductions in blanket drain material thickness can be achieved by placing the collection system around the perimeter of the pond bottom. This collection scheme will also directly collect the drainage from the side slope so that it does not have to flow through the bottom blanket drain. Finally, it can allow collection of water from the entire pond at any of several discharge points.

6.0 RECOMMENDED UNDERDRAIN SYSTEM

6.1 Description

The recommended underdrain system based on these analyses is a two layer system covering the bottom and side slope areas. The pond bottom, with liner, would be graded generally from the center of the pond down toward the toe of the embankment at a minimum 2% slope.

The drain material would be placed over the liner. The hydraulic capacity of the drain material is so great that only a nominal thickness of less than 6 inches (152 mm) is required in most areas. However, placement considerations on such a large scale dicatate that a drain thickness on the order of 12 inches ±3 inches (305 mm ± 76 mm) may be required.

The drain would be placed without damage to the liner by having the construction equipment working on the previously placed lift of drain and not on the liner. This is a common placement technique when working over liners.

The filter material would be placed over the drain, and tailings will cover it. The filter material is recommended to be on the order of 12 inches ±3 inches (305 mm ± 76 mm) thick on the pond bottom and 6 inches ±2 inches (152 mm ± 51 mm) thick on the side slopes, also based on placement considerations. Both the filter and drain materials need only receive the compaction that occurs during placement. Descriptions of the design for these materials were given in Section 2 and their approximate gradations were shown on Figures 2.1 and 2.2. In order to prevent large volumes of free pond water from entering the underdrain too quickly and to filter turbid free water which does enter, a layer of

till, waste rock, or tailings may have to be placed over the filter layer, particularly on the side slopes. If till is used as the filter layer, it would perform both functions.

A collection pipe should be placed around the inside toe of the side slope to collect water from the blanket drain and to conduct this water to the discharge station. A two foot diameter (0.6 m) perforated pipe, laid flat, can conduct 2 to 3 times the estimated flow. Such a collection pipe system has the advantage of being able to conduct all the water reaching the drain to any one (or more) discharge stations, regardless of their location around the perimeter of the pond bottom. Alternatively, the collection pipes could be smaller and sloped to the discharge point. Specifics of the system may be determined during the design phase of the project.

Discharge stations are sump areas from which the underdrain flow will be pumped out of the pond. Multiple stations should be considered to allow backup in case of sump, station access, or collection pipe failure. The unpumped stations could be used to monitor the water levels in the drain in order to evaluate the performance of the underdrain system. If necessary, these extra stations could also be pumped. Should the entire collection pipe collapse the underdrain system could be pumped from the discharge stations like a confined aquifer system. For this reason, discharge stations should include a section of perforated pipe in the sump.

6.2 Efficiency

The efficiency of the proposed underdrain system was reviewed for a nominal 100 acre (40 ha) pond with a storage depth of about 70 feet (21.3 m). Two approaches to effi-

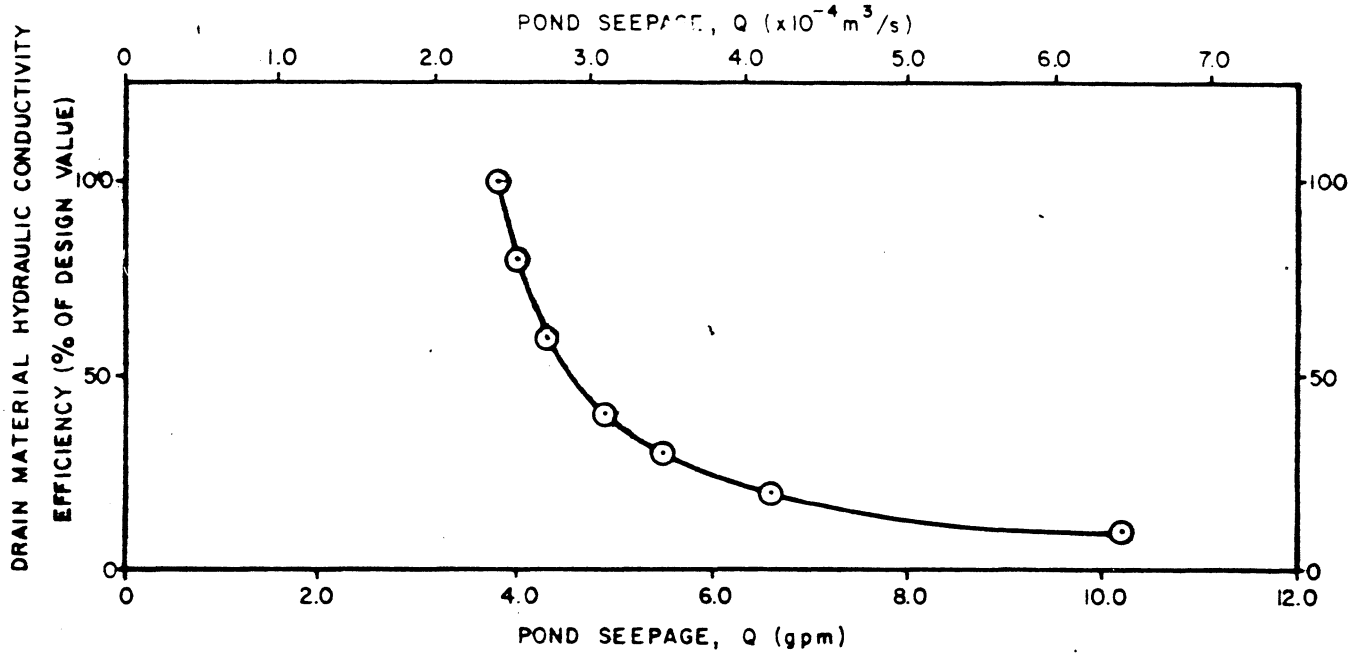
ciency were considered and related to seepage through a 6 inch (152 mm) thick pond liner. The seepage through the liner was estimated based on the head build up over the liner due to:

1. a gross reduction in permeability for the entire drain, and
2. total blinding (permeability reduced by clogging) of 100 foot by 100 foot (30.5 m by 30.5 m) areas of the drain.

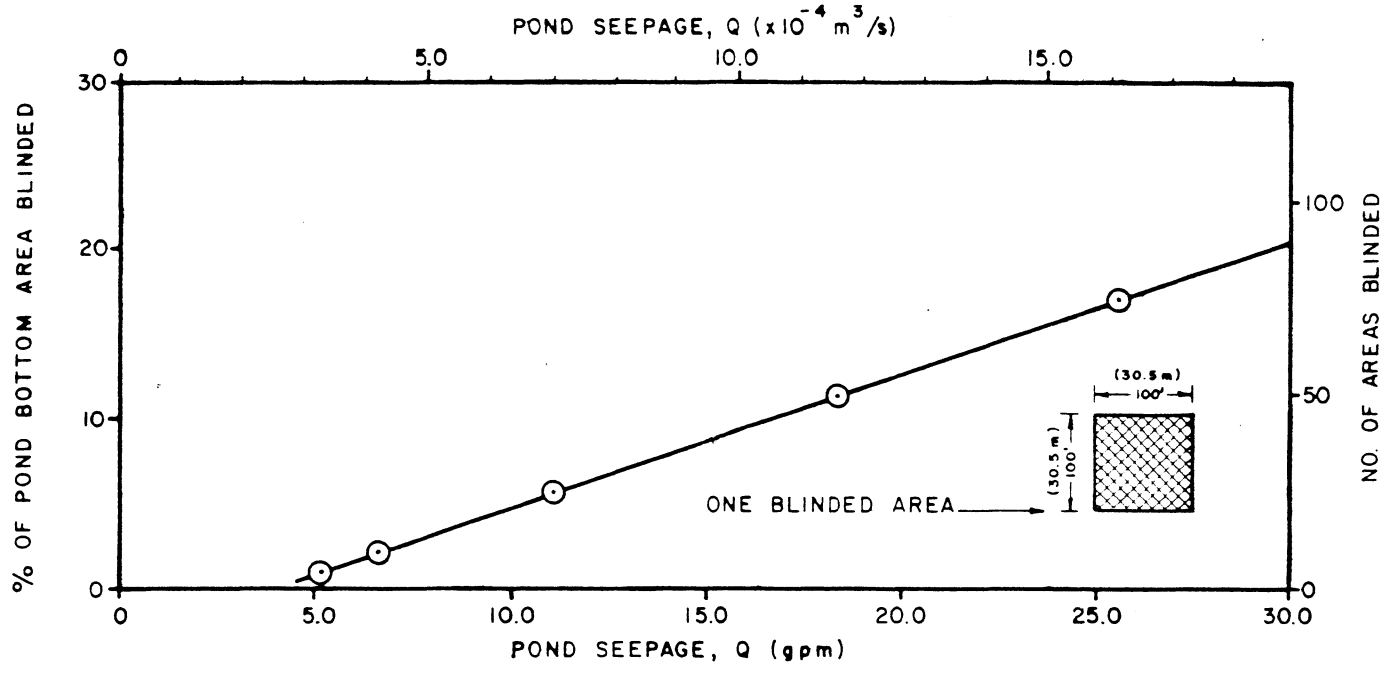
The gradients and hydraulic characteristics used were similar to those previously described in this report. The head build up over blind areas was assumed similar to that occurring between pipe drains 100 feet (30.5 m) apart. The plots of drain efficiency versus seepage are shown in Figure 6.1.

As indicated by Figure 6.1, there is a great deal of excess hydraulic capacity in the underdrain system. If the efficiency of the hydraulic conductivity of the drain is decreased to only 10% of its design value (one order of magnitude lower in permeability), the amount of seepage increases from 4 gpm (2.5×10^{-4} m^3/s) to approximately 10 gpm (6.3×10^{-4} m^3/s) for a nominal 100 acre (40 ha) pond. The effect of blind areas is more severe because of the assumed increase in head on the liner in the blind areas. However, even if 20% of the pond bottom area is covered by 100 foot by 100 foot (30.5 m x 30.5 m) blind areas the seepage through the liner is estimated to increase to approximately 30 gpm (1.9×10^{-3} m^3/s). Given such margin of efficiency in the underdrain system and considering that the drain materials, and possibly the filter materials, will likely be processed materials and will be placed under controlled con-

SEEPAGE THROUGH LINER vs. DRAIN HYDRAULIC CONDUCTIVITY EFFICIENCY



SEEPAGE THROUGH LINER vs. NUMBER OF 100ft.x 100ft. AREAS BLINDED



NOTES:

FIGURES ARE BASED ON:

1. A nominal 100 acre (39.4 hectares), 70 ft. (21.3 m) deep pond as shown on FIGURE 3.1
2. 6 in. (152 mm) thick liner with $k = 5 \times 10^{-10}$ m/s (1.6×10^{-9} ft./sec.)

JOB NO. 786085	SCALE AS SHOWN	FIGURE SEEPAGE vs. EFFICIENCY OF UNDERDRAIN SYSTEM
DRAWN SKB	DATE 1-6-81	
CHECKED	DWG NO. —	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 6.1

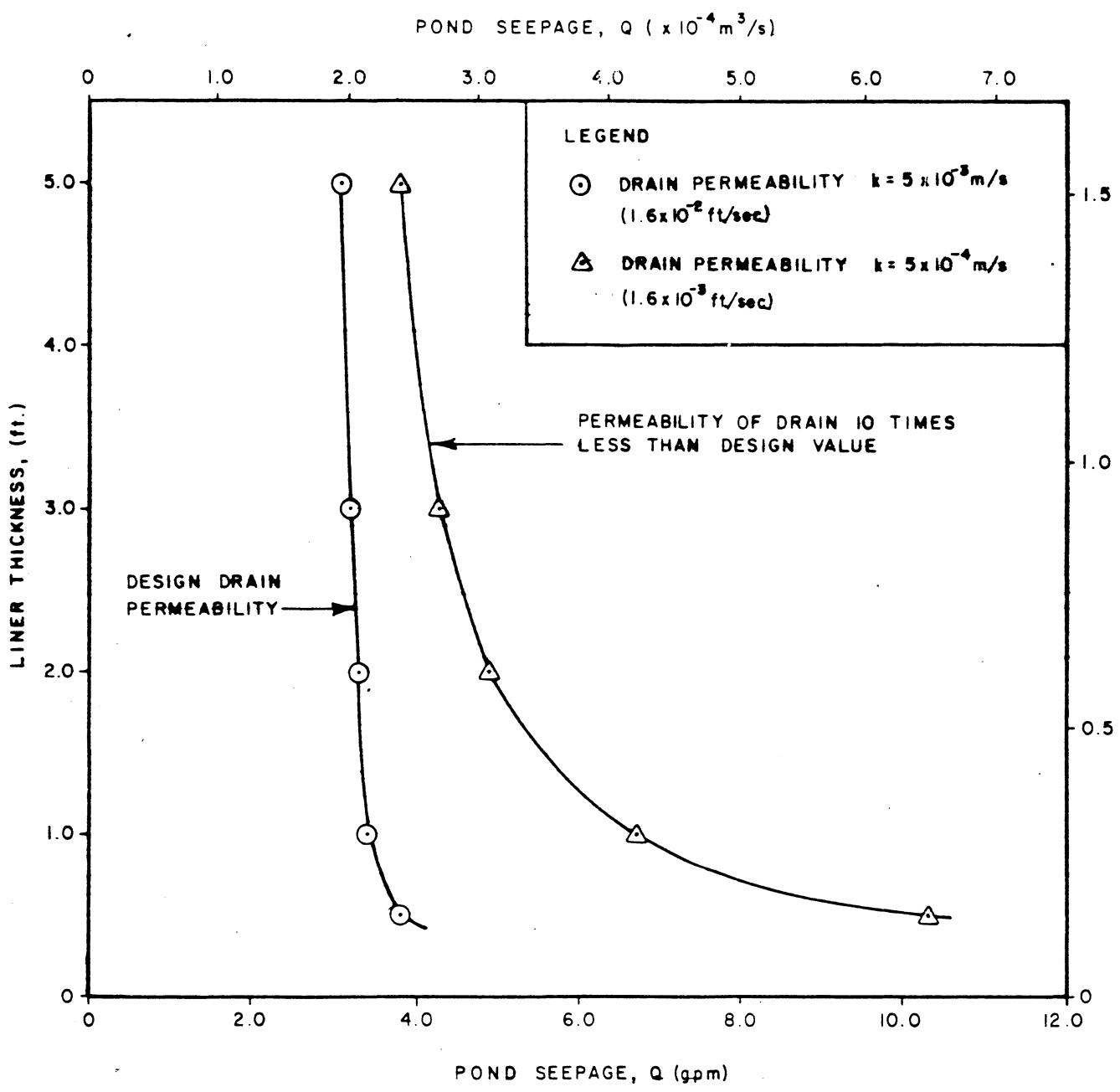
GAF DRAFTING MEDIA

ditions the underdrain system should certainly operate successfully in reducing the seepage through the liner.

The combined effect of liner thickness and drain layer permeability on pond seepage are presented graphically on Figure 6.2. For the design drain permeability of $5x10^{-3}$ m/s ($1.6x10^{-2}$ ft./sec.) the rate of seepage is nearly equal for liner thickness varying from 6 inches (152 mm) to 5 feet (1.5 m), less than 4 gallons per minute ($2.5x10^{-4}$ m^3/s). If the drain layer permeability was one order of magnitude less than that used in the design, the resulting seepage quantity would range from approximately 10 gallons per minute ($6.3x10^{-4}$ m^3/s) with a 6 inch (152 mm) thick liner to approximately 4 gallons per minute ($2.5x10^{-4}$ m^3/s) with a 5 foot (1.5 m) thick liner. Thus, for a wide range of drain permeability, the thickness of the liner does not substantially affect the rate of pond seepage. In all cases, the estimated seepage rates from the nominal 100 acre (40 ha), 70 foot (21.3 m) deep pond are considered to be very low.

Although not shown on Figure 6.2, the effect of increasing the drain layer permeability one order of magnitude higher than the design value will not significantly reduce the estimated pond seepage rate. For example, with a drain permeability of $5x10^{-2}$ m/s ($1.6x10^{-1}$ ft./sec.) and a 6 inch (152 mm) thick liner the estimated seepage rate is about 3 gallons per minute ($1.9x10^{-4}$ m^3/s).

The hydraulic capacity of the recommended system is also sufficient to drain the total volume of water being pumped into the pond with the tailings, about 690 gallons per minute ($4.3x10^{-2}$ m^3/s). Using the equation shown on Figure 3.2 part D and a drain material thickness of one foot (305 mm), the hydraulic capacity is 862 gallons per minute



NOTES:

1. A nominal 100 acre (40 hectare), 70 ft. (21.3 m) deep pond, as shown on FIGURE 3.1 is considered in the analyses.
2. Pond seepage quantities include seepage from both the pond bottom and side slopes.
3. Liner permeability $k = 5 \times 10^{-10}$ m/s (1.6×10^{-9} ft./sec)

JOB NO. 786085	SCALE AS SHOWN	EFFECT OF LINER THICKNESS ON SEEPAGE WITH UNDERDRAINS
DRAWN SKB	DATE 1-12-82	
CHECKED	DWG NO. ———	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 6.2

GAF DRAFTING MEDIA

(5.4×10^{-2} m^3/s); greater than the total volume of water input with the tailings plus the average annual retained precipitation. Additional hydraulic capacity is provided by the drain layer on the pond slopes. Using the equation shown on Figure 3.4 part C and a drain material thickness of twelve inches (305 mm), the hydraulic capacity is over 10,800 gallons per minute (0.68 m^3/s). The hydraulic capacity of the drain material on the slopes and pond bottom is over sixteen times the volume of water being pumped into the pond. Thus, there is a great deal of latitude available when final details are established to reduce the underdrain thickness, if adequate placement can be developed, without impacting the ability of the system to drain more water than is supplied.

6.3 Underdrain Materials Availability

Brief consideration has been given to processing waste rock to make the required gradation of filter and drain materials. Some of the factors associated with this approach are as follows:

1. There may be insufficient quantities of waste rock available.
2. The waste rock may not be available at the time needed.
3. The waste rock will probably need three stages of crushing with washing and screening to produce the needed materials.
4. The waste rock may contain sulfide bearing minerals which could produce acidic conditions in the washing operation.
5. The waste rock may be better used for slope protection on the inside pond side slopes.

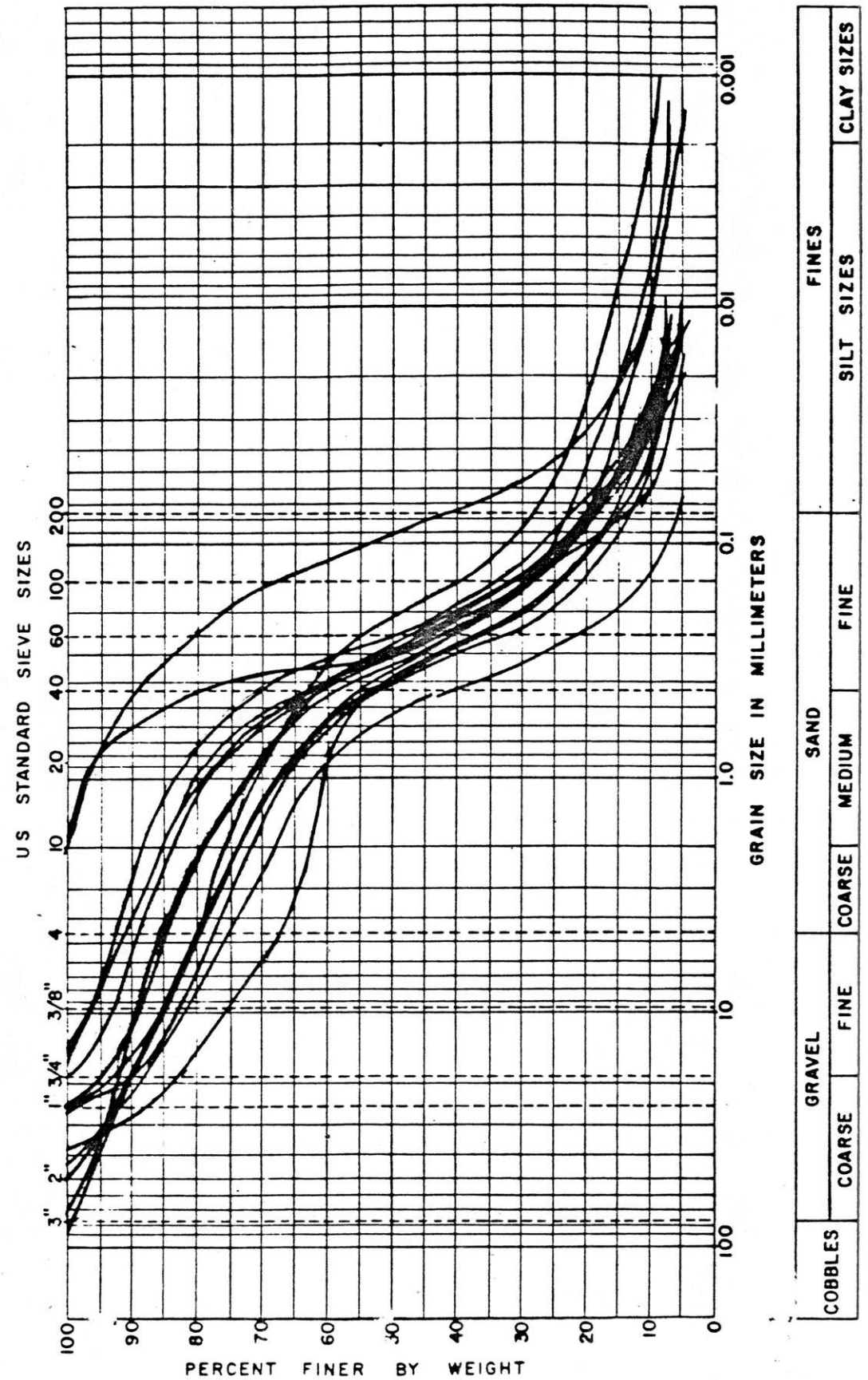
Based on the above, it is recommended that the waste rock not be used to process filter or drain materials. However, it could be used as a protective cover over the underdrain system where it may not need to be crushed, washed, and screened.

Comparison of the filter and drain material bands proposed in Section 2 of this report (Figures 2.1 and 2.2) to the composite gradation curves of the glacial till soils at the proposed disposal sites, Figures 6.3 and 6.4, suggests that these soils could be used to develop the filter and drain materials. These soils will have to be processed by screening, washing and perhaps some crushing to provide the desired gradations. Such a procedure is technically feasible with available technology. However, the costs associated with such an operation may be high because of the amount of fines (material finer than the #200 sieve) in some of the glacial soils. Also, we are not aware of any mine or commercial aggregate suppliers processing glacial till for graded aggregates on a large scale. Prior to a final decision on processing the glacial soils for filter and drain materials, we recommend additional investigation into practices of commercial aggregate suppliers in the Crandon area and perhaps performing a pilot processing trial by shipping on-site soils to an operating plant.

As discussed in Section 2.2, native till could also be used as a filter material without processing. However, since the range of till grain sizes is slightly wider than the desired filter material gradation, the thickness of the underlying drain, and perhaps the till filter, may have to be greater than for processed filter material because there may be some slight migration of till fines into the drain. This approach is technically feasible, but a cost comparison must be factored into the final decision.

GRAIN SIZE DISTRIBUTION

FIGURE 6.3



BORING NO.	ELEV. OR DEPTH	W_n	W_L	W_p	I_p	DESCRIPTION OR CLASSIFICATION
-	-	-	-	-	-	Samples of Till from Site 40 (Reference 8)

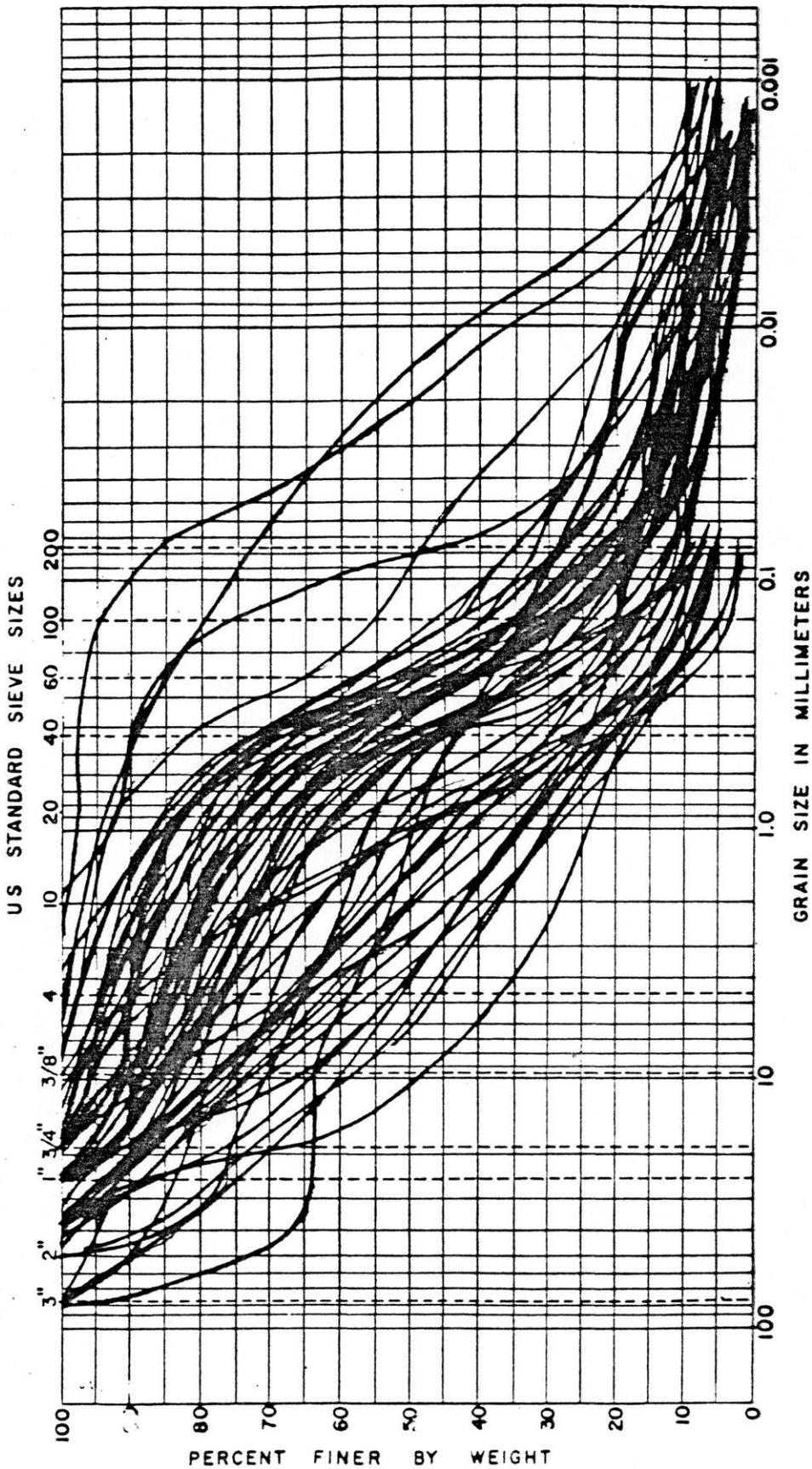
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Job No 786085

Golder Associates

Drawn
Checked
Reviewed

GRAIN SIZE DISTRIBUTION

FIGURE 6.4



BORING NO.	ELEV. OR DEPTH	W_n	W_L	W_p	I_p	DESCRIPTION OR CLASSIFICATION
-	-	-	-	-	-	Samples of Till from Site 41 (Reference 8)

Date 10-7-81
Job No. 786085

Golder Associates

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Checked
Reviewed

7.0 SUMMARY

The addition of an underdrain system above the liner of the tailings ponds at the Crandon Project is a feasible approach for reducing the amount of seepage from the ponds during operation. A two layer blanket underdrain over the pond bottom and side slopes with a perimeter pipe collection system is recommended over other options considered. The underdrain system can be designed and constructed to allow passage of water and retain the tailings materials. The recommended underdrain has sufficient hydraulic capacity to provide a wide margin of efficiency while limiting seepage through the liner. For a nominal 100 acre (40 ha), 70 foot (21.3 m) deep tailings pond, the underdrain system is estimated to reduce the seepage through the liner to less than 4.0 gpm (2.5×10^{-4} m^3/s) compared to an estimated seepage of approximately 170 gpm (107×10^{-4} m^3/s) for the same liner without an underdrain system. The use of an underdrain system to significantly reduce pond seepage is believed to be feasible both technically and from a construction standpoint for the Crandon Project.

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GHC:JFC:dap

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8. Golder Associates, "Geotechnical Review, Crandon Project Waste Disposal System, Project Report 2", October 1981.

CRANDON PROJECT

MINE WASTE DISPOSAL FACILITY

SYSTEM 41-114B

STATE DOCUMENTS
DEPOSITORY

SEP 17 1984

University of Wisconsin, LRC
Stevens Point, Wisconsin

TAILING PONDS WATER CLARIFICATION POOLS

Exxon Minerals Company

C. C. Schroeder

February 8, 1983, Rev. 0

March 22, 1983, Rev. 1

MINE WASTE DISPOSAL FACILITY

SYSTEM 41-114B

TAILINGS PONDS WATER CLARIFICATION POOLS

INTRODUCTION

Presented in this paper is an analysis of the potential for water to pool in the tailing ponds. Because of the underdrain system design which is continuous across the pond bottom and up the side slopes, there may be too much exposed filter and underdrain for water to pool in the ponds. Golder Associates, based on their knowledge and experience, believes there will be water pooling in the ponds; however, their preliminary engineering work did not attempt to analyze or quantify these conditions.

SYSTEM DESIGN AND OPERATION

Each of the four tailing ponds in the Crandon Project mine waste disposal facility (system 41-114B) is similarily designed according to the typical cross section shown in Figure 1. The details of the liner, underdrain, and filter system that cover the bottom and inside faces of the embankments for all ponds are shown in larger scale in Figure 2.

While the bottom section is identical for all ponds there are minor differences in the side slope cross sections, including:

1. A layer of rock slope protection is included on the sides where ponded water is anticipated, whereas on the tailings input side(s) the rock is omitted.

2. The filter layer is thickened on the ponded water sides (in tailing pond T1 since there is a surplus of waste rock available at the time of its construction, the waste rock layer is thickened and the filter layer thickness is not changed).

Each of the ponds has two or three benches along the side slope length to allow vertical staging of the construction of the liner and underdrain and filter system if desired.

The purpose of the liner, underdrain, and filter system is to reduce seepage from the tailing ponds to minimal levels. This system is required over the pond bottom and the pond sides, which by the time the pond is filled, have substantially more area exposed to the tailings than the pond bottom. The underdrain conducts seepage to a perimeter pipe system at a low point on the inside embankment toe where it is removed from the pond. With the underdrain functioning only a negligible depth of water is maintained on the liner and seepage through the liner is greatly reduced. The filter layer between the tailings and the underdrain prevents any migration of tailings into the underdrain layer.

In the operation of the MWDF, thickened tailings are discharged on one or two sides of the tailing ponds where it is anticipated beaches with slopes in the

range of 0.5 to 1.0 percent will develop. With tailings input at the locations shown, the estimated final tailings surface is shown in Figure 3.

Based on Golder Associates' experience, they suggest a water pool will develop in the tailing ponds. They expect the pooled water area would be approximately 20 percent of the pond area and the water pool would remain at about a 6.1 m (20 foot) depth. They further suggest that the bottom of the pooled water area would be essentially horizontal except for the area at the waters edge which would have an approximate 10 percent slope. In estimating these conditions, Golder Associates has reviewed tailing ponds that they consider to be similar to the proposed MWDF (system 41-114B). While the ponds do not employ similar underdrains, they are typically high leak rate facilities that would behave like the proposed system.

DEVELOPMENT OF WATER CLARIFICATION POOL

Because of the extent of exposed filter and underdrain, there would be no chance of any water pooling when a tailing pond is first put in use, until at least the pond bottom is covered with tailings. Based on average tails production this would require 2-3 months. However, after the bottom is covered with tailings, the area of exposed underdrain and filter is greatly reduced. In this situation, the filter layer will act to restrain seepage from the system and the typical condition for a tailing pond will be similar to Figure 4.

The figure shows the relationship between an assumed depth of ponded water and the slope length exposed for seepage. On a total area exposed basis, taking the typical length of the potential ponded water area around the embankment face to be about 500 m, the total seepage area is approximately 2,000 d (with the pooled water depth d in meters).

Following calculations presented in the appendix, using a Q = KiA relationship with average head on the filter being d/2 and Q taken from the anticipated pond water balance, a depth of water can be determined based on the permeability of the filter layer. With the above method of analysis the following K, d, and surface decant pumping relationship is obtained:

K of Filter	d of Pooled Water	Rate of Surface Decant Pumping
1×10^{-6} m/s	3.3 m (11 feet)	0
5×10^{-7} m/s	5.0 m (16.4 feet)	0
1×10^{-7} m/s	5.0 m (16.4 feet)	0.014 m^3/s (219 gpm)
5×10^{-8} m/s	5.0 m (16.4 feet)	0.016 m^3/s (247 gpm)

The pooled water analysis shows that the permeability of the filter must be about 1×10^{-6} m/s (3.3×10^{-6} feet per second) before any significant ponded water depth would develop. Also, the permeability would have to be below 5×10^{-7} m/s (1.64×10^{-7} feet per second) before an approximate 5.0 m (16.4 feet) depth of water would develop. Lower permeability values would result in greater ponding depth or would require decant pumping to maintain an approximate 5.0 m (16.4 feet) water pool depth.

The analysis indicates that ponded water development will be predictable and depend upon the filter permeability. The majority of the till materials, which are planned to be used for the filter, are estimated to have a permeability of less than 2×10^{-7} m/s when placed and compacted with normal efforts. In addition, there are other simple options available to assure that desired permeabilities are achieved such as special compactive effort or use of waste fines from underdrain material processing as a filter blend.

Once in operation, the system itself would probably be somewhat self adjusting as far as permeability is concerned. Areas of the filter that do seep faster because of higher permeabilities would likely experience some surface choking from any of the finest tailings fraction still suspended in the decant water.

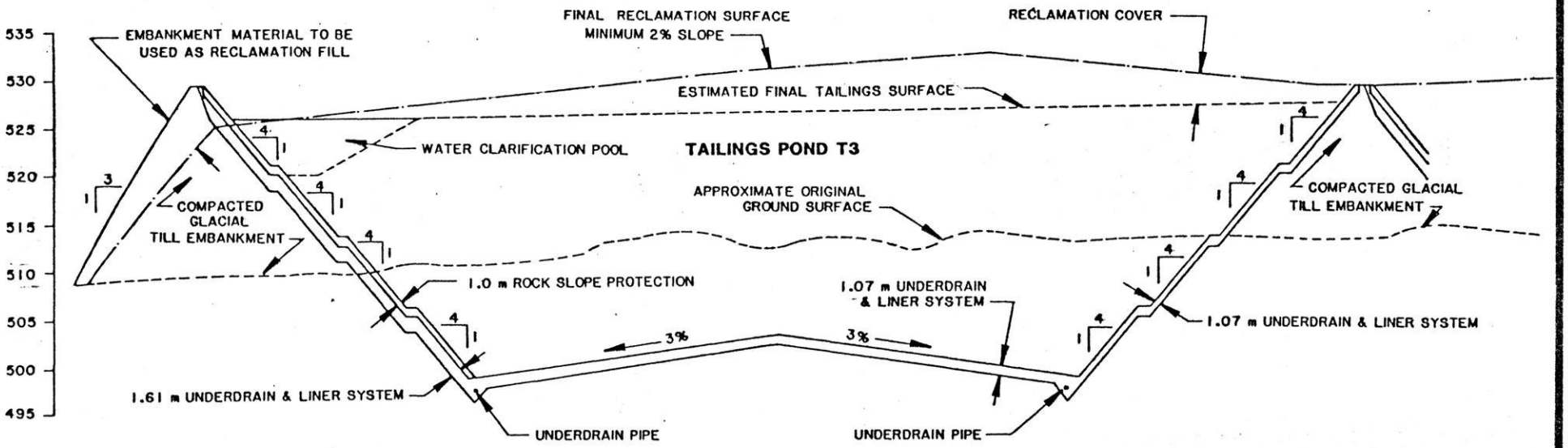
Finally, there is also opportunity to reduce the lateral extent of the exposed seepage area by movement of the tailings discharge lines around the pond perimeter. Thus the 500 m length used in the analysis can be field adjusted. Any reduction of exposed area by these means would also increase ponded water depth.

OPERATING FLEXIBILITY

While the above analysis does indicate that pooled water conditions will develop in the tailing ponds, the operation of the waste disposal facility is not dependent on maintaining a pool of water in the tailing pond. While normally a pooled water area would be maintained in the tailing pond for water

clarification, the filter layer will perform the same function in the proposed MWDF. With the rather limited water volumes entering the tailing pond, because of the thickened tailings, both the surface water decant pumping system and the underdrain pumping system can be sized to easily handle the entire tailing water volume with little size penalty in the equipment.

Regardless of tailing water decant requirements, a decant system for storm water removal would be included in the MWDF. Detailed requirements for the system based on storm water handling capacities have not been developed, but if storm water volumes such as the probable maximum precipitation (PMP) event have to be removed from the pond in a two or three week period, they will govern the sizing of the decant system.



0 100 200 300 400

HORIZONTAL SCALE IN FEET

0 25 50 75 100 125

HORIZONTAL SCALE IN METERS

VERTICAL SCALE EXAGGERATED 5x

EXXON MINERALS COMPANY

CRANDON PROJECT

TITLE

TYPICAL TAILINGS POND CROSS SECTION

SCALE	STATE	COUNTY
AS SHOWN	WISCONSIN	FOREST

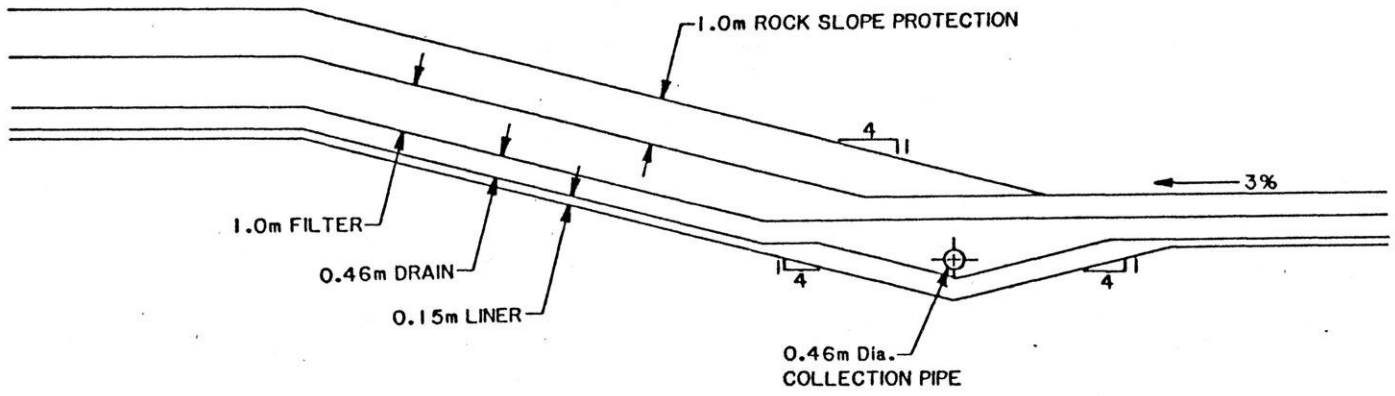
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S. J. HARVEY	1/83		
APPROVED BY	DATE	APPROVED BY	DATE
APPROVED BY	DATE	EXXON	DATE

DRAWING NO

FIGURE NO. 1

SHEET ____ OF ____

REVISION NO



0 1 2 3 4 5 6

SCALE IN METERS

EXXON MINERALS COMPANY

CRANDON PROJECT

TITLE

LINER, UNDERDRAIN, & FILTER DETAIL

SCALE	STATE	COUNTY
SHOWN	WISCONSIN	FOREST

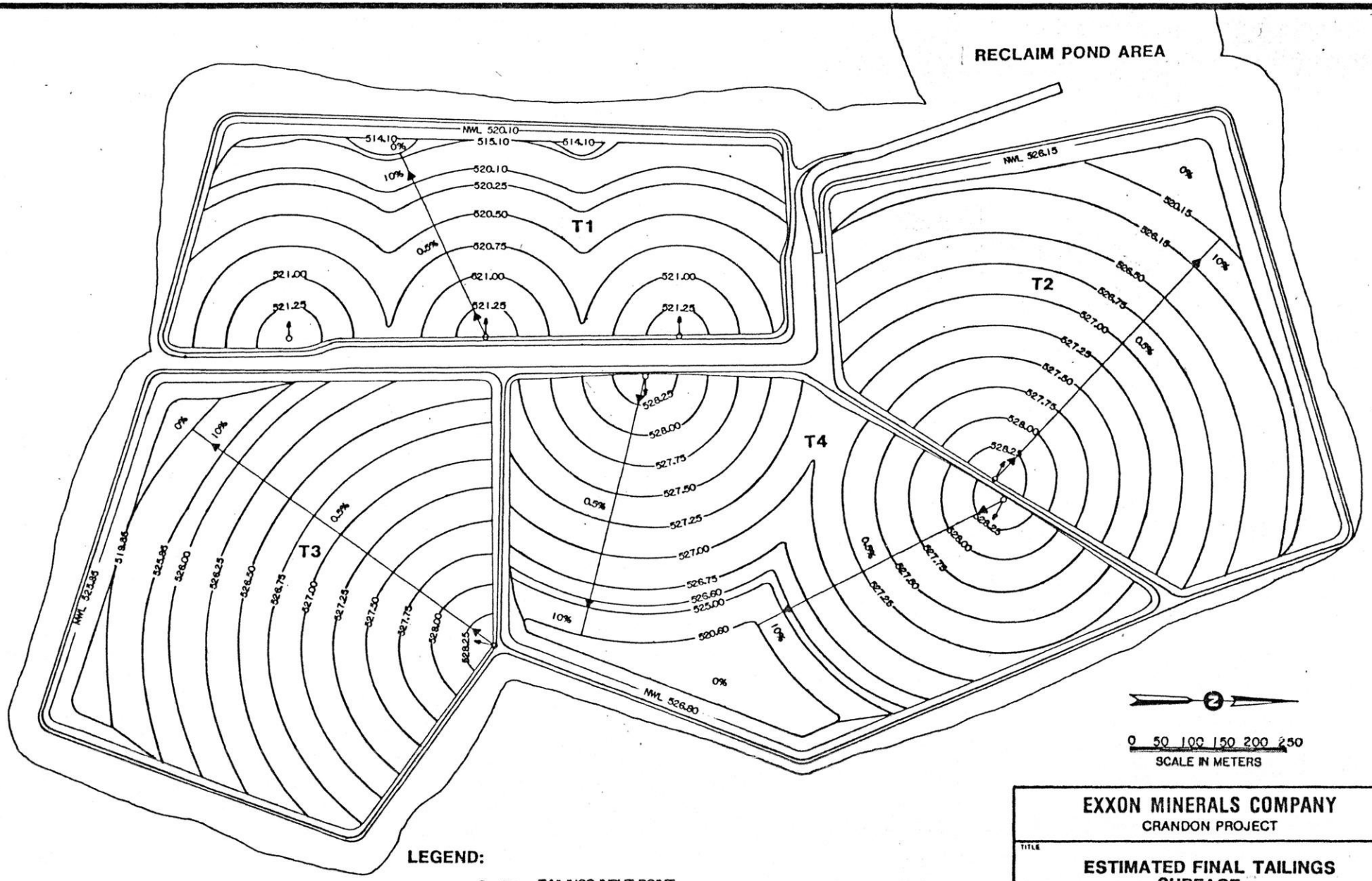
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DR SPRINGBORN	1783		
APPROVED BY	DATE	APPROVED BY	DATE
APPROVED BY	DATE	EXXON	DATE

DRAWING NO

FIGURE NO. 2

SHEET ___ OF ___

REVISION NO



LEGEND:

- o→ TAILINGS INPUT POINT
- 0% TAILINGS SURFACE SLOPE
- —NWL— NORMAL FULL POND WATER LEVEL

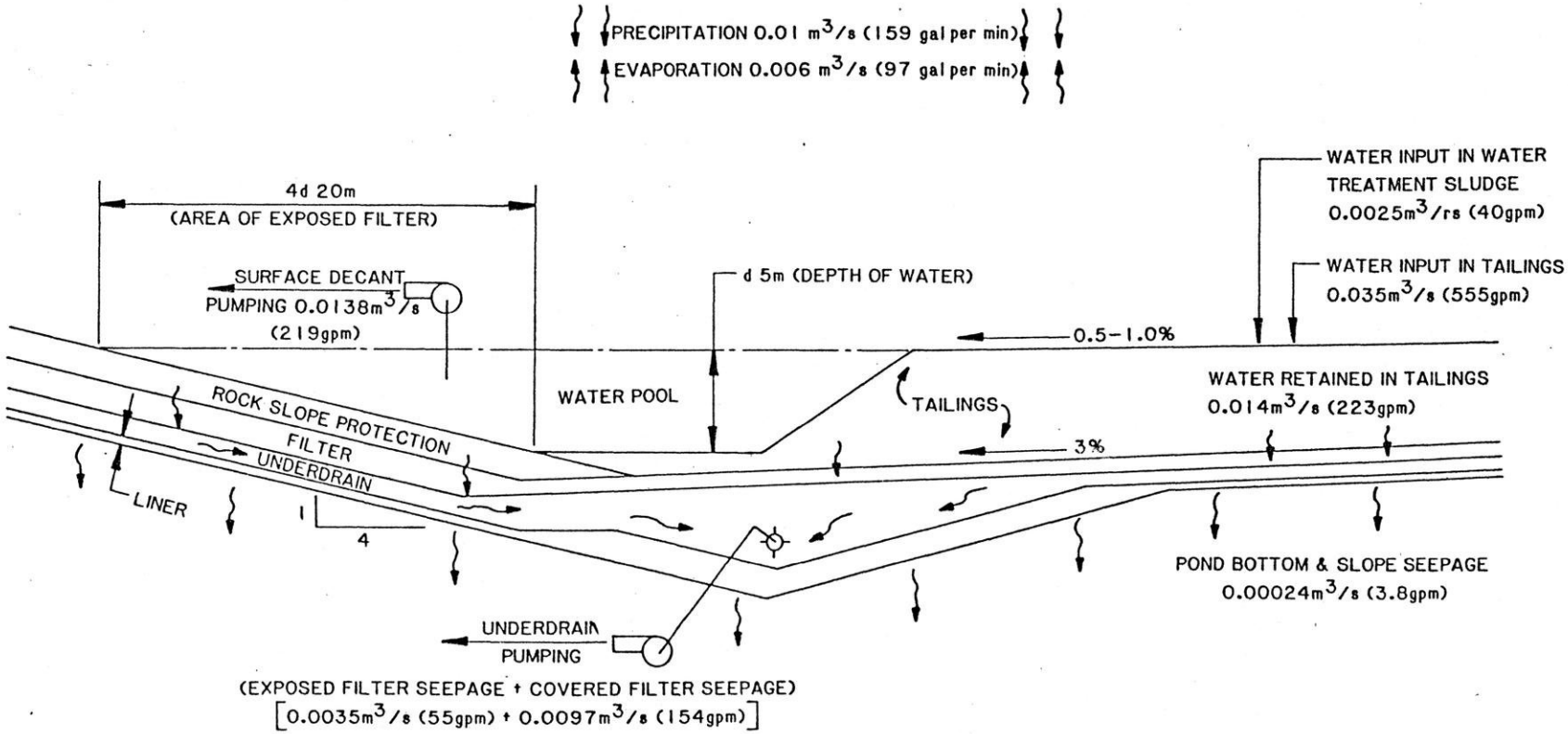
EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE
ESTIMATED FINAL TAILINGS SURFACE

SCALE	STATE	COUNTY
SHOWN	WISCONSIN	FOREST

DRAWN BY	DATE	CHECKED BY	DATE
DR SPRINGBORN	1/83		
APPROVED BY	DATE	APPROVED BY	DATE
APPROVED BY	DATE	EXXON	DATE

DRAWING NO **FIGURE NO. 3** SHEET ____ OF ____ REVISION NO



NOTES:

1) Typical pond water balance maintains
   Q Decant + Q Underdrain = Q Tails + Q Sludge + Q Prec. -
   Q Evap. - Q Seepage - Q Retained

2) Typical pond water balance during operation based on pond T3 area.

3) Precipitation 0.78 m/y (30.77 inches per year).

4) Evaporation 0.59 m/y (22.56 inches per year).
   Over 80 percent of pond area

5) Conditions shown at point in time when pond bottom has just
   been covered with tailings.

EXXON MINERALS COMPANY
CRANDON PROJECT

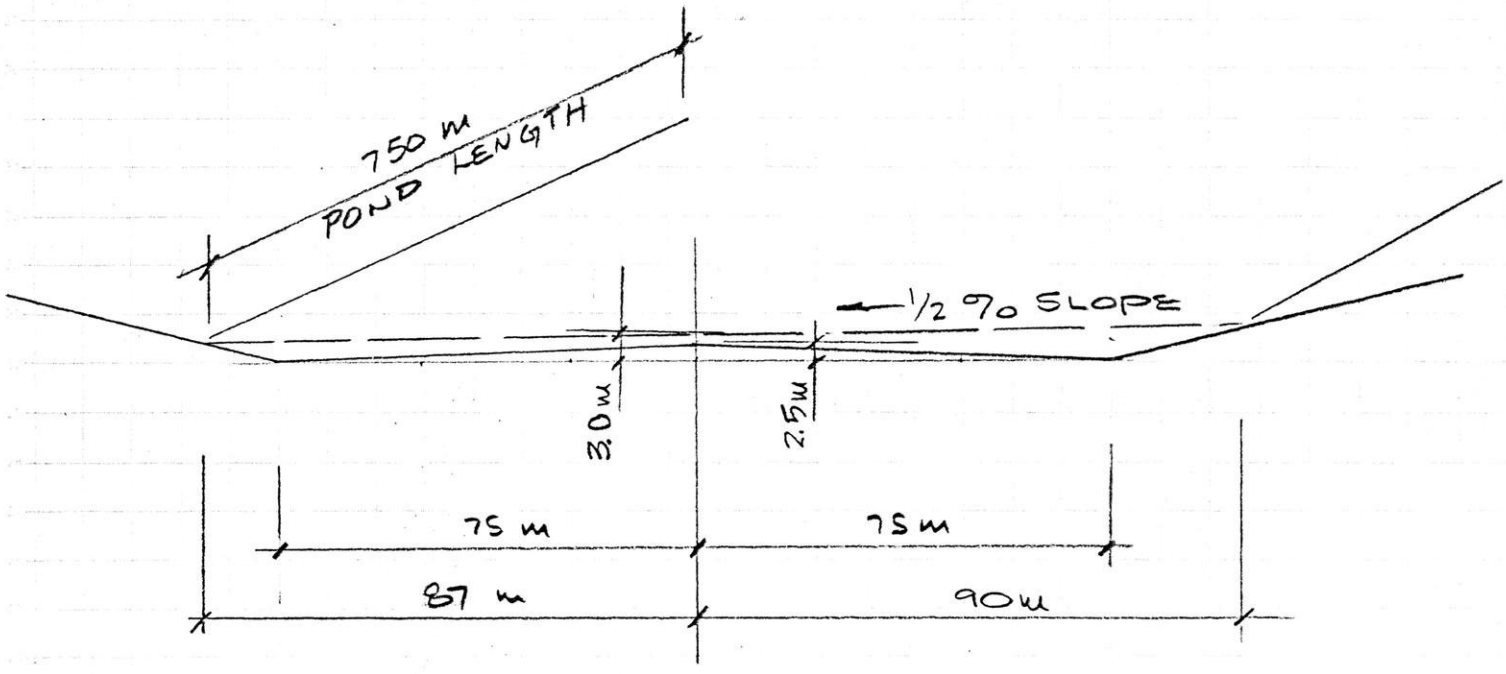
TITLE
TYPICAL POND
POOLED WATER CONDITION

SCALE	STATE	COUNTY
NONE	WISCONSIN	FOREST

DRAWN BY	DATE 3/21/83	CHECKED BY	DATE
APPROVED BY	DATE	APPROVED BY	DATE
APPROVED BY	DATE	EXXON	DATE

DRAWING NO
FIGURE 4
SHEET ___ OF ___
REVISION NO

DETERMINE TIME TO COVER POND BOTTOM WITH TAILS



END AREA = $177 \times 0.5 + 1/2 \times 87 \times 2.5 + 1/2 \times 90 \times 2.5$

END AREA = $310\ m^2$

APPROX. VOL. = $310\ m^2 \times 750m = 232,300\ m^3$

$= 389,530\ t$

at 4300 t/d of tailings ~ 90 days

DETERMINE DEPTH OF POOLED WATER IN TAILINGS PONDS

1) FROM TYPICAL POND WATER BALANCE (SEE ATTACHED FIGURE 1.4-18) DETERMINE POTENTIAL QUANTITY OF WATER AVAILABLE FOR SURFACE POOLING

   a. AVERAGE SURFACE DECANT AND UNDERDRAIN COLLECTION WATER RETURNED TO RECLAIM POND is 0.027 m^3/s (430 gal. per min.)

   b. PORTION SEEPING THROUGH TAILINGS VARIES AS PONDS ARE FILLED. $Q = K i A$, WITH $K = 5 \times 10^{-8}$ m/s (1.6×10^{-7} ft. per sec.), $i = 1$, and A varying from 0 to the inside POND Crest area. At the POINT IN TIME WHEN THE BOTTOM OF THE POND IS JUST COVERED (USING POND T3 AS THE TYPICAL POND) Q THROUGH THE TAILINGS IS:

$$Q = K i A$$

where $A = 12.92$ ha (31.9 ACS.)

$$Q = 5 \times 10^{-8} \times 1 \times 12.92 \times 10000$$

$$Q = 0.006 \ m^3/s \quad (102 \text{ gal. per. min.})$$

WHEN THE POND IS FILLED A has INCREASED TO 40.29 ha (99.5 acs.) and Q is:

$$Q = 5 \times 10^{-8} \times 1 \times 40.29 \times 10000$$

$$Q = 0.020 \ m^3/s \quad (319 \text{ gal. per min.})$$



c. determine TOTAL BOTTOM SEEPAGE ACCORDING to above figure where:

- 90% of bottom area has tailings seeping at unit gradient
- 10% of bottom area has tailings seeping at $i = 6$ $\left(i = \frac{5+1}{1}\right)$

Then Q = bottom Seepage $= KiA$

$= 5 \times 10^{-8} \times 1 \times 12.92 \times 10000 \times 90\%$

$+ 5 \times 10^{-8} \times 6 \times 12.92 \times 10000 \times 10\%$

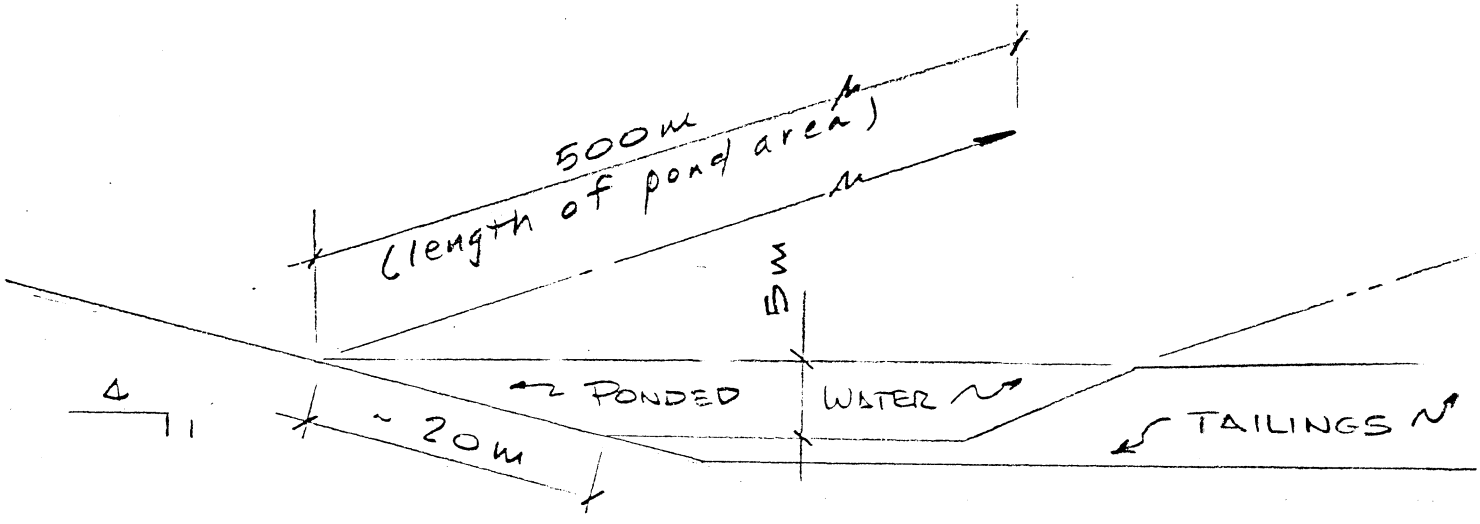
$Q = 0.00969\ m^3/s$ (154 gpm)

Then WATER AVAILABLE FOR POOLING = Q_{POOL}

$Q_{POOL} = 0.027 - 0.00969 = 0.0173\ m^3/s$ (274 gpm)

2. DETERMINE FLOW THROUGH EXPOSED FILTER AREA FOR DIFFERENT PERMEABILITIES, ASSUME THE FOLLOWING :

- CONFIGURATION ACCORDING TO SKETCH BELOW
- LENGTH OF EXPOSED FILTER = 500 m
- 5 m Ponded Water Depth



AREA (EXPOSED FILTER) = 20 m x 500 m
= 10000 m^2

i (EXPOSED FILTER) = (AVG. HEAD + LINER) / LINER
= (5/2 + 1) / 1
= 3.5

a. WITH $K = 1 \times 10^{-6}$

$Q = K \cdot i \cdot A$

$= 1 \times 10^{-6} \times 3.5 \times 10000$

$Q = 0.035\ m^3/s$ (too great - water won't pool to 5 m depth with $K = 1 \times 10^{-6}\ m/s$

determine depth with $K = 1 \times 10^{-6}$

then Exposed AREA $= 4 \times d \times 500$
$= 2000\ d$

Avg. Seepage gradient $= -(d/2 + 1)/1$

then: $Q = kiA$

$$0.0173 = 1 \times 10^{-6} \times [(d/2 + 1)/1] \times 2000d$$

$$8.65 = d^2/2 + d$$

$$d = 3.3$$

b. with $K = 5 \times 10^{-7}$

$$Q = 5 \times 10^{-7} \times 3.5 \times 10000$$

$Q = 0.0175\ m^3/s$ (277 gpm) (just about equal to water available for pooling; nothing to decant)

c. with $K = 1 \times 10^{-7}$

$$Q = 1 \times 10^{-7} \times 3.5 \times 10000$$

$Q = 0.0035\ m^3/s$ (55 gpm)

decant water $= 0.0173 - 0.0035$

$= 0.0138\ m^3/s$ (219 gpm)

d. With $K = 5 \times 10^{-8}$

$Q = 5 \times 10^{-8} \times 3.5 \times 10000$

$Q = 0.00175\ m^3/s$ (28 gpm)

decant Water = $0.173 - 0.00175$

$= 0.0156\ m^3/s$ (247 gpm)

APPENDIX

$$k = \frac{6.214 \times 10^5 (D_{10})^{1.478} (n)^{6.654}}{(P_{200})^{0.597}} \quad \text{(ft./day)}$$

n = Porosity = $(1 - \frac{\gamma_d}{62.4G})$

G = Specific Gravity
(Assumed = 2.70)

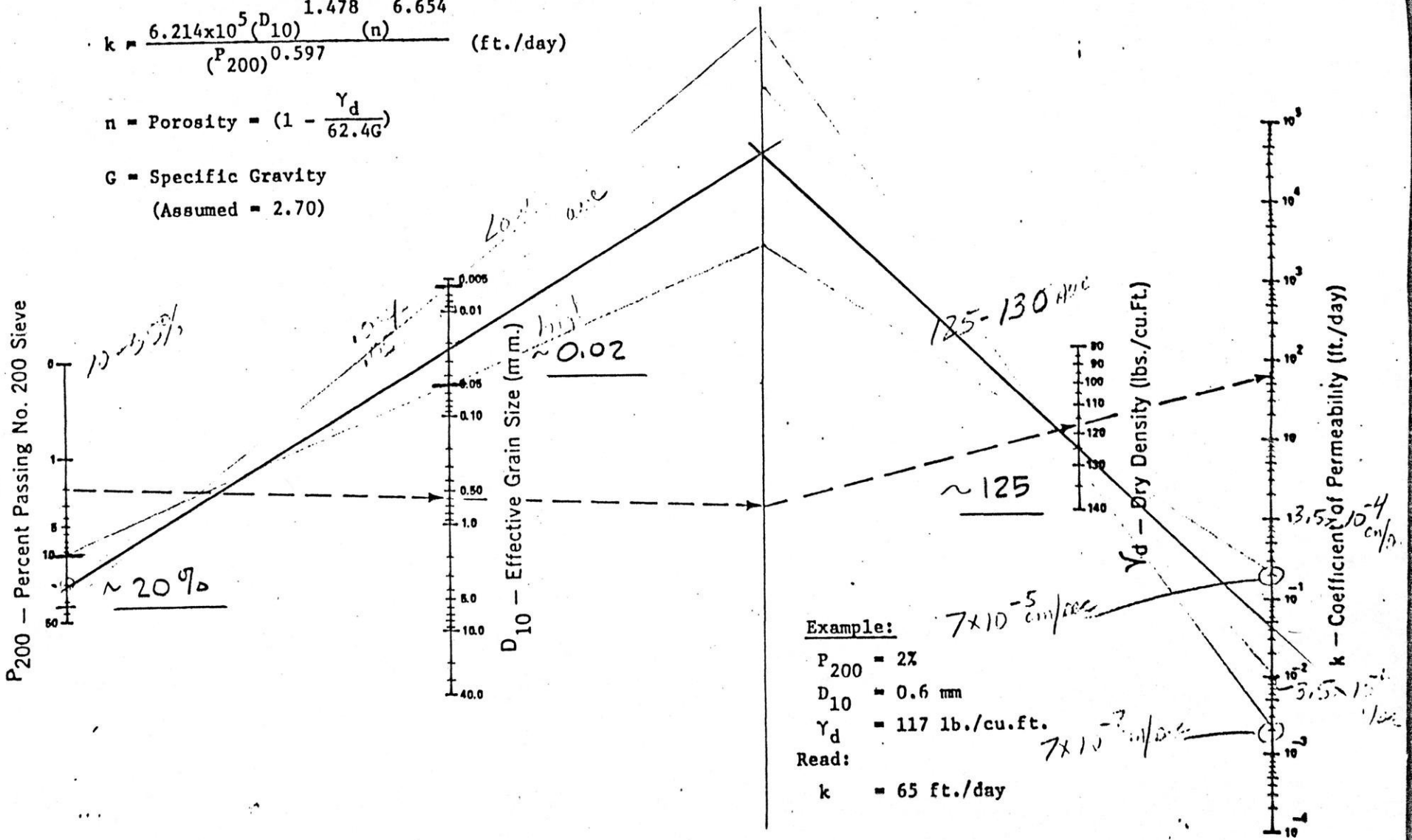


Figure 28. Chart For Estimating Coefficient of Permeability of Granular Drainage and Filter Materials

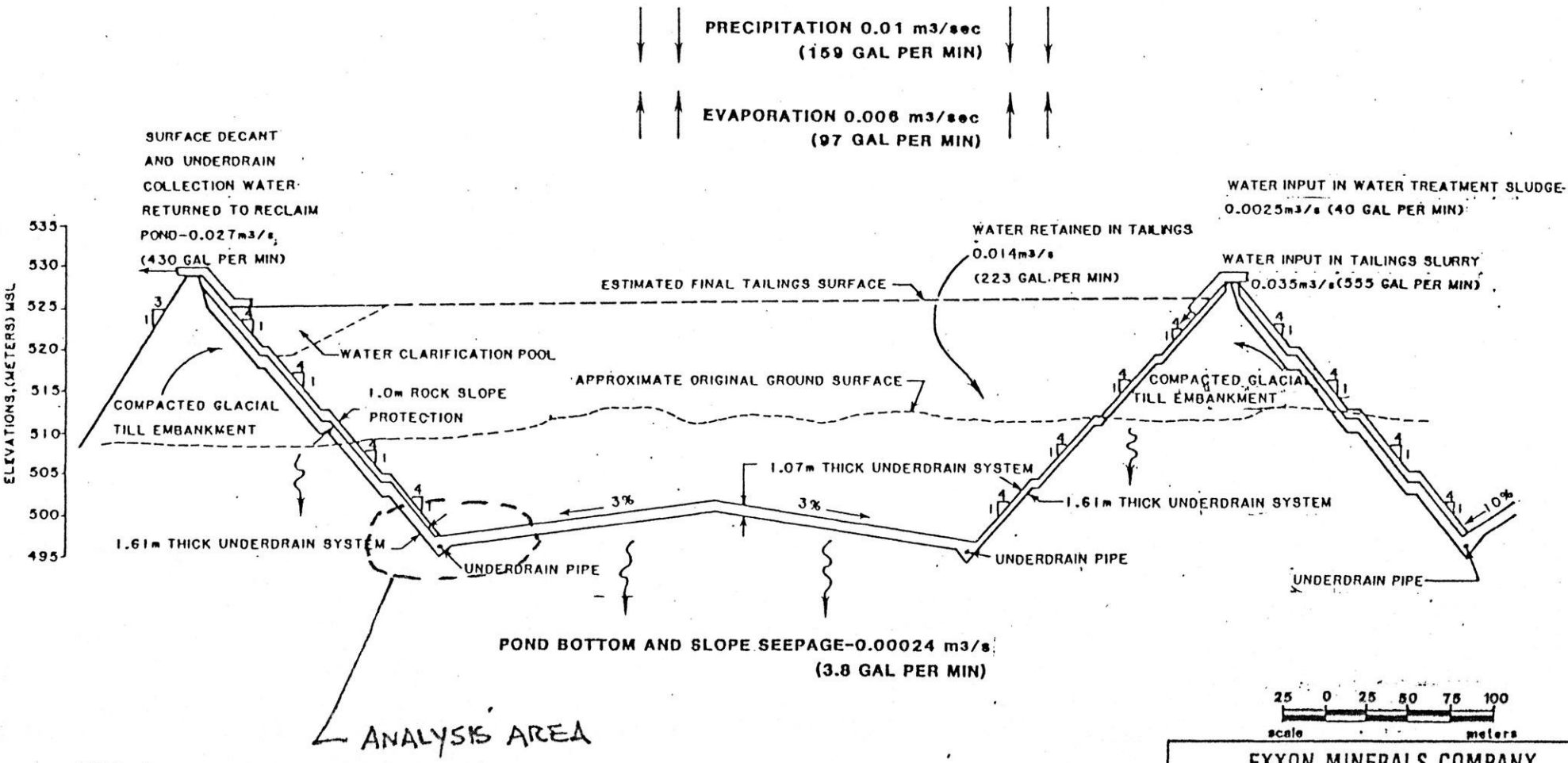
$K = 0.04$ ft/day

$K = 1.4 \times 10^{-7}$ m/s

Ref: Report No. FHWA-TS-80-224
Highway Subdrainage Design, August 1980

TYPICAL POND WATER BALANCE DURING OPERATION

(COMPUTATIONS BASED ON POND T3 AREA WITH AVERAGE PRECIPITATION)



NOTE:

1) VERTICAL SCALE EXAGGERATED

2) PRECIPATATION 0.78m/YEAR (30.77 IN PER YEAR)

3) EVAPORATION 0.59m/YEAR (22.36 IN PER YEAR) OVER 80% OF POND AREA

EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE
WASTE DISPOSAL FACILITY TYPICAL TAILINGS POND WATER BALANCE

SCALE ABOVE	STATE WISCONSIN		COUNTY FOREST		
DRAWN BY J.P. BECKER	DATE 8/82	CHECKED BY C.C. Schroeder	DATE 8-11-82		
APPROVED BY	DATE	APPROVED BY	DATE		
APPROVED BY	DATE	EXXON	DATE		
DRAWING NO FIGURE 1.4-18		SHEET OF	REVISION NO		

NOV 26 1984

14.15.5.23

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

Report on

EXXON CRANDON
PUMP TEST
RESPONSE TO DNR MEMORANDUM

Submitted to:

Exxon Minerals Company
P. O. Box 813
Rhinelander, Wisconsin 54501

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2 copies - Golder Associates

October 1984

834-1389

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

September 12, 1984 834-1389

William C. Walton
Rural Route 5
Box 131
Mahomet, Illinois 61853

Dear Bill:

Enclosed is a draft of Golder's response to the Wisconsin DNR letter concerning the Crandon Pump Test. We have analyzed the test by three different methods and found additional documentation supporting the analysis method used. As you can see from Table 1 of this response, we found no significant variation by using other analysis techniques, including the Neuman type curve method, and have concluded that the results presented in the Pump Test Report are valid.

Please review the analysis and provide us with your comment. If you agree with the analyses and the conclusion, we would appreciate a letter indicating such.

Very truly yours,

GOLDER ASSOCIATES

John F. Clerici, P.E.
Associate

JFC:das

Encl.

cc: Mr. Carlton Schroeder, Exxon
Mr. Charles R. Glore, Exxon

William C. Walton

CONSULTANT IN WATER RESOURCES
RR 5 BOX 131 • MAHOMET, ILLINOIS 61853
TELEPHONE 217-586-4285

RECEIVED
SEP [illegible] 1984
GOLDER ASSOC.

September 17, 1984

Mr. John F. Clerici
Golder Associates
Consulting Geotechnical and Mining Engineers
3772 Pleasantdale Road
Suite 165
Atlanta, Georgia 30341

Dear John:

In response to your letter dated September 12, 1984, I reviewed the report "Exxon Crandon Pump Test Response To DNR Memorandum." I agree with the analyses and I concur with the conclusions. The three methods used to reanalyze the test data are appropriate for existing groundwater conditions. The computed values for parameters seem reasonable in light of my past experiences. The values are in tune with drillers logs.

If I can be of further assistance in this mater please inform me.

Yours truly,

William C. Walton

William C. Walton

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

October 29, 1984 834-1389

Exxon Minerals Company
P.O. Box 813
Rhinelander, Wisconsin 54501

Attn: Mr. Carlton C. Schroeder

RE: SITE 41 PUMP TEST
CRANDON PROJECT

Gentlemen:

Mr. Carlton Schroeder requested in December, 1983, that Golder Associates respond to an internal memorandum from Mr. K. Bradbury to Mr. K. Wade with the State of Wisconsin. This memorandum, dated November 15, 1982, contained specific comments on the pump tests performed at the Exxon Crandon waste disposal Site No. 41 area during June, July and August, 1980. A copy of the State's Memorandum is attached in Appendix A for reference. The test results were presented in a report by Golder Associates dated September 30, 1981 and titled "Pump Test and Analyses, Crandon Project Waste Disposal System, Project Report 4", hereafter referred to as the Report.

The primary purpose of the pump tests was to obtain data for evaluation of hydraulic characteristics of the aquifer system in the Site 41 waste disposal area. This data was considered necessary for future evaluation of the impact of the proposed waste disposal system. Specific objectives of the tests were:

1. To estimate the horizontal hydraulic conductivity of the coarse grained stratified drift, hereafter referred to as drift,

2. To estimate storage coefficients for the aquifer system,

3. To estimate the vertical hydraulic conductivity of the confining till materials, and

4. To demonstrate the extent of groundwater gradient reversals which a pumping well could impose on the groundwater system beneath the Site 41 area.

Response to the specific comments in the letter from the Wisconsin DNR are listed below.

Item 1 VERTICAL HYDRAULIC CONDUCTIVITY OF CONFINING LAYER

Vertical hydraulic conductivities for the confining materials for the Crandon Site 41 pump test were calculated in the Report using a method similar to that developed by M. S. Hantush and C. E. Jacob, Reference 1. For this analysis the leakance factor, B, is given as:

$$B = (Tm'/k_v)^{0.5}$$

where: T = transmissivity of the aquifer (L^2/T)
m' = thickness of the confining layer (L)
k_v = vertical hydraulic conductivity of the confining layer (L/T)

This method was applied to the Boulton analysis to estimate the vertical hydraulic conductivity by rewriting the equation to get:

$$k_v = Tm'/B^2$$

The Boulton analysis, however, is concerned with storage coefficients and not the leakage of confining layers. The "B" parameter in Boultons' analysis is defined differently, References 2 and 3, as follows:

$$B = (T/aS')^{0.5}$$

where: a = reciprocal of the delayed index
S' = total delayed yield volume per unit area per unit drawdown

Detail review of the Boulton work was made by Richard Cooley and Clinton Case, Reference 4, indicating the analysis is the same as that developed by Hantush and that the vertical hydraulic conductivity of the confining layer can be calculated directly as described above. In addition, this work also showed that the problem analyzed by Boulton is for the hydrogeologic setting at Crandon, namely a low permeability material overlying an aquifer with a water table in the upper material.

The similarity of the Boulton and Hantush analyses can indirectly be shown by comparing type curves for the Hantush and Jacob leakage analyses developed by H. H. Cooper, Jr., to type curves developed by Boulton. Both sets of curves are presented in Reference 5 and are used with drawdown data plotted at the same scale. Overlay of the two type curves in Reference 5, Plates 3 and 8 respectively, shows that 2v (Cooper) = R/B (Boulton). Applicable portions of these curves are shown on Figure 1 for reference. Note that in the leaky aquifer range of these curves the drawdowns shown are identical. Cooper also indicates the following equation for estimating the vertical hydraulic conductivity:

$$v = (R/2)(k_v/m'T)^{0.5}$$

where: R is the distance from the pump well.

Rearranging the equation and substituting 2v = R/B we get:

$$k_v = (2v/R)^2(Tm')$$

$$k_v = Tm'/B^2$$

This is exactly the equation used in the Site 41 pump test analysis for estimating the vertical hydraulic conductivity of the confining materials, till.

Additional analyses were performed for different assumed hydrogeologic systems to indicate a reasonable range for the vertical hydraulic conductivity. These analyses were:

1. The Hantush and Jacob/Cooper type curve method: This analysis is for a leaky-confined aquifer system.

2. Neuman and Witherspoon Ratio method: This analysis is for a leaky-confined or leaky-unconfined aquifer system.

3. Neuman delayed yield type curve method: This analysis is for a unconfined anisotropic aquifer with delayed yield response.

These analyses were performed on selected wells to illustrate the range of parameter values that can be expected from each analyses method and for comparison to the results presented in the Report. The results of these analyses are shown in Table 1 and the calculations are attached in Appendix B to this Report. Review of the results in Table 1 indicate that the calculated aquifer

transmissivities and confining layer vertical hydraulic conductivities for different assumed hydrogeologic scenarios generally vary by less than a factor of 3. Therefore, since the analyses used in the Report most closely approximate the hydrogeologic setting at the Crandon Site 41 area, the results presented in the Report are considered to be the most applicable hydraulic parameter estimates.

The leaky aquifer theory is based on large hydraulic conductivity contrasts and vertical flow through the confining layer. With a fully penetrating well some horizontal flow occurs. Close review has been made of the induced gradients to define the direction of flow. Vertical and horizontal gradients for wells in the upper till were estimated after a time period when leakage was affecting the well response, yet drainage of the upper till had not been initiated. This corresponds to the near horizontal, constant drawdown with time, portion of drawdown verses time curves used for the Boulton analyses. Therefore, for times near about 1,000 minutes the gradients were calculated using the following procedure, see Figure 2:

1. Drawdown versus distance was plotted for the upper till and the drift.

2. The horizontal gradient was estimated as the slope of the drawdown versus distance curve at the well location.

3. The saturated thickness of the upper till was estimated as the difference between the water level in the till and top of the drift.

4. The vertical gradient was estimated as the head difference between the upper till and the drift divided by saturated thickness of the upper till (Step 3).

This data indicates that near vertical gradients were occurring in the confining material, since the flow direction was between 2° and 22° off vertical. Neglecting the horizontal component of flow to estimate vertical hydraulic conductivity in the confining material was justified.

The aquifer was described as semi-confined to emphasize the degree of confinement. Yet, during periods of the test the system exhibited leaky behavior with near

vertical flow. The Boulton method for analyzing pump test data is based on a hydrogeologic system similar to that found at the Crandon Site 41 area. The Boulton analysis considers drainage of the material, as observed, and allows estimates of specific yield. In addition, the Boulton analysis provides a method for estimating the vertical hydraulic conductivity of the confining materials.

Item 2 AQUIFER THICKNESS

An average aquifer thickness of 21.3 m (70 ft.) was used in the analysis of the pump test as was indicated in the Management Summary and in the Conclusions to the Report. This value was based on the saturated thickness of drift encountered in the pump test well and other wells in the area of the pump test, as shown on Figures 3.4, 3.5 and 3.6 of the Report. Table 2 (attached) lists each of these wells and the associates aquifer thickness. The calculated average aquifer thickness from Table 2 is 20.8 m (68 ft.) which substantiates the average value of 21.3 m (70 ft.) used in the pump test analysis.

Subsequent data collected at the site, and presented in Reference 7, substantiate the use of 21.3 m (70 ft.) as an average aquifer thickness. Figure 4.10 of the "Geohydrologic Characterization, Crandon Project" report (Reference 7), titled "Saturated Coarse Drift Isopach Contours Project Area" shows saturated drift thicknesses between 15 m (50 ft.) and 40 m (130 ft.) in the area of the pump test. For the area enveloped by the 0.03 m (0.1 ft.) drawdown circle on Figure 6.13 of the Report, the average drift thickness is about 23 m (75 ft.), see Figure 3 (attached). Therefore, the average thickness of 21.3 m (70 ft.) used in the pump test analysis is reasonable.

Item 3 RADIAL DRAWDOWN PATTERN

The circular drawdown pattern shown on Figure 6.13 of the Report was not based on analytical methods. It was developed from the drawdown versus radial distance plot for the drift shown on Figure 6.12 of the Report. The plot of drawdowns on Figure 6.12 shows a well defined curve regardless of the direction from the pumping well; sufficient to define an essentially radial drawdown pattern.

Limited geologic and groundwater data from the area north of well TW-41 was available at the time of the pump test. However, sufficient geologic data was available when the

pump test was designed to reasonably assume that the drift was extensive throughout the Site 41 area. Subsequent data collected at the site supports this initial evaluation. Figure 4.10 of Reference 7 (and Figure 3 attached) shows relatively uniform saturated drift thicknesses within the area of influence of the pump test. Therefore, drawdown north of TW-41 could be expected to be similar to those south of TW-41 and the radial pattern presented on Figure 6.12 of the Report was a reasonable approximation.

Page iv, Para. 1 SCREENED INTERVAL

The test well was screened from about 5 feet above the top of rock to about 10 feet above the coarse grained stratified drift, except for a 43 foot blank section of casing at the location of a fine grained glacial drift. The filter material extended from below the screen to the groundwater table which was about 39 feet above the top of the upper screen section, see Figure 4.1 of the Report. The fine grained glacial till were therefore connected to the well screen intervals via the filter material. The well screen was not extended through these layers in order to reduce the potential for drawing fine grained soil particles directly into the well screen. Therefore, the entire saturated glacial deposit was open to the well with the screen sections only located within the coarse grained portion of the stratified drift. Although those portions of the glacial deposit with high fines content contributed little flow to the well, the filter material was sufficient to achieve pressure reduction in the materials. The entire saturated aquifer (coarse grained stratified drift) thickness was screened.

Page iv, Para. 4 WELL EFFICIENCY

Water levels measured in the well TW-41 were considered representative of the water levels adjacent to the well as shown on Figure 6.1 of the Report. Well efficiency calculations using drawdown data from the flow velocity test and the long term pump test indicate relatively small well losses for the rates used. This is not surprising since extreme care was taken to fully develop well TW-41 prior to testing. The well was surged with air extensively and subsequently jetted at about 350 psi. A

significant amount of fines was removed from the formation during development which would maximize the wells efficiency.

Well efficiency can be calculated as described by W. C. Walton on p. 313 of Reference 8. Using pumping rates from the flow velocity test and the main pump test at times of about two hours the wells loss, C, can be calculated as:

$$C = \frac{\Delta s_i/\Delta Q_i - \Delta s_{i-1}/\Delta Q_{i-1}}{\Delta Q_i + \Delta Q_{i-1}}$$

where: Δs is the increment of drawdown for each corresponding Q rate of pumping.

with: $s_0 = 0$ ft. for $Q_0 = 0$ cfs
$s_1 = 24$ ft. for $Q_1 = 1.2$ cfs
$s_2 = 68$ ft. for $Q_2 = 3.1$ cfs

we get: $C = 1.0$ ft./$(\text{cfs})^2$

At 3.1 cfs (1390 gpm) this corresponds to about 2.9 m (9.6 ft.) of well loss, which is a small well loss (14%) considering that these were not stable water levels. However, the water level shown on Figure 6.1 of the report applies to the well and not the formation.

Page 3, Para. 2 LAKE AND WETLAND INFLUENCE

The effects of the surrounding lakes and wetlands on the pump test was not considered significant, since flow contribution to the well and drawdowns at these sources were small. Duck Lake and the adjacent wetland are perched. As such, flow to the groundwater system is small and would not significantly affect the flow to well TW-41.

Hemlock Slough is considered to be a groundwater discharge area, and could have affected the pump test. Significant drawdowns in the drift, in excess of 0.1 m (0.3 ft.), were not observed beyond about 850 m (2800 ft.) (see Figure 6.12 of the Report) and Hemlock Slough is about 1100 m (3600 ft.) from the well. Therefore, although portions of Hemlock Slough are within the potential area of influence shown on Figure 6.12 of the Report, the very small drawdowns at Hemlock Slough would result in small flow contributions to the well.

Page 30, Para. 2 FLOW VELOCITY TEST

The flow velocity test analysis was based on "the approximate laminar flow equation" below:

$$k = \Delta Q/LH$$

where: k = hydraulic conductivity
ΔQ = flow for a given portion of the well
L = length of the well portion
H = head drop in the well

This procedure is presented on p. 161 of Reference 9, where the specific capacity of the well is used as an estimate of the transmissivity, T. The thickness of the aquifer used was the interval of the well receiving flow. The hydraulic conductivity for each portion was then calculated by dividing the transmissivity by the open interval. So, with T=kL and T=Q/H we get:

$$k = \Delta Q/LH$$

The analysis of the flow velocity test using the above equation indicated hydraulic conductivities at about 2.3×10^{-4} m/s (7×10^{-3} ft./sec), which is in close agreement with the values on Table 7.6 of the Report of 1.3×10^{-4} m/s (4.3×10^{-4} ft./sec). Therefore, for a short duration test such as the flow velocity test, the specific capacity calculation was a good method for estimating the hydraulic conductivity.

Page 41, Para. 4 OPEN WELL INTERVAL

The test well was screened from about 5 feet above the top of rock to about 10 feet above the coarse grained stratified drift, except for a 43 foot blank section of casing at the location of a fine grained glacial drift. The filter material extended from below the screen to the groundwater table which was about 39 feet above the top of the upper screen section, see Figure 4.1 of the Report. The fine grained glacial till were therefore connected to the well screen intervals via the filter material. The well screen was not extended through these layers in order to reduce the potential for drawing fine grained soil particles directly into the well screen. Therefore, the entire saturated glacial deposit was open to the well with the screen sections only located within the coarse grained portion of the stratified drift. Although those portions of the glacial deposit with high fines content contributed little flow to the well, the filter material was suffi-

cient to achieve pressure reduction in the materials. The entire saturated aquifer (coarse grained stratified drift) thickness was screened.

Page 60 and 51 ASSUMPTIONS

The assumptions should be in quotations and the source referenced as noted by the Wisconsin DNR.

Page 60, Para. 2 VERTICAL HYDRAULIC CONDUCTIVITY

The use of the Boulton method for estimating the vertical hydraulic conductivity of the confining layer is discussed in Item 1 above.

Page 74, Para. 3 HYDRAULIC CONDUCTIVITY ANISOTROPIC RATIOS

Anisotropic ratios of hydraulic conductivity for the glacial materials were estimated to be between 3:1 and 10:1, horizontal to vertical. Both the coarse grained stratified drift and the outwash materials were very uniform; all the material was essentially the same size. Depositional sorting of these materials was small and large hydraulic conductivity contrasts could not occur. Large contrasts would have been expected if lenses of silt or clay had been encountered; but, since they were not, the range of 10 horizontal to 1 vertical was a reasonable estimate.

The till materials, on the other hand, were very well graded; the material has a large size range. Generally, till materials do not exhibit any depositional sorting or layering, as was observed in the samples obtained at the site. For most soils this would suggest isotropic soil conditions, 1:1 ratios. The ratio of 3 horizontal to 1 vertical was given to account for consolidation effects.

The reported ratios were given as guides, so some variation in these ratios is justified for modeling. Ratios on the order of 1000:1, however, did not seem reasonable for the coarse grained stratified drift or outwash, nor were ratios on the order of 100:1 considered reasonable for the till materials.

Page 78, Conc. 1 AQUIFER THICKNESS

The 21.3 m (70 ft.) aquifer thickness presented in Conclusion 1 is simplistic since the aquifer thickness varies areally. However, as stated in Items 1 and 3 above, this thickness is a reasonable average that applies north of the pumping well.

Golder Associates hopes that this addresses the concerns of Exxon and the State regarding the pump test and its analyses. Please contact us if you have any questions.

Very truly yours,

GOLDER ASSOCIATES

John F. Clerici, P.E.
Associate

JFC:das

Encl.

REFERENCES

1. Hantush, M.S. and C.E. Jacob, "Non-Steady Radial Flow in an Infinite Leaky Aquifer", Transactions American Geophysical Union, Vol. 36, No. 1, February, 1955.

2. Boulton, N.S., "Unsteady Radial Flow to a Pumped Well Allowing for Delayed Yield from Storage", Publication No. 37 de l'Association Internationale d'Hydrologic, Assemblee generale de Rome, Tome II, 1955.

3. Boulton, N.W., "Analysis of Data from Non-Equilibrium Pumping Tests Allowing for Delayed Yield from Storage", Proceedings of the Institute of Civil Engineers, Vol 26., 1963.

4. Cooley, R.L. and C.M. Case, "Effects of a Water Table Aquitard on Drawdown in an Underlying Pumped Aquifer", Water Resources Research, Vol. 9, No. 2, April 1973.

5. Lohmen, S.W., "Groundwater Hydraulics", Geological Survey Professional Paper 708, U.S. Department of Interior Geologic Survey, 1979.

6. Jacob, C.E., "Radial Flow in a Leaky Artesian Aquifer", Transactions American Geophysical Union, Vol. 27, No. 11, 1946.

7. Golder Associates, "Geohydrologic Characterization Crandon Project", October 6, 1982.

8. Walton, W.C., Groundwater Resource Evaluation, McGraw Hill Book Co., New York, 1970.

9. Groundwater Manual, Water Resources Technical Publication, U.S. Department of Interior Bureau of Reclamation, 1977.

TABLE 1

SUMMARY OF SUPPLEMENTAL PUMP TEST ANALYSIS

Well No.	BOULTON $T^{(1)}$ m^2/s (ft.^{2}/sec.)	BOULTON $K_v^{(2)}$ m/s (ft./sec.)	HANTUSH & JACOB/COOPER $T^{(1)}$ m^2/s (ft.^{2}/sec.)	HANTUSH & JACOB/COOPER $K_v^{(2)}$ m/s (ft./sec.)	RATIO TEST $T^{(3)}$ m^2/s (ft.^{2}/sec.)	RATIO TEST $K_v^{(2)}$ m/s (ft./sec.)	NEUMAN TYPE CURVE $T^{(4)}$ m^2/s (ft.^{2}/sec.)	NEUMAN TYPE CURVE $K_v^{(5)}$ m/s (ft./sec.)
G41-G14A	4.6×10^{-3} (5×10^{-2})	1.2×10^{-6} (4.0×10^{-6})	--	--	--	2.0×10^{-7} (6.5×10^{-7})	--	--
G41-G14B	2.3×10^{-3} (2.5×10^{-2})	1.1×10^{-6} (3.6×10^{-6})	--	--	--	--	--	--
G41-G14D	2.1×10^{-3} (2.3×10^{-2})	2.1×10^{-6} (6.9×10^{-6})	2.1×10^{-3} (2.3×10^{-2})	2.7×10^{-6} (8.8×10^{-6})	--	--	2.9×10^{-3} (3.1×10^{-2})	2.8×10^{-6} (9.3×10^{-6})
G41-G14F	2.4×10^{-3} (2.6×10^{-2})	3.6×10^{-7} (1.1×10^{-6})	--	--	--	--	--	--
G41-G15B	2.6×10^{-3} (2.8×10^{-2})	9.2×10^{-7} (3.0×10^{-6})	2.6×10^{-3} (2.8×10^{-2})	9.0×10^{-7} (3.0×10^{-6})	--	3.6×10^{-7} (1.2×10^{-6})	8.6×10^{-4} (9.3×10^{-3})	8.2×10^{-7} (2.7×10^{-6})
G41-G15	3.5×10^{-3} (3.8×10^{-2})	6.4×10^{-7} (2.1×10^{-6})	--	--	--	--	--	--
G41-E13	6.9×10^{-3} (7.4×10^{-3})	1.9×10^{-6} (6.3×10^{-6})	6.5×10^{-3} (7.0×10^{-2})	2.1×10^{-6} (6.8×10^{-6})	--	--	--	--
G41-K13	3.1×10^{-3} (3.3×10^{-2})	5.8×10^{-7} (1.9×10^{-6})	--	--	--	--	--	--

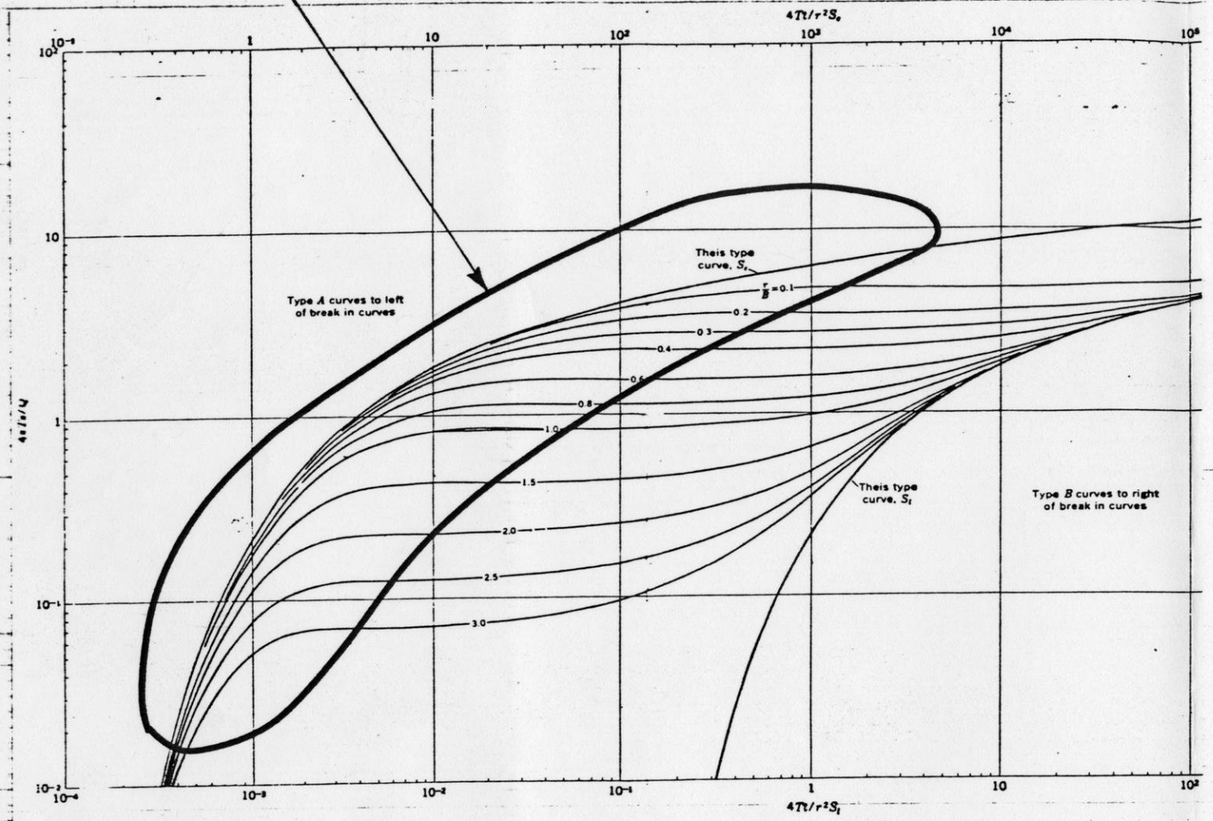
NOTES:
1. T is the transmissivity of the pumped aquifer.
2. K_v is the vertical hydraulic conductivity of confining layer.
3. T is transmissivity of pumped aquifer as calculated by other methods and was, therefore, not included in the table.
4. T is transmissivity of unconfined aquifer (entire saturated zone).
5. K_v is vertical hydraulic conductivity of unconfined aquifer (entire saturated zone).

TABLE 2

ESTIMATED AQUIFER THICKNESS FROM
OBSERVATION WELL DATA

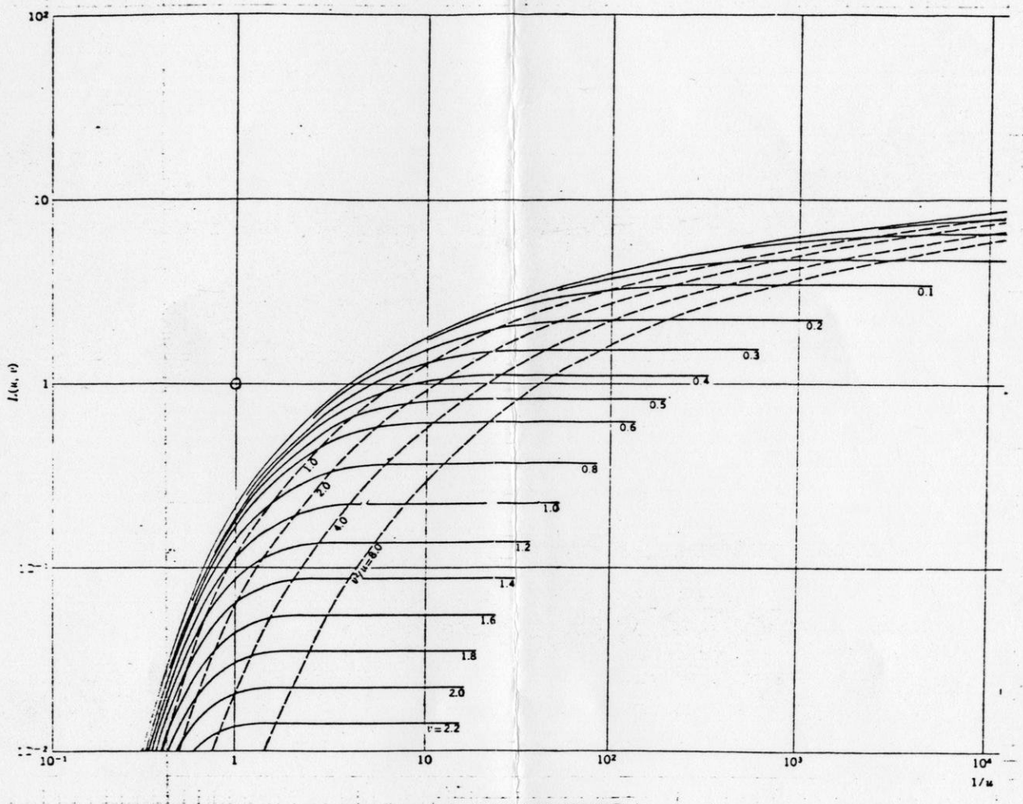
Well Nos.	Thicknesses in m (ft.)
TW-41	22.8 (75)
G41-G14A, B, C	19.5 (64)
G41-G14D, E, F	20.7 (68)
G41-G15, A, B	21.9 (72)
G41-E13 & DMB-1A	35.1 (115)
G41-K13, A	4.6 (15)
AVERAGE	20.8 (68)

PORTION OF THE TYPE CURVES FOR LEAKY AQUIFER BEHAVIOR.



DELAYED-YIELD TYPE CURVES

After Boulton(1963, fig. 1)



A. TYPE CURVES

CURVES FOR NONSTEADY RADIAL FLOW IN AN INFINITE LEAKY ARTESIAN AQUIFER

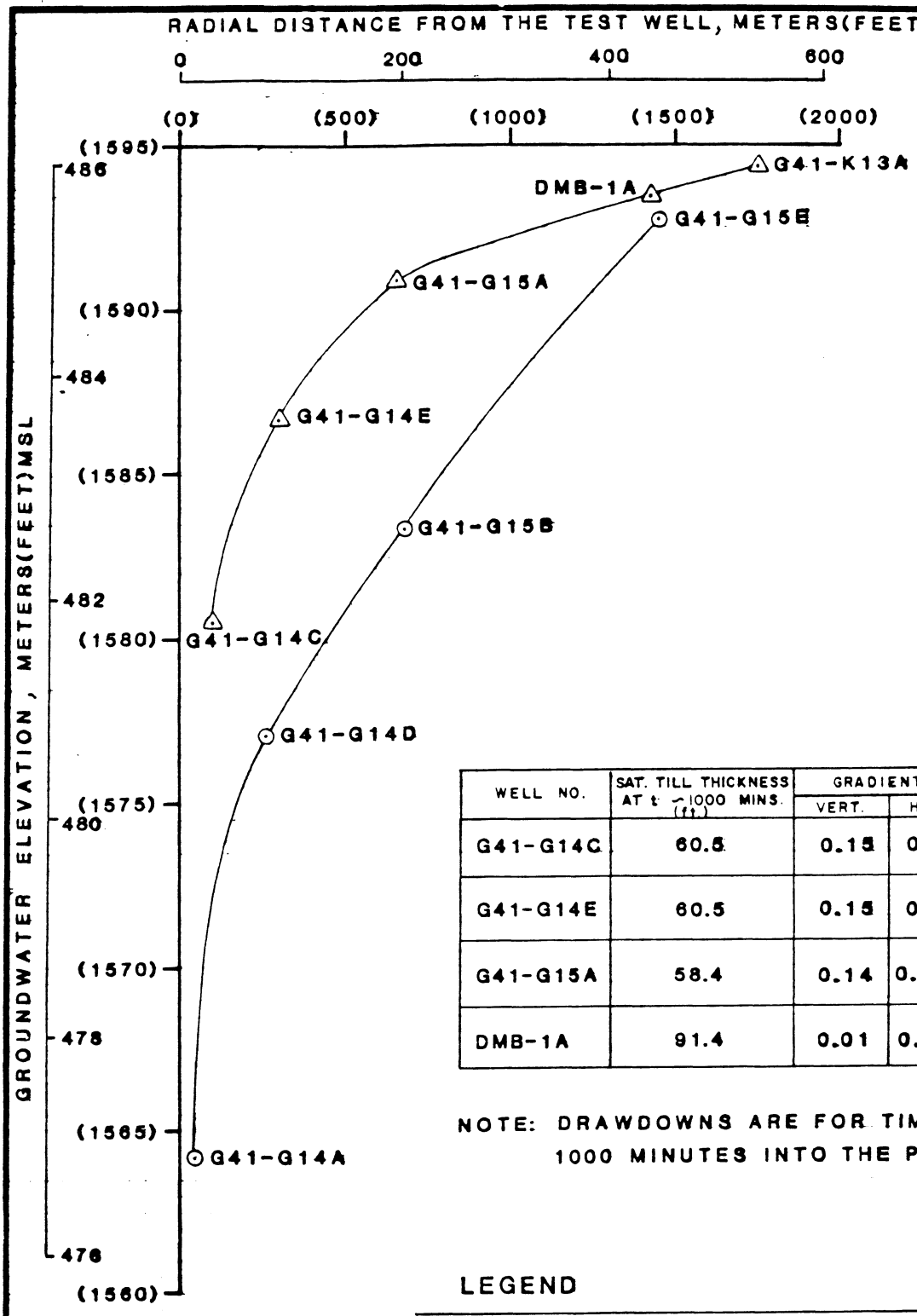
After Cooper,(1963, pl. 4)

NOTES

1. CURVES COPIED FROM PLATES 3 and 8 OF REFERENCE 4.
2. 2V(COOPER) = R/B(BOULTON)

JOB NO. 834-1389	SCALE AS SHOWN	TYPE CURVE COMPARISON	
DRAWN CAB	DATE 1-18-84		
CHECKED JFL	DWG. NO. 050-1-81131		
Golder Associates		EXXON MINERALS COMPANY	FIGURE 1

AF 20440A-B



WELL NO.	SAT. TILL THICKNESS AT t ≈ 1000 MINS. (ft.)	GRADIENTS		DEGREES FM. VERTICAL
		VERT.	HORZ.	
G41-G14C	60.5	0.15	0.06	21.9
G41-G14E	60.5	0.15	0.01	3.8
G41-G15A	58.4	0.14	0.005	2.0
DMB-1A	91.4	0.01	0.001	5.7

NOTE: DRAWDOWNS ARE FOR TIMES APPROXIMATELY 1000 MINUTES INTO THE PUMP TEST.

LEGEND

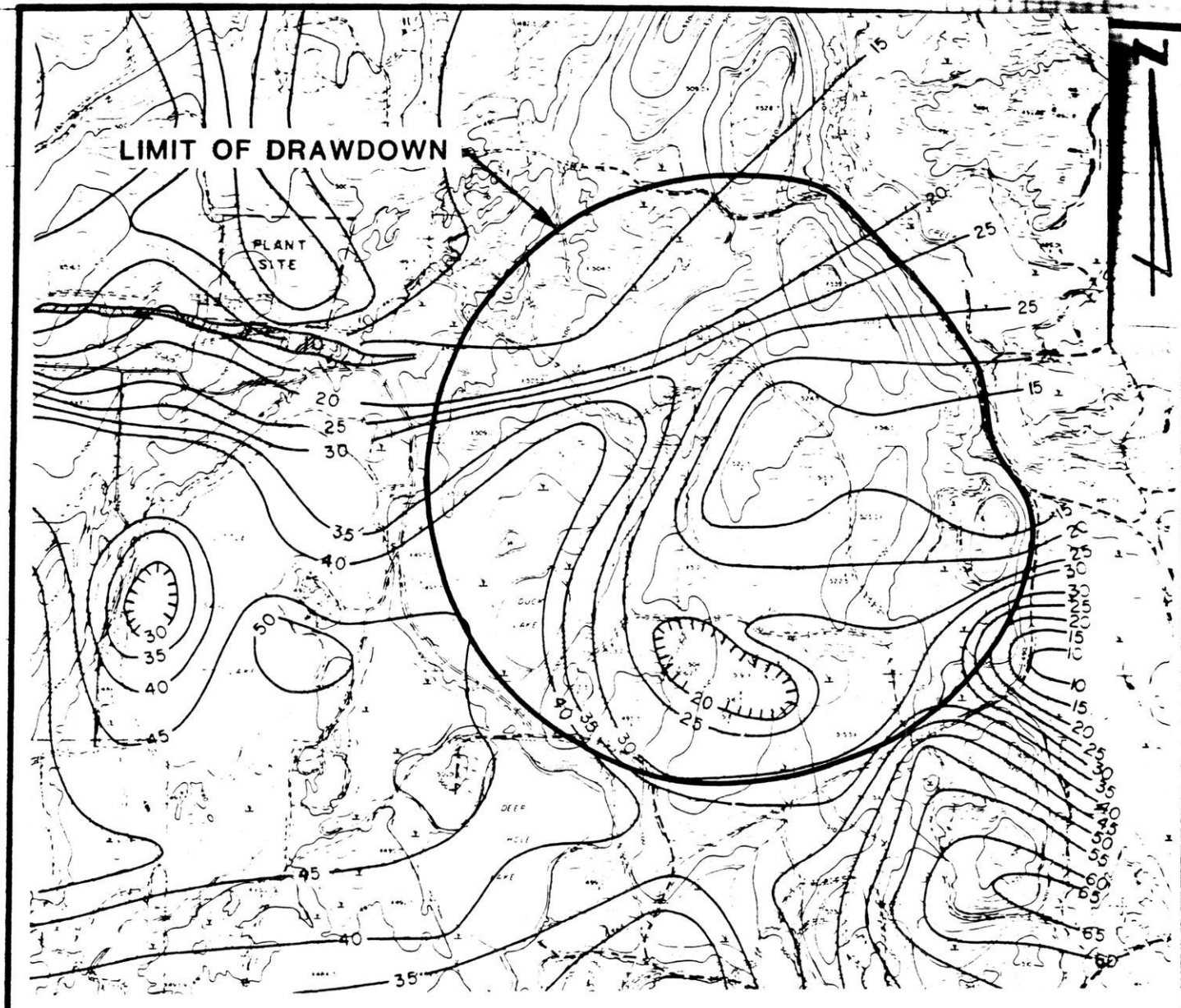
UPPER TILL WATER LEVELS

STRATIFIED DRIFT WATER LEVELS

JOB NO. 834-1388	SCALE AS SHOWN	GROUNDWATER ELEVATION vs. DISTANCE FROM THE TEST WELL AND FLOW DIRECTION IN TILL
DRAWN CAB	DATE 1-12-84	
CHECKED	DWG. NO. 050-1-81130	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 2

DRAFTING MEDIA

AF 20440A-B



250 0 250 500 750 1000

SCALE IN METERS

NOTES

1. ISOPACH THICKNESS REPRESENTS TOTAL THICKNESS OF ALL SATURATED COARSE GRAINED STRAIFIED DRIFT.
2. IN THE CRANDON UNIT AND PLANT SITE VICINITY COARSE TILL MATERIAL IS ADDED TO TOTAL ISOPACH THICKNESS.
3. IN ALL AREAS OTHER THAN THE CRANDON UNIT AND THE PLANT SITE ONLY COARSE GRAINED STRATIFIED DRIFT IS ADDED TO THE ISOPACH.
4. SEE FIGURE 4.10 FOR REFERAL OF ORIGINAL DRIFT ISOPACH MAP.
5. SEE FIGURE 6.13 OF PUMP TEST REPORT FOR LIMIT OF DRAWDOWN BOUNDARY.

JOB NO. 834-1389	SCALE AS SHOWN	SATURATED DRIFT ISOPACH IN AREA OF TW-41 PUMP TEST
DRAWN CAB	DATE 1-12-84	
CHECKED JFC	DWG NO. 050-1-81126	
Golder Associates	EXXON MINERALS COMPANY	FIGURE 8

DRAFTING MEDIA

APPENDIX A

Wisconsin DNR Memorandum

GHC

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WISCONSIN GEOLOGICAL AND NATURAL HISTORY SURVEY
1815 University Avenue, Madison, Wisconsin 53706 608-262-1705

MEMO

to: Ken Wade, Dept. of Natural Resources date: November 15, 1982

from: Ken Bradbury, Hydrogeologist KRB

Re: Review of Crandon Project Pump Test and Analysis, Project Report 4, by Golder Associates

At the request of Ken Wade and Bob Grefe I have reviewed Golder's pump test report on the Crandon site. Although the report is generally complete and well documented, several areas may require additional explanation.

1. The method used to calculate the vertical hydraulic conductivity of the confining layer (K_v) is not clear. Determination of this parameter is not discussed in the Boulton method described by Kruseman and De Ridder(Golder's reference 5). It appears that the K_v calculations are based on the Hantush and Jacob leaky artesian formulae, which assume strictly vertical flow through a leaky confining bed overlying an artesian aquifer. In these equations parameter B is proportional to vertical hydraulic conductivity. On page 59, the Golder report describes the Crandon aquifer as semi-unconfined, which implies that horizontal flow does take place in the semi-confining layer (upper till, in this case). If horizontal flow is occurring, as implied, then the parameter B in the Boulton analysis is a function of both horizontal and vertical flow components, and cannot be used to estimate K_v without additional data. Kruseman and De Ridder make a statement to this effect on page 101 of their text. Thus some additional explanation is needed as to the determination of K_v for the till.

2. The method of determination of effective aquifer thickness as 70 feet (page 78) is not clear, and there is a seeming contradiction between "... a 70 foot thick sheet of coarse grained stratified drift ..." (p. 78) and the three aquifer zones (separated by 35 and 40 feet) described on page 30. At various places in the report the aquifer could be assumed to be 40-110 ft. thick (fig. 3.4-3.6), 122 ft. thick (screened interval on fig. 4.1) or 60 ft. thick (total thickness of zones in table 5.1). How was 70 ft. determined as the best average?

3. There are few data in the report to support the radial drawdown pattern in fig. 6.13 (and elsewhere). The perfectly circular pattern of the contours on this map suggest that they are analytically calculated drawdowns. If so, the analytical

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equation and parameters used should be clearly stated. According to the figure there are absolutely no data north of the pumping site with which to evaluate the shape of the drawdown cone there.

Specific comments follow:

Page iv. Paragraph 1. States that "well is screened over essentially the full depth of saturated material." According to fig. 4.1 the saturated material is 218 ft. thick and the screened interval is 122 ft. long. Thus it appears that only 56% of the saturated material was screened.

Page iv. Paragraph 4. Because drawdown was measured inside the pumped well, the maximum reported drawdown of 73 feet is probably caused by well loss effects, and it is unlikely that as much drawdown occurred in the aquifer adjacant to the well. Thus the final water level as shown outside the well in fig. 6.1 is probably in error.

Page 3. Paragraph 2. States that the test was not influenced by wetlands and lakes. Yet according to Fig. 6-13, the cone of depression did reach several lakes and wetlands. The data appear too sparse to adequately evaluate the influence of wetlands and lakes.

Page 30. Paragraph 2. What is the reference for the "approximate laminar flow formula?" How reliable are hydraulic conductivity determined by this method?

Page 41. Paragraph 4. Explanation assumes "well is fully open to the formation" which, from figure 4.1, is not the case.

Page 60 and 61. Assumptions given here are directly quoted from Kruseman and DeRidder (1979, pages 46 and 97). Such quotations should reference the source.

Page 60. Paragraph 2. The statement that the Boulton method allows evaluation of vertical hydraulic conductivity of the confining layer is contradicted by Kruseman and DeRidder (1979, p. 101).

Page 74. Paragraph 3. The discussion of anisotropy ratios in the stratified drift and outwash is weak. Knowledge of the anisotropy ratio has been shown to be critical in calibration of 3-dimensional numerical models of the type Exxon's consultants are preparing, and other model studies in glacial materials have used Kh:Kv ratios ranging from 2:1 to 1000:1 (Winter, 1978, WRR V14 No 2; Munter and Anderson, 1981, Ground Water V 19 No. 6). Thus a general "common" assumption

of 10:1 and 3:1 ratios for the outwash and till may not be adequate for the modeling study.

Page 78. Conclusion(1) is simplistic and somewhat misleading because it refers to the conceptual model of figure 7.2 rather than to the true field situation. From the cross sections in figures 3.4, 3.5, and 3.6 the stratified drift ranges in thickness from 15 to 110 feet and is somewhat discontinuous. Furthermore according to fig. 3.3 there are no data points north of well TW 41 in order to evaluate the extent of the aquifer.

KB:gl

APPENDIX B

Supplemental Pump Test Analyses

Hydrogeologic data suggest that an applicable hydrogeologic model for the Crandon site would consist of a laterally extensive drift aquifer overlain and underlain by till aquitards. The top of the flow system was defined by the initial phreatic surface which was assumed to represent a constant head boundary. At the bottom of the flow system was an impermeable boundary conforming to the top of the bedrock. If the ratio of aquifer to aquitard hydraulic conductivity is sufficiently high, the model would predict essentially downward vertical flow in the upper aquitard, an upward vertical component of flow in the lower aquitard, and horizontal flow in the aquifer. Analyses presented in the 1981 report were based on modifications to this hydrogeologic model.

Golder Associates has reanalyzed the Site 41 pump test drawdown data using three different methods. This reanalysis is presented in detail in this appendix. The vertical hydraulic conductivity of the confining layer calculated with these new methods is consistent with the estimated values in the Golder 1981 report. Consequently, no revision of the 1981 estimated values of the vertical hydraulic conductivity of the confining layer is recommended.

METHODS OF ANALYSIS

The three methods used to reanalyze the Crandon pump test drawdown data were: the Hantush and Jacob/Cooper type curve method, the ratio test developed by Neuman and Witherspoon and the Neuman delayed yield type curve method. Each of these three methods of analysis are presented below; the results of the analyses are shown in Table 1.

HANTUSH AND JACOB/COOPER TYPE CURVE METHOD

The Hantush and Jacob analysis was developed for estimating the hydraulic characteristics of a leaky-confined aquifer. The particular analysis method used was based on type curve plots subsequently prepared by H. H. Cooper, Reference 1. This method is based on the following assumptions:

1. Flow in the aquifer is horizontal and axially symetric.

2. Flow in the aquitard is vertical.

3. The aquifer and aquitard are seemingly infinite in radial extent.

4. The aquitard has negligible compressibility.

5. Leakage into the aquifer is derived from instantaneous desaturation with lowering of the water table.

6. The aquifer is confined and of uniform thickness and permeability.

7. Well discharge is constant.

According to Cooper, the original leaky aquifer equation by Hantush and Jacob can be rewritten as:

$$s = (Q/4\pi T)L(u,v)$$

with $$u = R^2S/4Tt$$

$$v = R/2\ (k_v/b'T)^{1/2}$$

where:
- s = aquifer drawdown (L)
- Q = pump discharge rate (L^3/T)
- T = transmissivity of pumped aquifer (L^2T)
- L = function
- R = distance from pump well (L)
- S = storage coefficient
- k_v = vertical hydraulic conductivity of the confining layer (L/T)
- b' = thickness of the confining layer (L)

Cooper developed type curves for the function L(u,v). By plotting drawdown verses time on standard 3x5 cycle log paper and superimposing it on the type curves, estimates of T, S and k_v can be made. The results of Golder's analysis of the drawdown data is for selected wells presented in Table 1 and Figures A1 to A3.

RATIO TEST, NEUMAN AND WITHERSPOON

This method is based on essentially the same assumptions as the Cooper type curve method. The only exceptions are that the aquifer can be confined or unconfined and the nature of the upper aquitard boundary is of no consequence. If the aquitard is heterogeneous and isotropic this method will predict the average vertical permeability over the thickness of the confining layer being tested.

The ratio method is valid provided that the following conditions are met:

$$\frac{R}{4}\left(\frac{K_v S_s'}{TS}\right)^{1/2} < 1.0 \qquad \text{Criteria (A)}$$

$$t < \frac{0.1\,(b')^2}{D} \qquad \text{Criteria (B)}$$

where: S_s' = specific storage of the confining layer (L^{-1})
D = diffusivity of the confining layer (L^2/T)
other terms are as previously defined

If criterion (A) and (B) are satisfied, the ratio of aquitard to aquifer drawdown, at a specified radial distance, is given by:

$$s'/s = F\,(t_D, t_D')$$

$$t_D = Tt/SR^2$$

$$t_D' = Dt/z^2$$

where: F = dimensionless time function
s' = drawdown in the confining layer (L)
t_D = aquifer dimensionless time
t_D' = confining layer dimensionless time
z = distance from the top of the aquifer to the bottom of the confining layer piezometer (L)

The function $F(t_D, t_D')$ is tabulated in Reference 2 and is shown graphically in Figure 3. Neuman and Witherspoon describe the following procedure for calculating the vertical hydraulic conductivity:

1. Calculate the ratio (s'/s) from two aquitard/aquifer piezometers of the same radial distance at a specified pumping time. If radial distance to the aquifer piezometer differs from that in the aquitard, the value of (s) can be interpolated using measurements from other aquifer piezometers.

2. Calculate t_D based on known values of T and S. In this case, aquifer properties are determined using the Boulton method.

3. Determine t_D' from Figure A4 using the appropriate t_D curve.

4. Calculate aquifer diffusivity by solving the equation:

$$D = t_D' \, z^2/t$$

5. Based on an assumed value of aquitard specific storage, calculate aquitard vertical hydraulic conductivity:

$$K_v = D \, S_s'$$

6. Verify that calculated and assumed parameters satisfy criterion (A) and (B).

Figures A4 and A5 show aquifer/aquitard responses for piezometer clusters at various radial distances from the pumping well. Responses in the G41-G14A,C and G41-G15A,B clusters (Figures A5 and A6, respectively) have characteristics consistent with those predicted by the ratio method. This is indicated by a distinct time lag in aquitard response. Note that for well group G41-G14A and C that a correction was made for the distance-drawdown as shown on Figure A7. Criteria (B) was slightly violated in the G41-G15A,B calculation. Because this criteria tends to be over-conservative in practical application the results are not expected to have significant errors. Calculated vertical hydraulic conductivities are shown on Figures A5 and A6 and are summarized in Table 1.

The values of vertical hydraulic conductivity given by the ratio method are 2.4 to 6.0 times lower than the K_v values given by the Boulton analyses (see Table 1). Discrepancy between results of the Boulton and ratio methods may be explained as follows:

- Open standpipe piezometers in the aquitard may have exhibited a response lag time (i.e., piezometer water level response slower than actual formation hydraulic response). This would result in an under estimation of K_v when the ratio method is applied.

- In applying the ratio method, it was assumed that aquitard specific storage was identical to that of the aquifer. In most hydrologic situations, aquitard specific storage is greater than aquifer specific storage. If this is the case at Crandon, the assumed value of S_s' was probably artificially low, leading to an underestimation of K_v.

As a result of the above uncertainties, the ratio method is expected to give lower bound values of aquitard vertical hydraulic conductivity.

NEUMAN, DELAYED YIELD TYPE CURVE METHOD

This method is described in Reference 4 and is based on an unconfined homogeneous anisotropic aquifer with delayed gravity response. The following assumptions are also made:

1. Flow in the aquifer is axially symetric.
2. The aquifer is seemingly infinite in radial extent.
3. The aquifer is anisotropic with principal hydraulic conductivities in the horizontal and vertical directions.1
4. The process of delayed water table response is simulated using constant values of aquifer storativity and specific yield. Unsaturated flow above the phreatic surface is neglected.
5. Decline of the water table is small in relation to aquifer thickness.

For the conditions described, Neuman presents the following analytical solution for the general case of partially penetrating production and observation wells:

$$s = \frac{Q}{4\pi T} s_D \left(t_s, t_y, \sigma, \beta, \frac{p}{b}, \frac{d}{b}, \frac{z_1}{b}, \frac{z_2}{b}\right)$$

with:
$t_s = Tt/SR_2$
$t_y = Tt/S_yR^2$
$\sigma = S/S_y$
$\beta = K_z(R^2/Tb)$

where:
s_D = function for dimensionless drawdown
t_s = dimensionless time with respect to aquifer storativity
t_y = dimensionless time with respect to aquifer specific yield
β = dimensionless type-curve value
b = aquifer thickness (L)
b' = aquitard thickness (L)

p = distance from initial position of water table to bottom of perforations in pumping well (L)
d = distance from initial position of water table to top of perforations in pumping well (L)
z_1 = distance from base of aquifer to bottom of perforations in observation well (L)
z_2 = distance from base of aquifer to top of perforations in observation well (L)
S_y = specific yield
K_z = aquifer vertical hydraulic conductivity (L/T)
and all other parameters are as previously defined.

Parameters p and d describe the partial penetration characteristics of the production well, while z_1 and z_2 are related to characteristics of the observation well.

The large number of parameters in the dimensionless function (s_D) makes it practically impossible to construct a sufficient number of type-curves to cover the entire range of values necessary for field application. However, if the value of sigma (σ) is assumed small and geometric parameters (p, d, z_1, and z_2) are specified, it is possible to construct a single set of type-curves having a format similar to the type-curves of Boulton. In this case, each type-curve is associated with a particular value of β. This procedure requires that a unique family of type-curves be developed for each observation well, a task accomplished using a computer program developed by Neuman (Reference 4).

Determination of aquifer parameters follows the same curve-matching procedure as for the Boulton analysis. Matching early time drawdown data to the chosen type-curve (B*) give the following values for any arbitrary (early) match point:

$$s_D^*,\ t_s^*,\ s^*,\ t^*$$

Matching late time data to the same type-curve gives the following (late) match point values:

$$s_D^{**},\ t_y^{**},\ s^{**},\ t^{**}$$

Aquifer transmissivity can be calculated by either of the following two equations:

$$T^* = Q\ S_D^*/4\pi S^*$$

$$T^{**} = Q\ S_D^{**}/4\pi S^{**}$$

Transmissivities given by the above equations should be similar, although estimates based on early time data is generally considered more reliable. Aquifer storativity and specific yield are given by the following equations:

$$S = Tt^*/R^2 t_s^*$$

$$S_y = Tt^{**}/R^2 t_y^{**}$$

and aquifer vertical hydraulic conductivity is calculated as follows:

$$K_z = \beta^* Tb/R^2$$

In applying the Neuman method to Crandon pump test data, it was found that the type-curve program was very costly to execute. Since generation of a complete set of type-curves for an observation well would be exceedingly expensive, individual curves were produced and iteratively compared with field data until a curve having the "best fit" was selected. In this way, computer costs were kept to a minimum. In addition, only two wells were analyzed for comparison to Report values.

Drawdown data and superimposed best-fit curves are presented in Figures A8 to A11 for piezometers G41-G14D and G41-G15B, respectively. Associated calculations of aquifer parameters as shown on Figures A10 and A11 and the results are summarized in Table 1.

CONCLUSIONS REGARDING VERTICAL HYDRAULIC CONDUCTIVITY

The supplemental pump test calculations performed provide three additional independent methods for estimating the aquitard vertical hydraulic conductivity. Two of the methods (Hantush Jacob/Cooper, ratio test) were based on a leaky aquifer model, while the third (Neuman) assumed an anisotropic unconfined aquifer. Vertical hydraulic conductivities given by the above methods generally differed by a factor of less than 3 and were consistent with estimated values quoted in the Pump Test Report of 1981. Lowest values were given by the ratio method. However, due to practical considerations, it can be hypothesized that the ratio test should tend to provide

lower bound values. Based on the work presented in this attachment, it is concluded that there is no justification for substantially revising the best estimate values of vertical hydraulic conductivity of the confining layer as originally presented in the Report.

REFERENCES

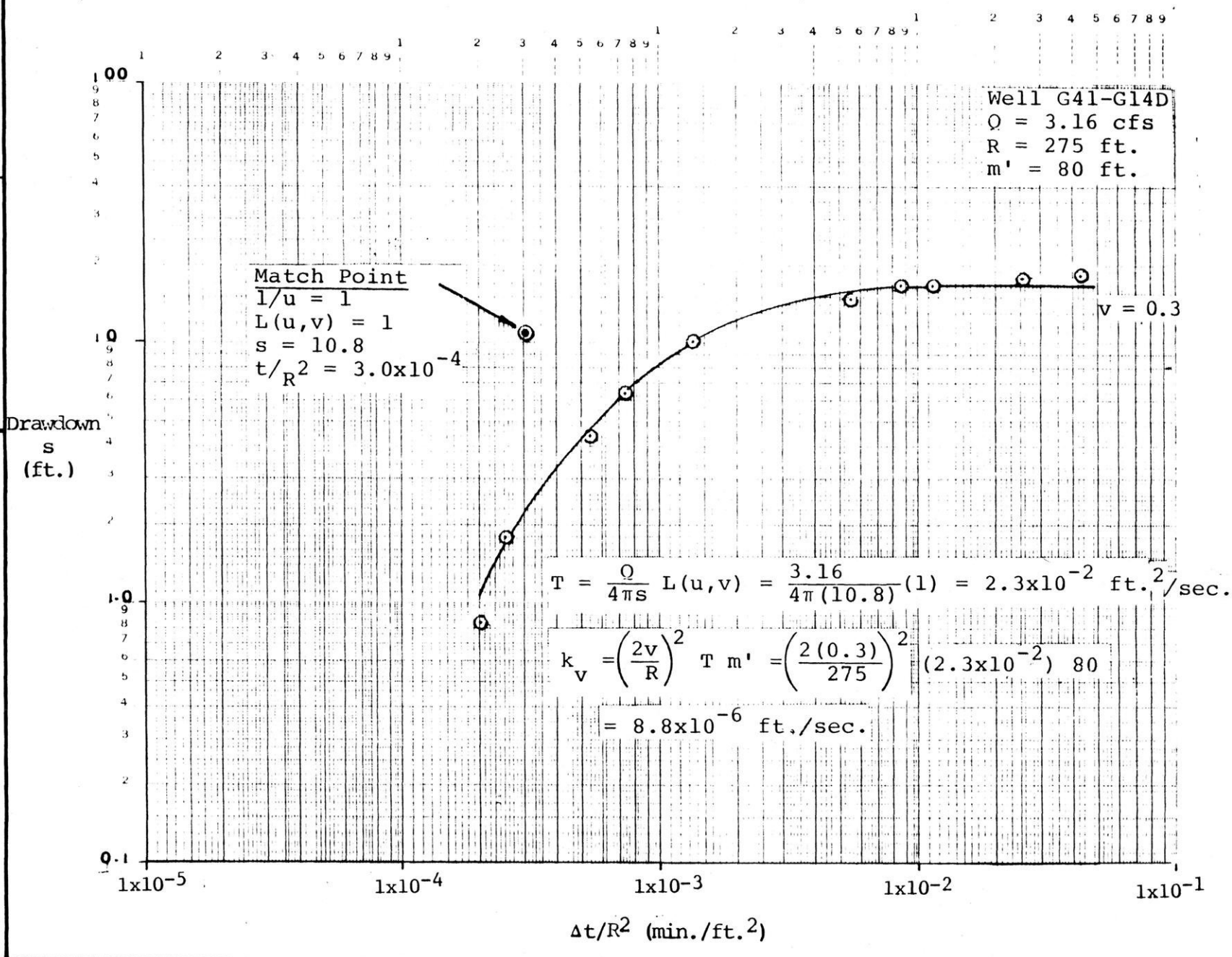
1. Lohmen, S.W., "Groundwater Hydraulics", Geologic Survey Professional Paper 708, U.S. Department of Interior Geologic Survey, 1979.

2. Witherspoon, P.A., I. Javandel, D.P. Neuman, and R.A. Freeze, "Interpretation of Aquifer Gas Storage Conditions from Water Pumping Tests", American Gas Association, Inc., New York, 1967.

3. Neuman, S.P., and P.A. Witherspoon, "Field Determination of the Hydraulic Properties of Leaky Multiple Aquifer System", Water Resources Research, Vol. 8, No. 5, 1972.

4. Neuman, S.P., "Analysis of Pumping test Data from Anisotropic Unconfined Aquifers Considering Delayed Gravity Response", Water Resources Research, Vol. II, No. 2, 1975.

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HANTUSH and JACOB/COOPER ANALYSIS

WELL G41-G14D

FIGURE B1

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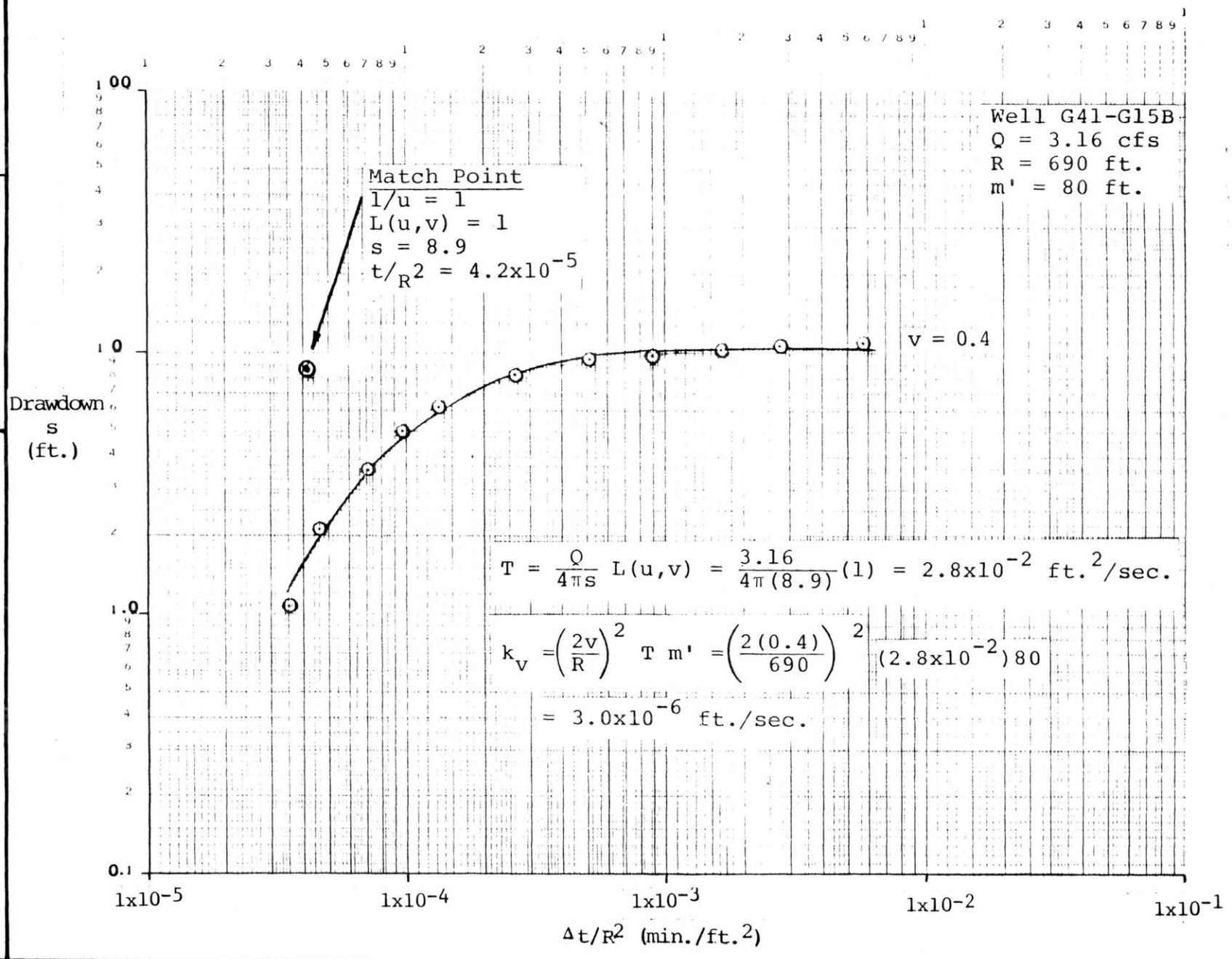
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DATE 1-12-84
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EXXON MINERALS COMPANY

HANTUSH and JACOB/COOPER ANALYSIS
WELL G41-G15B

FIGURE B2

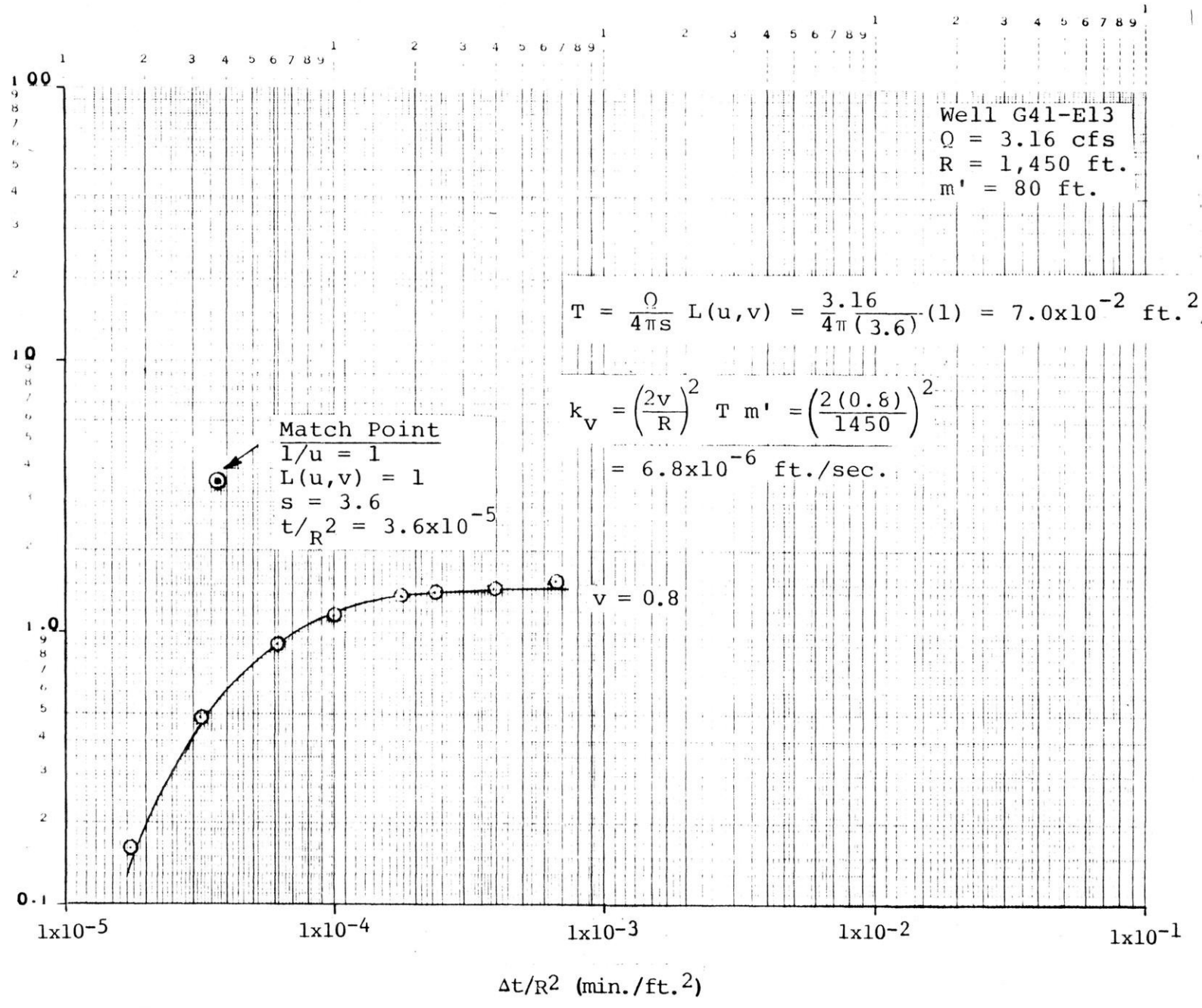


$$T = \frac{Q}{4\pi s} L(u,v) = \frac{3.16}{4\pi(8.9)}(1) = 2.8\times10^{-2} \text{ ft.}^2/\text{sec.}$$

$$k_v = \left(\frac{2v}{R}\right)^2 T\, m' = \left(\frac{2(0.4)}{690}\right)^2 (2.8\times10^{-2})80$$

$$= 3.0\times10^{-6} \text{ ft./sec.}$$

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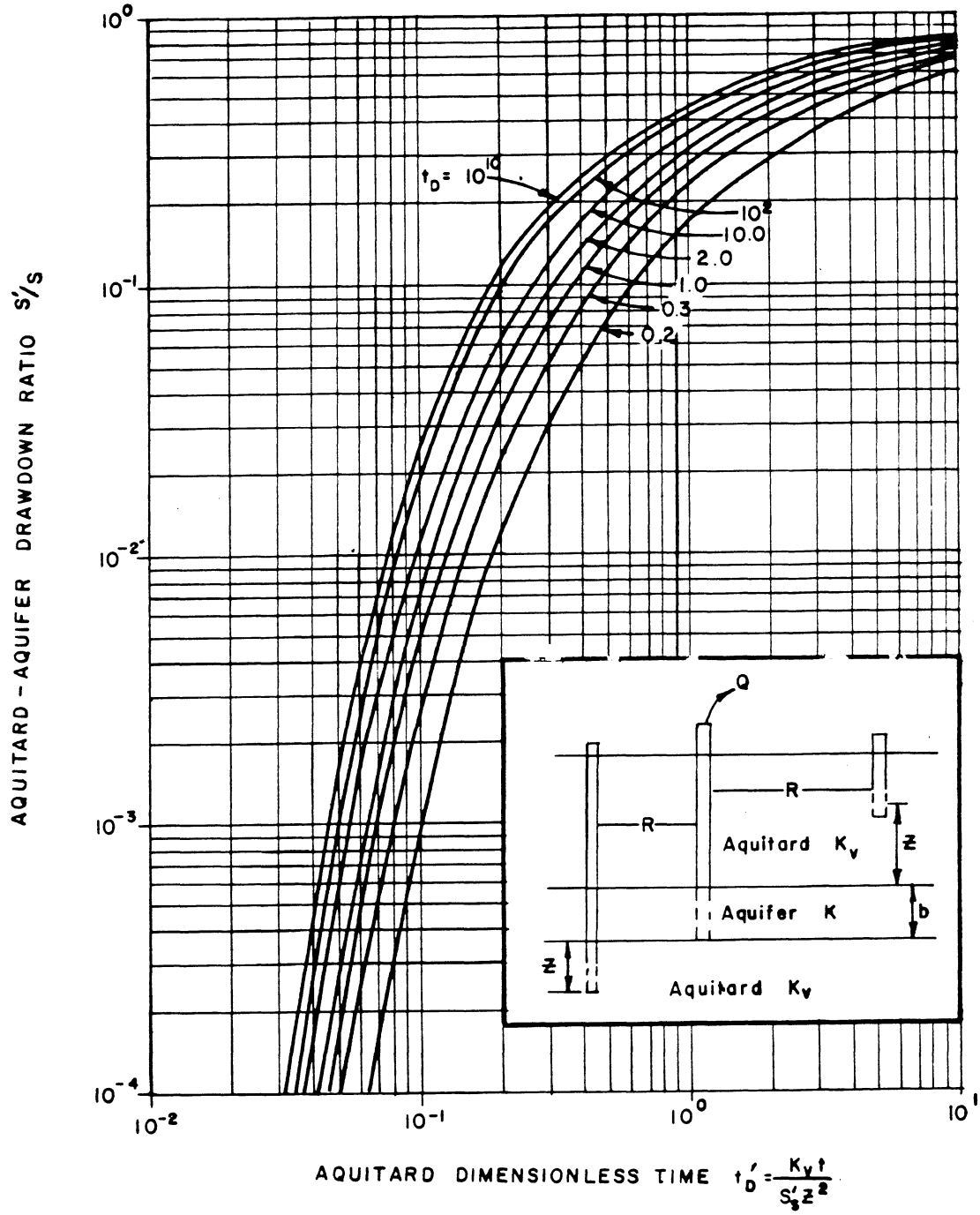
HANTUSH and JACOB/COOPER ANALYSIS

WELL G41-E13

FIGURE B3

AF 20440A-B

DIMENSIONLESS TIME AS A FUNCTION OF AQUITARD - AQUIFER DRAWDOWN RATIO



NOTE: TAKEN FROM REFERENCE 4.

JOB NO. 834-1389	SCALE AS SHOWN	DIMENSIONLESS TIME FUNCTION
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CHECKED	DWG. NO.	
Golder Associates	EXXON MINERALS COMPANY	FIGURE B4

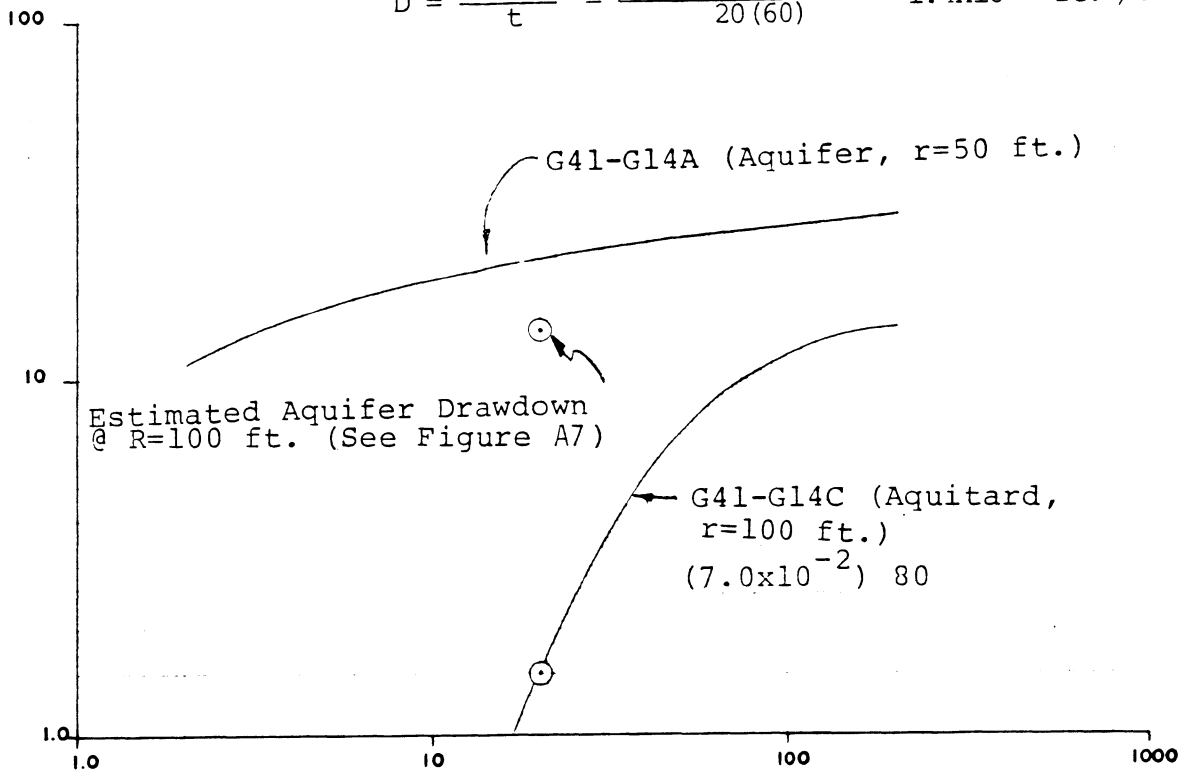
Well G41-G14A & C
s' = 1.5 ft.
s = 14.1 ft.
t = 20 min.
R = 100 ft.
T = 5×10^{-2} ft.^{2}/s
S = 5.8×10^{-4}
Z = 25 ft.
S_s' = 4.6×10^{-6} ft.$^{-1}$

$$\frac{s'}{s} = .11$$

$$t_D = \frac{Tt}{Sr^2} = \frac{(5\times10^{-2})(20)(60)}{(5.8\times10^{-4})(100^2)} = 10.34$$

$$t_D' = 2.7\times10^{-1}$$

$$D = \frac{t_D' Z^2}{t} = \frac{(2.7\times10^{-1})(25^2)}{20(60)} = 1.4\times10^{-1} \text{ ft.}^2/\text{s}$$

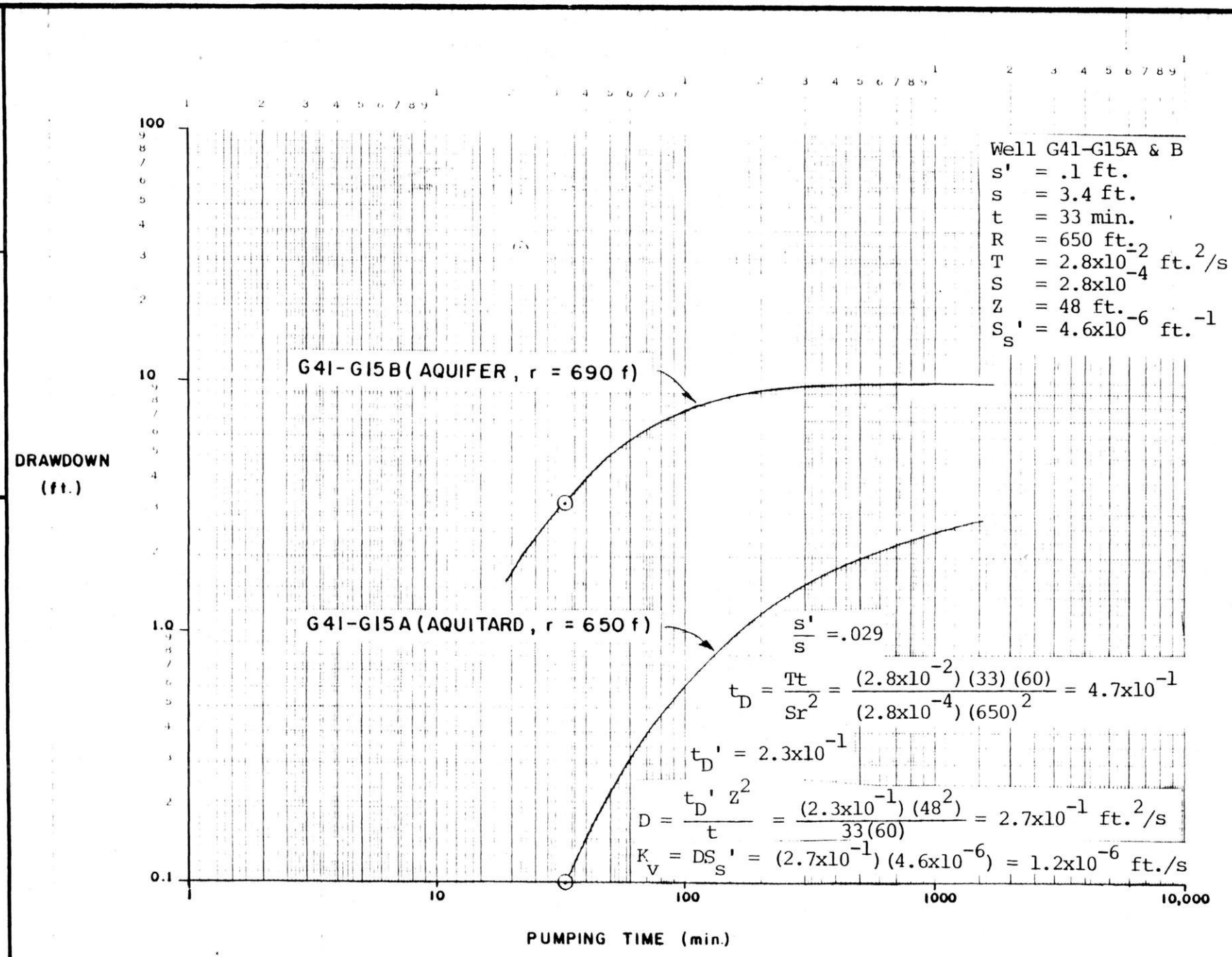


$$K_v = DS_s' = (1.4\times10^{-1})(4.6\times10^{-6}) = 6.5\times10^{-7} \text{ ft./s}$$

JOB NO. 834-1389	SCALE AS SHOWN	RATIO METHOD ANALYSIS G41-G14A,C	
DRAWN SKB	DATE 9-11-84		
CHECKED FC	DWG. NO. 050-1-81131		
Golder Associates		EXXON MINERALS COMPANY	FIGURE B8

OZALID DRAFTING MEDIA

OZALID DRAFTING MEDIA



JOB NO.	834-1389
DRAWN	SKB
CHECKED	
SCALE	AS SHOWN
DATE	9-11-84
DWG. NO.	050-1-81132
FIGURE	86

Golder Associates

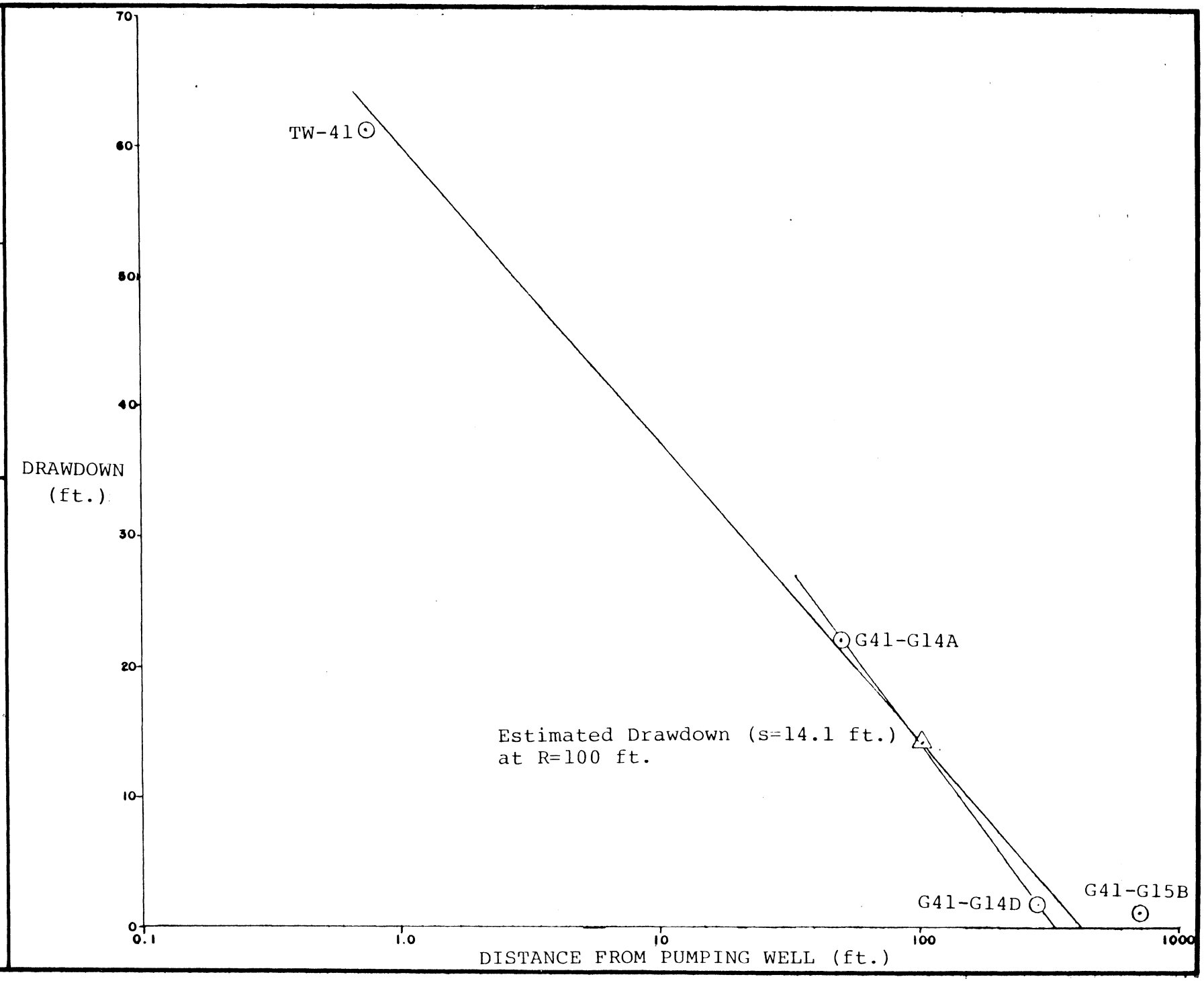
EXXON MINERALS COMPANY

RATIO METHOD ANALYSIS

G41-G15A,B

AF 20440A-B

OZALID DRAFTING MEDIA



Golder Associates

JOB NO.	834-1389
DRAWN	SKB
CHECKED	
SCALE	AS SHOWN
DATE	9-11-84
DWG NO.	050-1-81133

EXXON MINERALS COMPANY

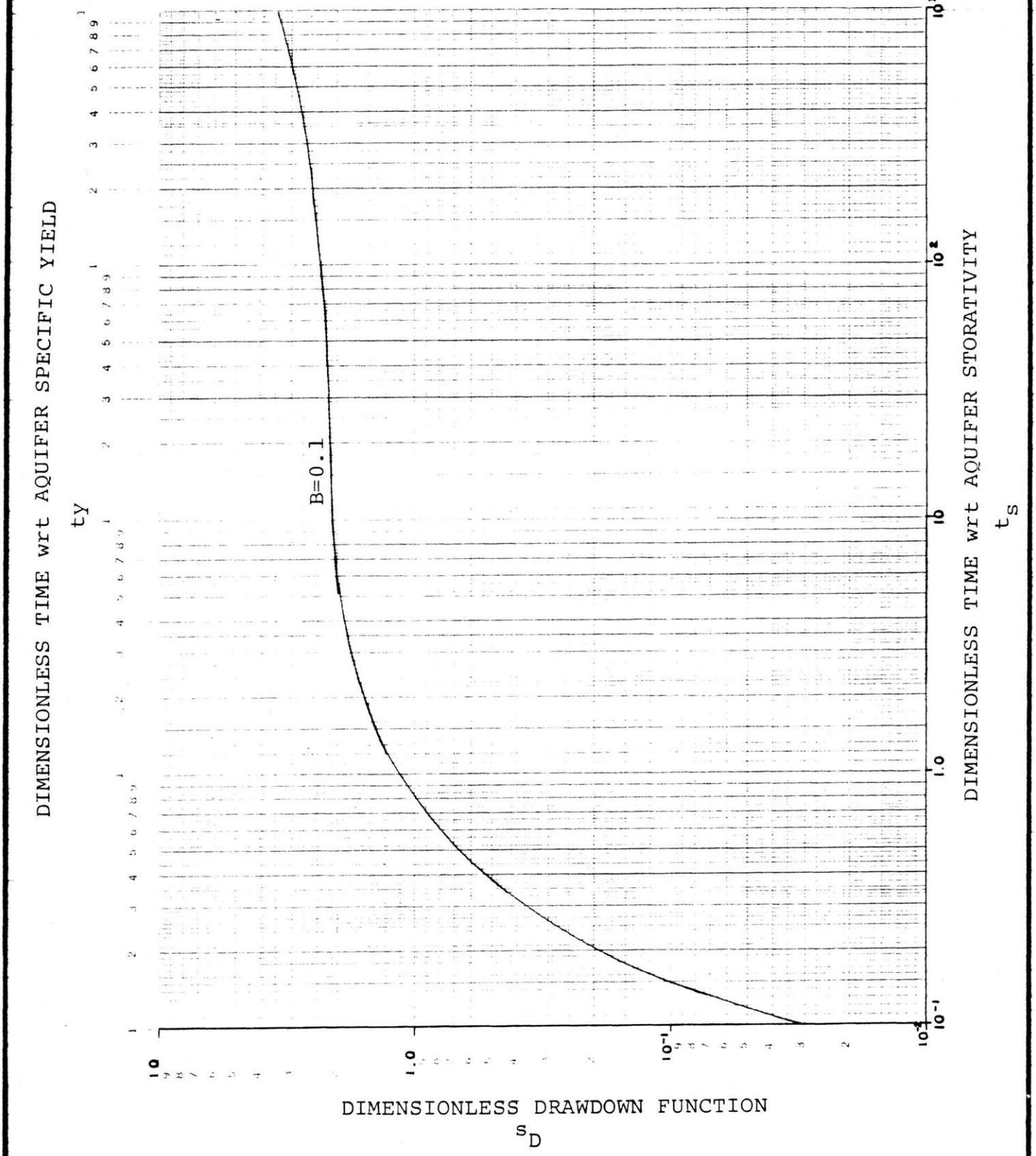
DISTANCE-DRAWDOWN PLOT

AT TIME-20 MINUTES

FIGURE B7

AF 2044OA-B

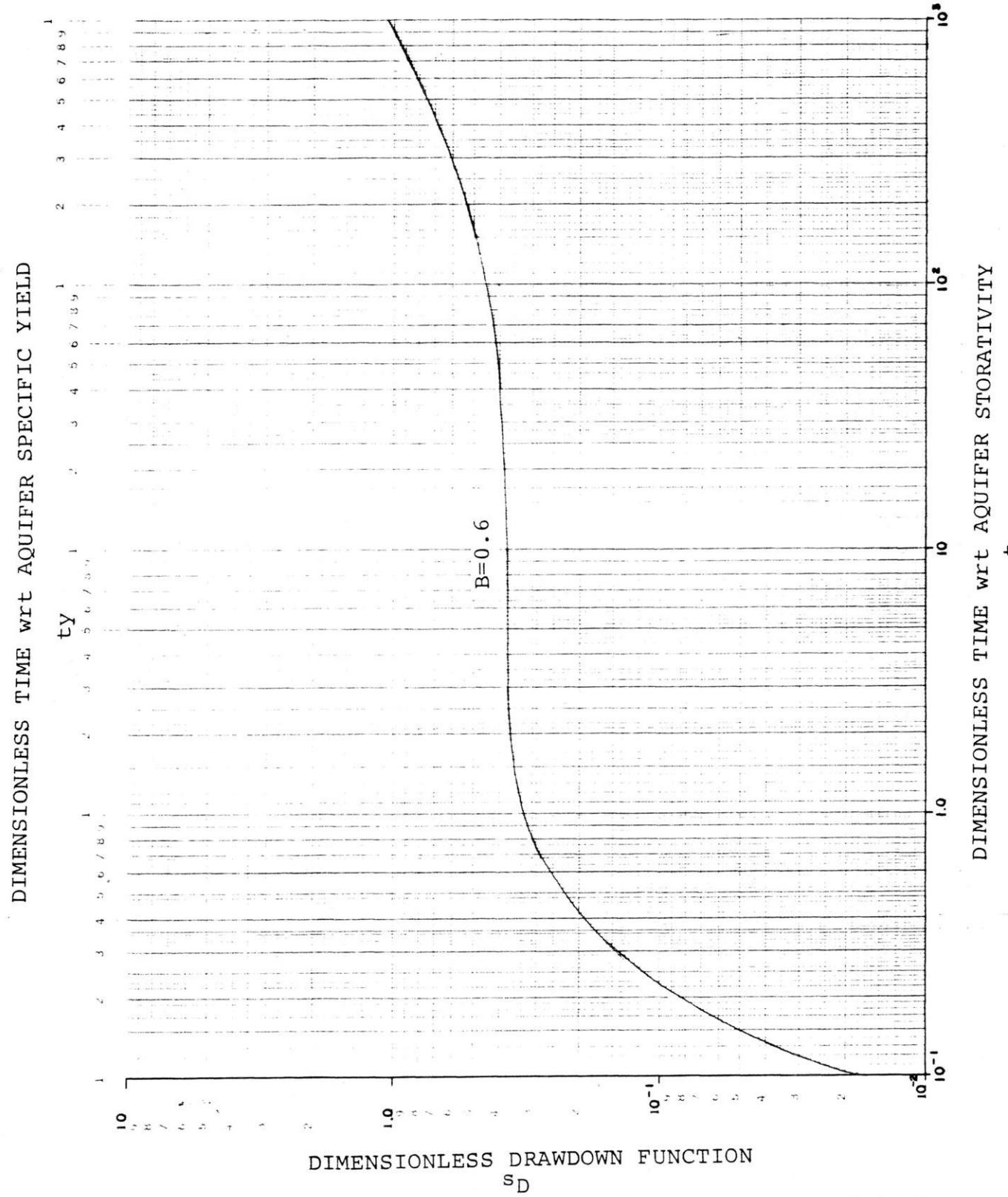
AF 20440A-B



JOB NO. 834-1388	SCALE AS SHOWN	DELAYED YIELD TYPE CURVE G41-G14D	
DRAWN SKB	DATE 9-11-84		
CHECKED JFC	DWG. NO. 050-1-81134		
Golder Associates		EXXON MINERALS COMPANY	FIGURE B8

ozalid DRAFTING MEDIA

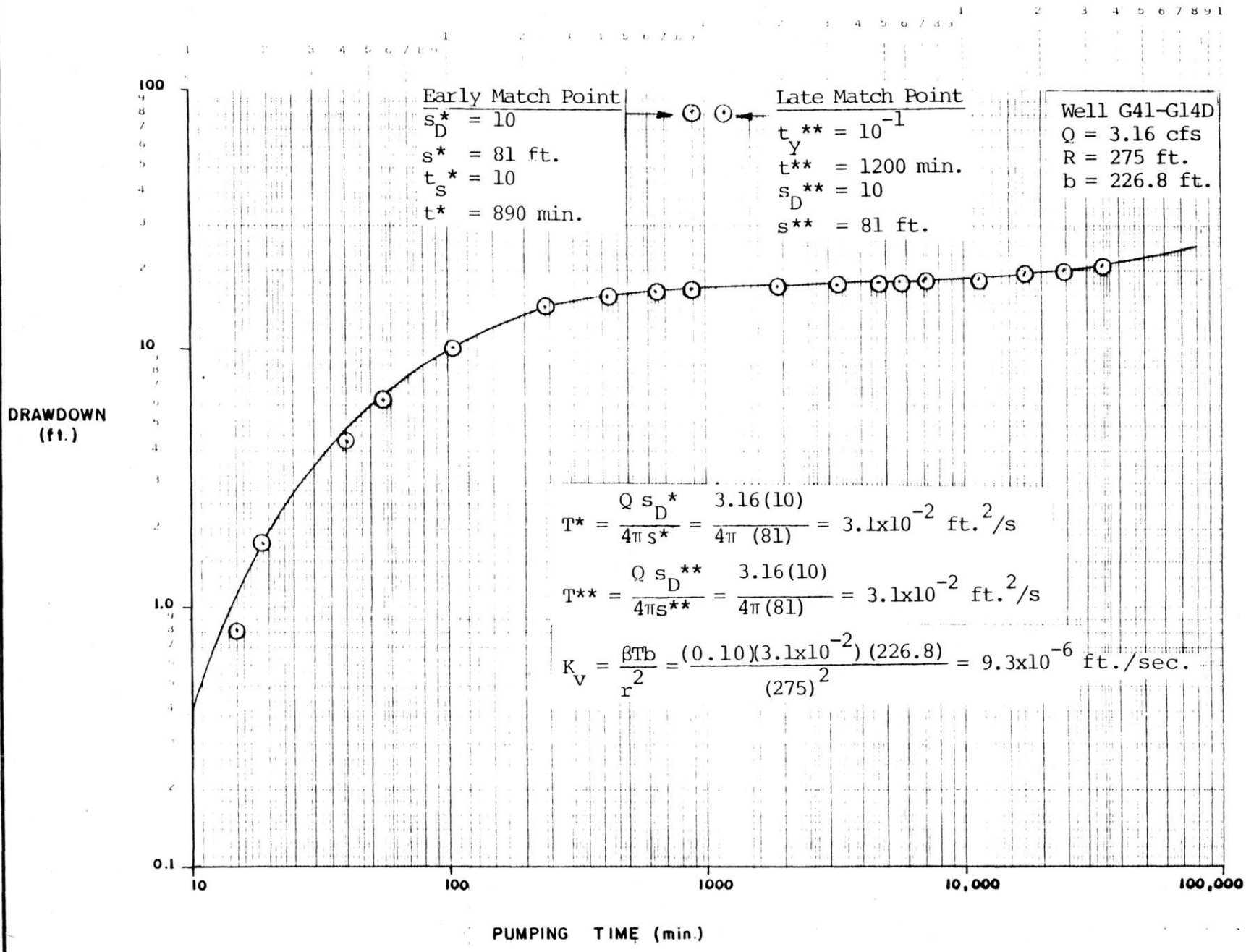
A-# 20440A-B



JOB NO. 834-1389	SCALE AS SHOWN	DELAYED YIELD TYPE CURVE G41-G15B	
DRAWN SKB	DATE 9-11-84		
CHECKED JFC	DWG. NO. 050-1-81135		
Golder Associates		EXXON MINERALS COMPANY	FIGURE B9

GOLDER DRAFTING MEDIA

DRAFTING MEDIA



Golder Associates

JOB NO.	834-1389
DRAWN	SKB
CHECKED	
SCALE	AS SHOWN
DATE	9-11-84
DWG. NO.	050-1-81139

EXXON MINERALS COMPANY

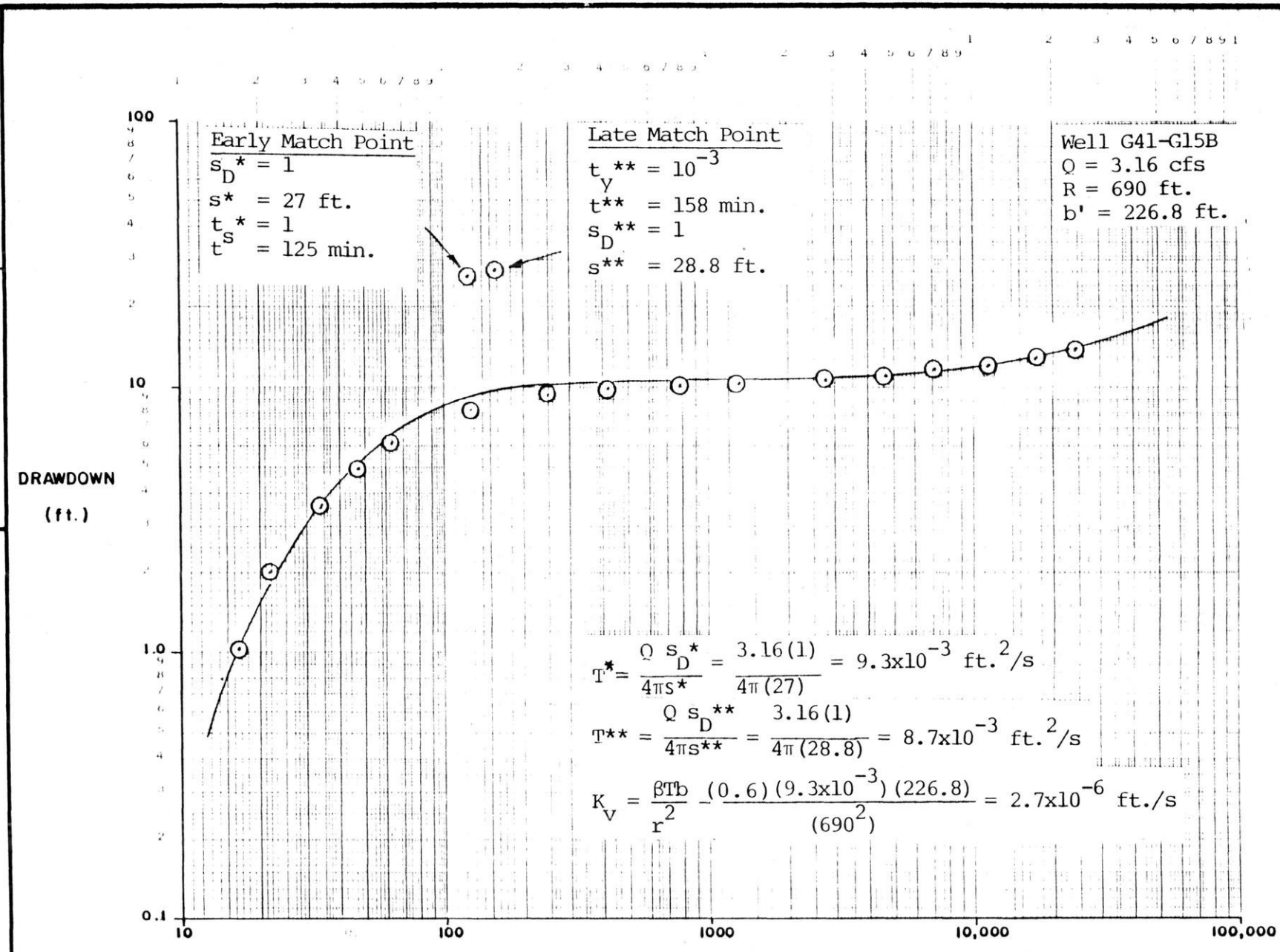
DELAYED YIELD TYPE CURVE ANALYSIS

G41-G14D

FIGURE B10

AF 20440A-B

KEUFFEL & ESSER CO. DRAFTING MEDIA



Golder Associates

JOB NO.	834-1388
DRAWN	SKB
CHECKED	
DWG. NO.	050-1-81137
DATE	9-11-84
SCALE	AS SHOWN

EXXON MINERALS COMPANY

DELAYED TYPE CURVE ANALYSIS

G41-G15B

FIGURE B11

A-F 20440A-B

HYDRAULIC RELATIONSHIP BETWEEN SITE AREA LAKES AND THE MAIN GROUND WATER AQUIFER CRANDON PROJECT FOREST COUNTY, WISCONSIN

FOR

EXXON MINERALS COMPANY
655 Washington Street
Rhinelander, Wisconsin 54501

D&M Job No. 08837-189-07
September 20, 1984

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1.0 INTRODUCTION

This report examines the hydraulic relationship between Oak, Duck, Deep Hole, Little Sand and Skunk lakes and the main ground water aquifer in the Crandon Project site area. The potential effect of ground water drawdown upon these five lakes is examined. The results indicate that movement of water between the lakes and the aquifer is limited. Different potentiometric levels for the lakes and aquifer, as well as the presence of lacustrine deposits of low permeability, indicate a very restricted hydraulic connection.

The degree of hydraulic connection between the five lakes and the ground water was defined using existing ground and lake water data, and the effect of the lacustrine deposits in the lake bottoms was quantified for each lake. The principal sources of data were the Geology and Ground Water sections of the Crandon Project Environmental Impact Report (Exxon Minerals Company, 1982) and reports by STS Consultants, Ltd. (1982, 1984) for Exxon Minerals Company.

2.0 METHODOLOGY

The interaction of the main ground water aquifer with the lake water level in Oak, Duck, Deep Hole, Little Sand and Skunk lakes was examined using three different approaches, with each approach concentrating on different hydraulic and geologic aspects of the problem. For each lake, the following were examined:

- Differences in the elevation of the potentiometric surface of the underlying aquifer and the lake water surface elevation;
- Results of the boring and soils tests of the lake bottom sediments; and
- Estimates of lake bottom seepage for current and "worst-case" conditions were prepared.

The study relied extensively on the large data base collected since 1977 for the Crandon Project. Figure 1 presents many of the important features of the drilling program. Seepage of lake water through the lake sediments was calculated using Darcy's Law for water movement:

$$Q = k \times \Delta h/L \times A$$

where: Q = discharge (L^3/T)

k = coefficient of permeability (L/T)

Δh = difference in head between the lake surface and ground water (L)

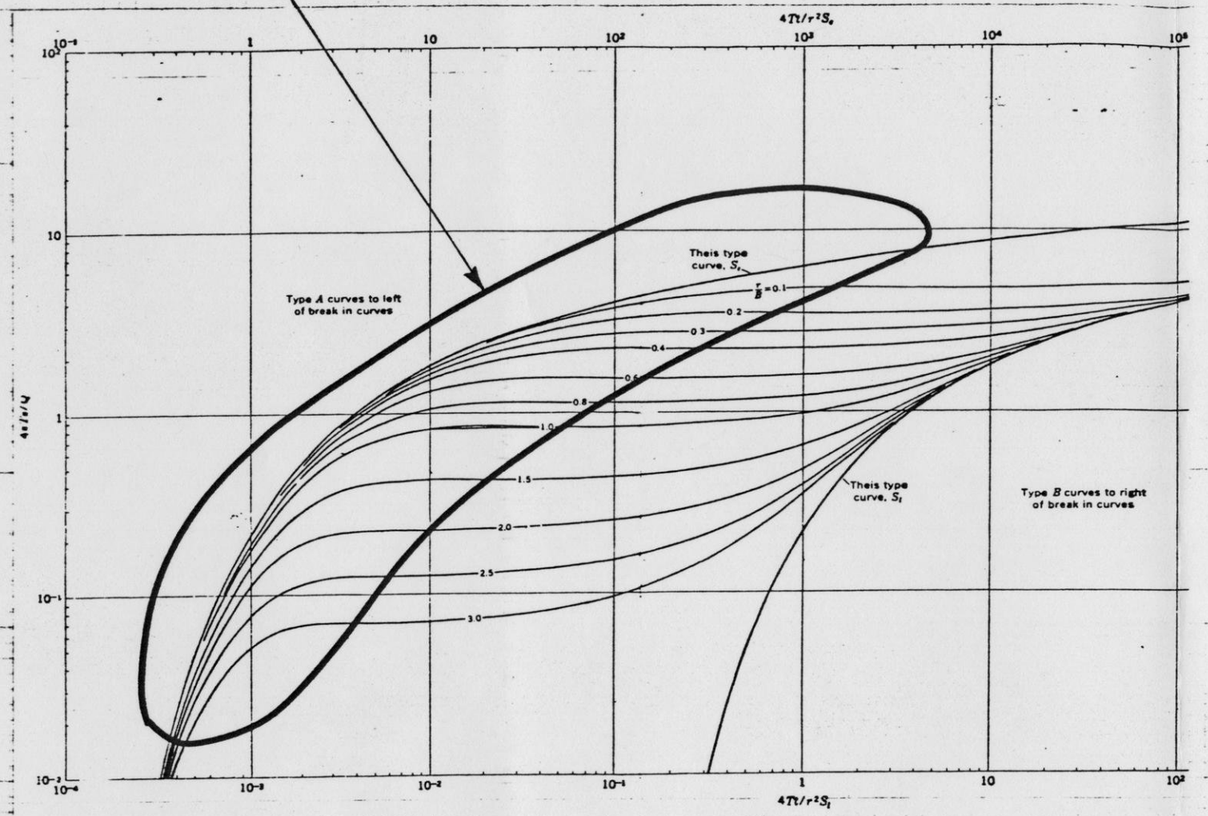
L = sediment thickness (L)

A = area of flow ($L \times L$)

The time of travel of lake water through the sediments was calculated by rearranging the preceding equation in the following manner:

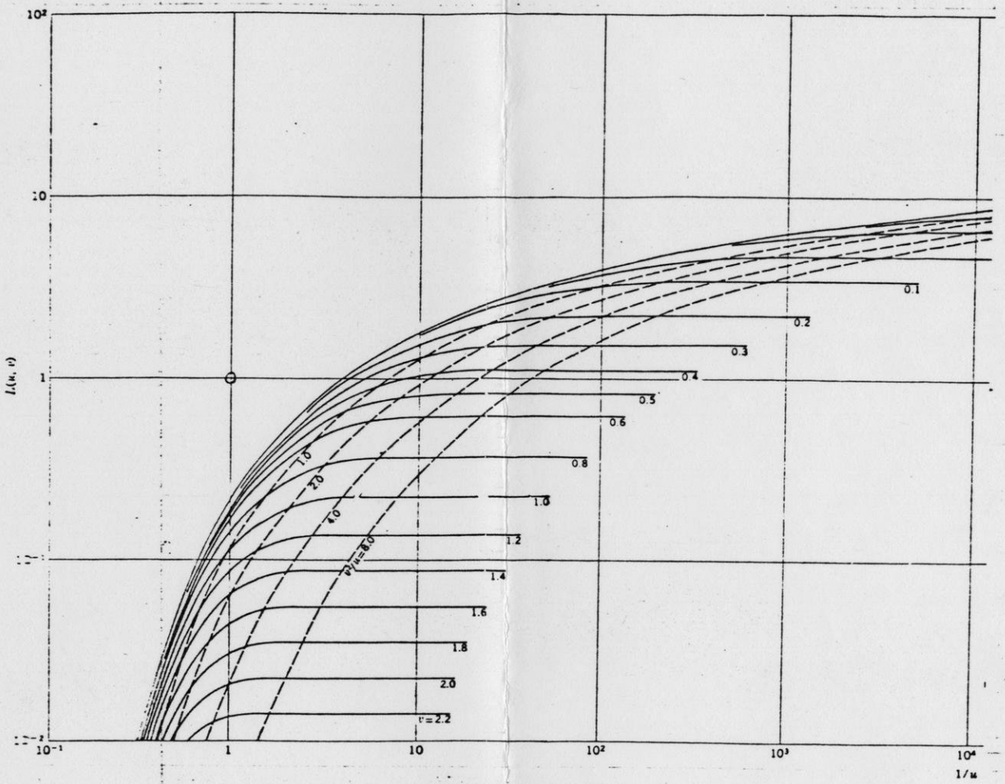
$$\text{time of travel} = L^2/(k \times \Delta h)$$

PORTION OF THE TYPE CURVES FOR LEAKY AQUIFER BEHAVIOR.



DELAYED-YIELD TYPE CURVES

After Boulton(1963, fig. 1)



A. TYPE CURVES

CURVES FOR NONSTEADY RADIAL FLOW IN AN INFINITE LEAKY ARTESIAN AQUIFER

After Cooper,(1963, pl. 4)

NOTES

1. CURVES COPIED FROM PLATES 3 and 8 OF REFERENCE 4.
2. 2V(COOPER) = R/B(BOULTON)

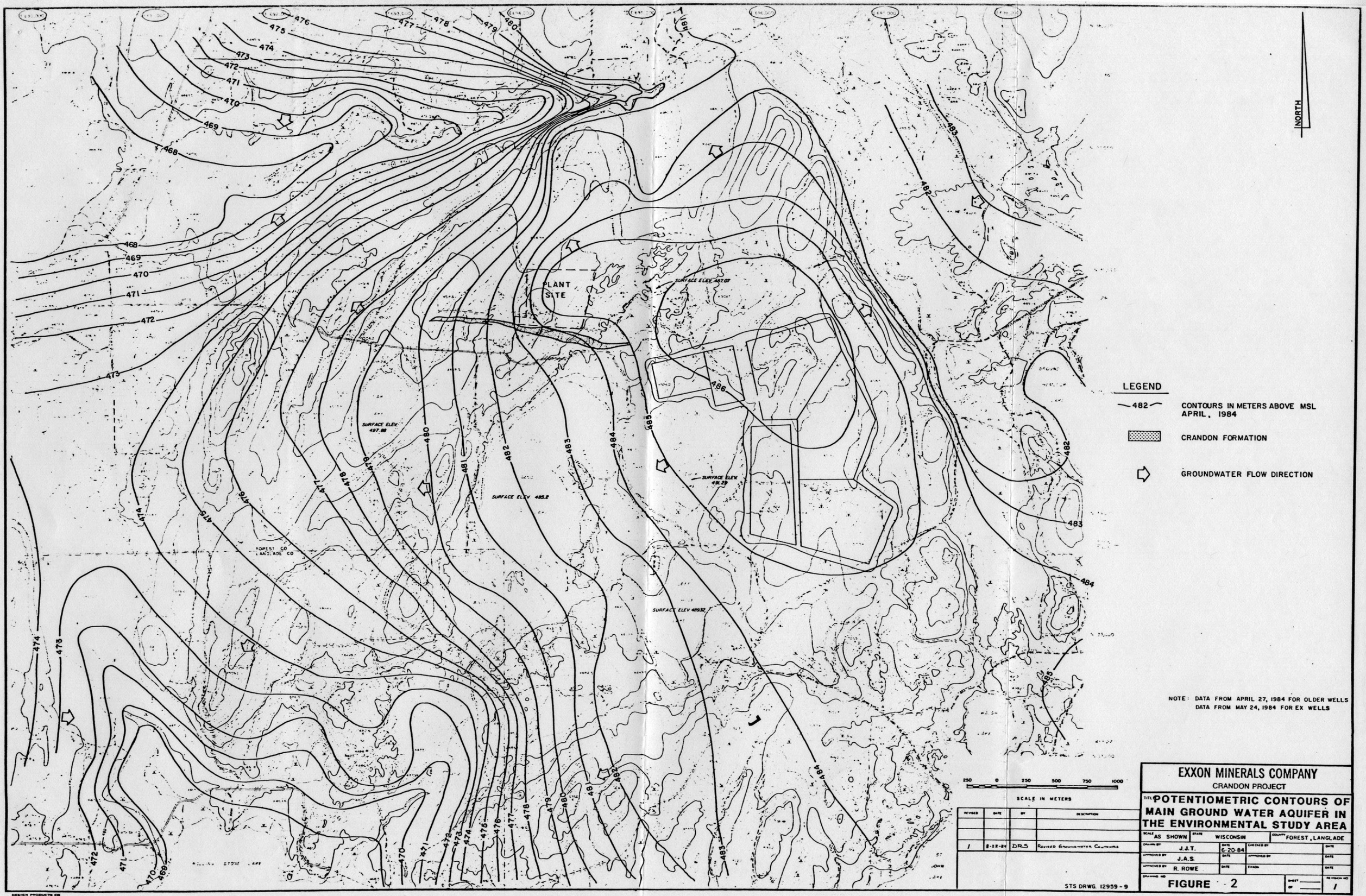
JOB NO. 834-1389	SCALE AS SHOWN	TYPE CURVE COMPARISON	
DRAWN CAB	DATE 1-18-84		
CHECKED JRL	DWG. NO. 050-1-81131		
Golder Associates		EXXON MINERALS COMPANY	FIGURE 1

In some cases, the ground water potentiometric surface was below the bottom of the lake sediment, indicating that an unsaturated zone of soil existed beneath the lake. For this situation, a soil suction potential of 1.5 m was included in the calculation of the difference in heads (Δh), and Δh included three terms: the depth of water, the thickness of the sediment of the deposits, and an estimate of the soil suction potential. Estimates of soil suction potential were taken from the literature (Hillel, 1982).

3.0 ANALYSIS OF GROUND WATER AND LAKE LEVEL DATA

Five nearby lakes in the site area (Oak, Duck, Deep Hole, Little Sand, and Skunk) were monitored for lake levels regularly from April 1977 through November 1980 (Exxon Minerals Company, 1982). Ground water elevations from monitor wells were also measured regularly from May 1977 through September 1980 (Exxon Minerals Company, 1982). Ground water elevation measurements from all wells were taken in April and May 1984, and a potentiometric contour map was developed for the site area (Figure 2). Table 1 summarizes lake elevation data and relevant September 1980 ground water data. The potentiometric contour map shows the lake elevations to be measurably above the potentiometric surface of the ground water aquifer underlying each lake. This difference in water level elevation indicates that these water bodies are hydraulically separated from each other, since connected water sources will seek an equilibrium represented by similar potentiometric elevations. Simply stated, the lake water would have drained into the ground water aquifer if a free-flowing connection existed.

The possibility of erroneous data interpretation due to unusually high lake level elevations, combined with unusually low aquifer potentiometric elevations, was considered. Three lakes (Oak, Duck, and Deep Hole) had lake level vs. aquifer potentiometric level differences of more than 5.5 m (18 feet), which is much greater than recorded ground water fluctuations. Oak Lake had a recorded low surface elevation of 1632.11 feet, approximately 18 m (59 feet) above the aquifer potentiometric surface of September 1980. A monitoring well (DMA-3) about 140 m (460 feet) hydrologically upgradient of the lake was dry to its screened elevation of 1603 feet. Another nearby well



NOTE: DATA FROM APRIL 27, 1984 FOR OLDER WELLS
DATA FROM MAY 24, 1984 FOR EX WELLS

REVISED	DATE	BY	DESCRIPTION
1	8-22-84	DRS	REVISED GROUNDWATER CONTOURS

STS DRWG. 12959-9

EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE: POTENTIOMETRIC CONTOURS OF MAIN GROUND WATER AQUIFER IN THE ENVIRONMENTAL STUDY AREA

SCALE: AS SHOWN	STATE: WISCONSIN	COUNTY: FOREST, LANGLADE	
DRAWN BY: J.J.T.	DATE: 6-20-84	CHECKED BY:	DATE
APPROVED BY: J.A.S.	DATE	APPROVED BY	DATE
APPROVED BY: R. ROWE	DATE	EXXON	DATE
DRAWING NO: FIGURE 2	SHEET OF	REVISION NO: 1	

DESIGN PRODUCTS CO

Table 1

SELECTED WATER SURFACE ELEVATIONS (msl)

Lake	Lake Surface (feet)				Underlying Potentiometric Elevation (feet) September 1980	Lake/Water Table Difference (feet)
	Lowest (date)	Highest (date)	Level Change	September 1980		
Little Sand	1590.82 (7-30-77)	1592.96 (4-20-79)	2.14	1591.80 (9-10-80)	1587	5
Oak	1632.11 (8-03-77)	1634.21 (4-21-79)	2.20	1632.69 (9-11-80)	1574	59
Duck	1610.23 (7-29-77)	1612.32 (7-20-80)	2.09	1611.56 (9-11-80)	1590	22
Deep Hole	1604.96 (7-30-77)	1607.10 (4-21-79)	2.14	1605.79 (9-10-80)	1588	18
Skunk	1596.48 (10-30-77)	1598.26 (5-16-80)	1.78	1597.49 (9-11-80)	1594	3

SOURCE: Exxon Minerals Company (1982).

(DMB-15) has a recorded water table fluctuation of 0.37 m (1.2 feet). These data show that the lake does not have direct hydraulic connection to the aquifer. Duck Lake, with a recorded low surface elevation of 1610.23 feet, is approximately 6.71 m (22 feet) above the aquifer potentiometic surface. The nearest upgradient well (G41-C15B) is about 50 m (164 feet) away and had a recorded water elevation of 1592.4 feet. The greatest ground water elevation change recorded in nearby wells is 0.37 m (1.2 feet) for DMA-4 (approximately 140 m southwest). These data indicate that Duck Lake is not connected to the ground water aquifer.

Deep Hole Lake had a recorded minimum surface elevation of 1604.96 feet. The September 1980 potentiometric surface elevation beneath the lake at its closest approach is about 1588 feet. Monitoring well DMB-27 is situated upgradient of the lake as a whole, but potentiometrically is approximately equal to the aquifer beneath the northeastern edge of the lake. DMB-27 has a recorded high water elevation of 1588.1 feet, which is 5.15 m (16.9 feet) below the lowest recorded lake surface elevations. This large difference is strong evidence that this lake and aquifer are not hydraulically connected.

Little Sand Lake has a recorded low surface elevation of 1590.82 feet. Monitoring well DMP-2 has a recorded aquifer elevation high of 1586.8 feet. This well reflects a potentiometric elevation similar to the aquifer beneath the northeastern edge of the lake, where the aquifer surface most closely approaches the lake surface. This elevation difference of 1.22 m (4.0 feet), while not as great as those of the Oak, Duck, and Deep Hole lakes, shows that the lake and aquifer are not hydraulically well connected.

Skunk Lake's lowest recorded surface elevation was 1596.48 feet on 30 October 1977. In September 1980, the lake elevation was 1597.49 feet and the aquifer potentiometric elevation was approximately 1593 to 1594 feet, or about 1.07 to 1.37 m (3.5 to 4.5 feet) below the lake surface. No upgradient wells are near the lake, so the aquifer potentiometric elevation had to be estimated using other methods. Interpolating the potentiometric contour map gave an estimate of 1594 feet for the eastern (upgradient of the underlying aquifer) edge of the lake. Calculating the water table gradient of two downgradient wells (DMS-2 and DMA-12) and extrapolating the data to the eastern edge of the lake gave an elevation of 1593.3 feet for September 1980. This extrapolated elevation shows the aquifer potentiometric surface to be 1.28 m (4.2 feet) below the lake surface during September 1980. Interpolation of the potentiometric contour map and extrapolation of the aquifer gradient both show the lake and aquifer to have different potentiometric elevations, and they are therefore hydraulically separate.

4.0 LAKE BOTTOM SOILS DATA

During March 1982, STS Consultants installed six wells and sampled lake sediments in Little Sand Lake (STS Consultants, Ltd., 1982). The results of laboratory tests show permeability coefficients ranging from 1.6×10^{-6} to 1.1×10^{-7} cm/s with an average of 4.2×10^{-7} cm/s (1.4×10^{-8} ft/s). Permeabilities of 10^{-5} to 10^{-7} cm/s are considered "very low," and less than 10^{-7} cm/s is considered "practically impermeable" (Spangler and Handy, 1982). During February 1984, the other four lakes had borings drilled by STS Consultants through ice and into the lake bottom. Table 2 presents laboratory testing results for all five lakes, and indicates that the lake bottom sediments exhibit very low permeabilities.

Figures 3 through 7 illustrate the soils classification derived from the boring program for all the lakes. The thickness of the silt and clay deposits beneath each lake is also shown. Table 3 presents the measured thickness of the lake sediments and the depth to ground water from the lake surface for selected wells. The large differences between the lake and ground water levels indicate limited hydraulic connectiveness. The thickness of the nearly impermeable sediment layer beneath each lake is the primary reason for observed differences in water elevations.

TABLE 2

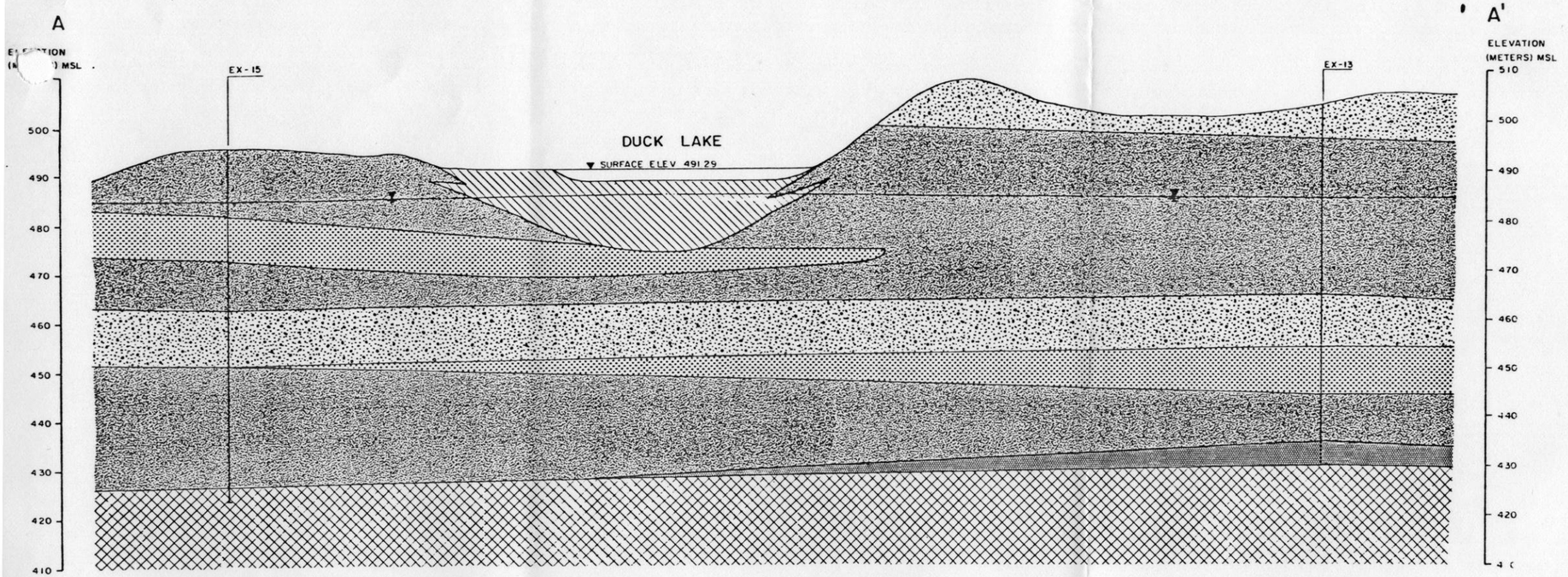
SUMMARY OF LABORATORY TEST RESULTS FROM LAKE DRILLING PROGRAMS
IN DUCK, SKUNK, OAK, DEEP HOLE, AND LITTLE SAND LAKES

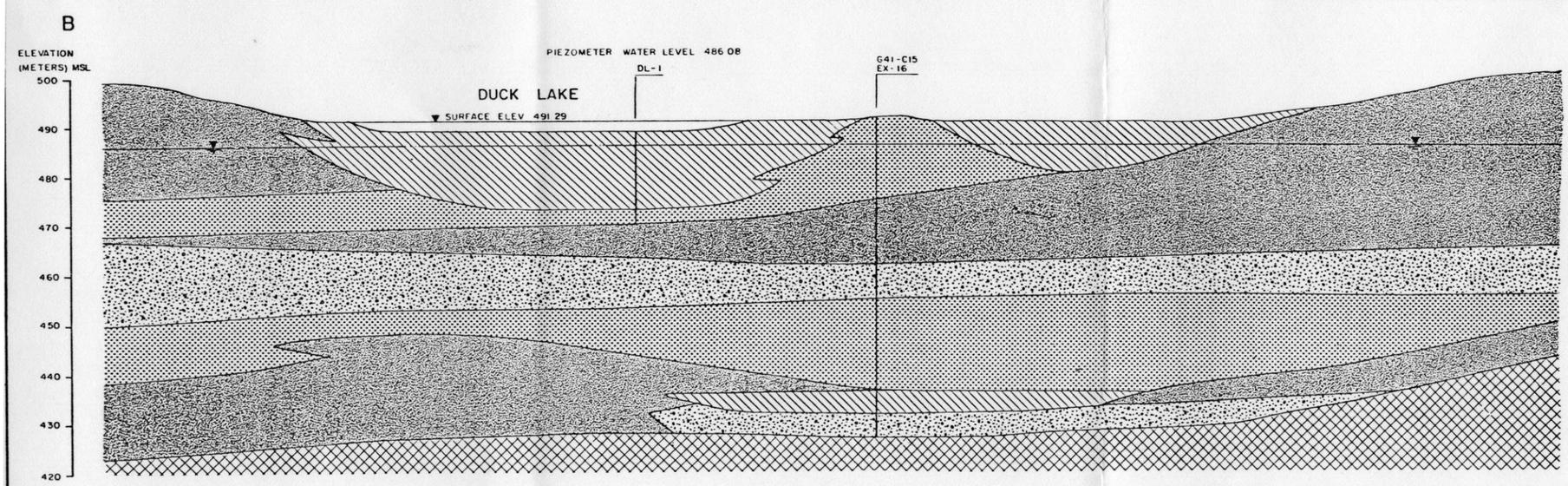
BORING NUMBER	SAMPLE NUMBER	DEPTH (feet)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ORGANIC CONTENT (%)	PERCENT P-200	PERCENT CLAY[a]	ATTERBERG LIMITS (%) LL	PL	PI	USCS CLASSIFICATION	PERMEABILITY COEFFICIENT (cm/s)
DL-1	10	25.5 - 28.0	19.9	104.	--	78.	--	--	--	--	(CL)	1.3×10^{-6}
DL-1	15	38.0 - 40.0	28.7	99.	1.4	93.	--	30.4	21.3	9.1	(CL)	1.0×10^{-7}
SL-2	1A	9.1 - 10.0	20.6	108.	1.8	93.	--	31.9	19.5	12.4	(CL)	4.3×10^{-8}
OL-1	4	37.5 - 39.5	27.2	103.	--	88.	--	--	--	--	(CL)	2.3×10^{-6}
OL-1	6	42.5 - 44.5	32.4	101.	0.7	93.	--	29.2	21.1	8.1	(CL)	3.5×10^{-7}
DHL-1	6	21.0 - 23.0	43.6	95.	1.7	95.	--	28.8	25.0	3.8	(ML)	6.8×10^{-7}
DHL-1	11	33.5 - 35.2	35.2	92.	--	88.	--	--	--	--	(ML)	6.8×10^{-8}
LSL-1	9[b]	30.0 - 32.0	51.5	73.1	--	99.2	14.0	37.1	11.8	25.3	--	1.4×10^{-7}
LSL-1	15	44.5 - 47.0	33.6	88.6	--	94.4	19.0	--	--	--	--	1.7×10^{-7}
LSL-2	5	24.5 - 26.5	36.6	80.0	--	97.7	11.5	--	--	--	--	7.3×10^{-7}
LSL-2	12	39.0 - 40.5	--	--	--	4.6	0.0	--	--	--	--	--
LSL-3	8	28.0 - 30.0	56.3	67.1	--	99.0	19.0	--	--	--	--	3.7×10^{-7}
LSL-3	12	37.0 - 39.0	35.9	87.8	--	99.5	15.0	--	--	--	--	1.5×10^{-7}
LSL-3	15	43.5 - 45.0	--	--	--	4.2	0.0	--	--	--	--	--
LSL-4	9	30.5 - 32.5	43.3	81.0	0.8	96.8	15.0	32.0	15.8	16.2	--	1.1×10^{-7}
LSL-4	16	45.5 - 47.0	--	--	--	18.7	--	--	--	--	--	--
LSL-5	5	20.0 - 22.0	45.7	78.5	--	98.4	17.0	--	--	--	--	1.6×10^{-4}[c]
LSL-5	9	28.0 - 30.0	37.3	84.5	--	96.8	19.0	--	--	--	--	1.5×10^{-7}
LSL-6	4	16.0 - 18.0	107.4	41.8	5.4	93.3	8.0	115.3	62.6	52.7	--	3.7×10^{-7}
LSL-6	10	28.0 - 30.0	34.8	86.8	--	98.3	19.0	--	--	--	--	1.6×10^{-8}

[a]Percent clay based on 0.005 mm size.

[b]Specific gravity = 2.66.

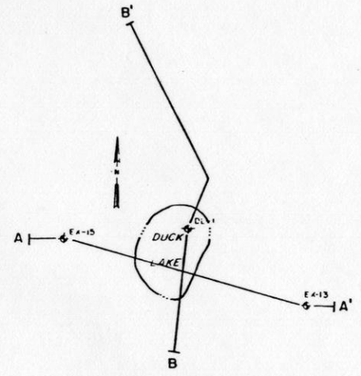
[c]Erroneous permeability coefficient due to side channeling.





LEGEND

- LACUSTRINE
- TILL
- COARSE GRAINED STRATIFIED DRIFT
- FINE GRAINED STRATIFIED DRIFT
- BASAL TILL
- BEDROCK
- LAKE WATER LEVEL
- GROUNDWATER POTENTIOMETRIC SURFACE



CROSS-SECTION INDEX
DUCK LAKE
NOT TO SCALE

NOTE PROJECTIONS OF SOIL STRATA BASED ON DATA FROM BORING LOCATIONS SOIL CONDITIONS BETWEEN BORINGS MAY VARY.

0 50 100 150 200 250
HORIZONTAL SCALE IN METERS
VERTICAL EXAGGERATION 5x

EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE
EXPANDED GEOLOGIC CROSS SECTION OF DUCK LAKE

SCALE	STATE	COUNTY
SHOWN	WISCONSIN	FOREST

DRAWN BY STS	DATE 5/84	CHECKED BY	DATE
APPROVED BY	DATE	APPROVED BY	DATE
APPROVED BY	DATE	EXXON	DATE
DRAWING NO. FIGURE 3		SHEET OF	REVISION NO.

A

ELEVATION (METERS) MSL

530 520 510 500 490 480 470 460 450 440 430 420

G40-Y22

DEEP HOLE LAKE

▼ SURFACE ELEV 489.32

A'

ELEVATION (METERS) MSL

530 520 510 500 490 480 470 460 450 440 430 420

B

ELEVATION (METERS) MSL

510 500 490 480 470 460 450 440 430 420 410

G41-A24

PIEZOMETER WATER LEVEL 484.0

DHL-1

DEEP HOLE LAKE

▼ SURFACE ELEV 489.32

B'

ELEVATION (METERS) MSL

510 500 490 480 470 460 450 440 430 420 410

B'

A'

A

G40-Y22

DEEP HOLE LAKE

DHL-1

G41-A24

B

CROSS-SECTION INDEX
DEEP HOLE LAKE
NOT TO SCALE

LEGEND

- LACUSTRINE
- TILL
- COARSE GRAINED STRATIFIED DRIFT
- FINE GRAINED STRATIFIED DRIFT
- BASAL TILL
- BEDROCK
- LAKE WATER LEVEL
- GROUNDWATER POTENTIOMETRIC SURFACE

NOTE: PROJECTIONS OF SOIL STRATA BASED ON DATA FROM BORING LOCATIONS. SOIL CONDITIONS BETWEEN BORINGS MAY VARY.

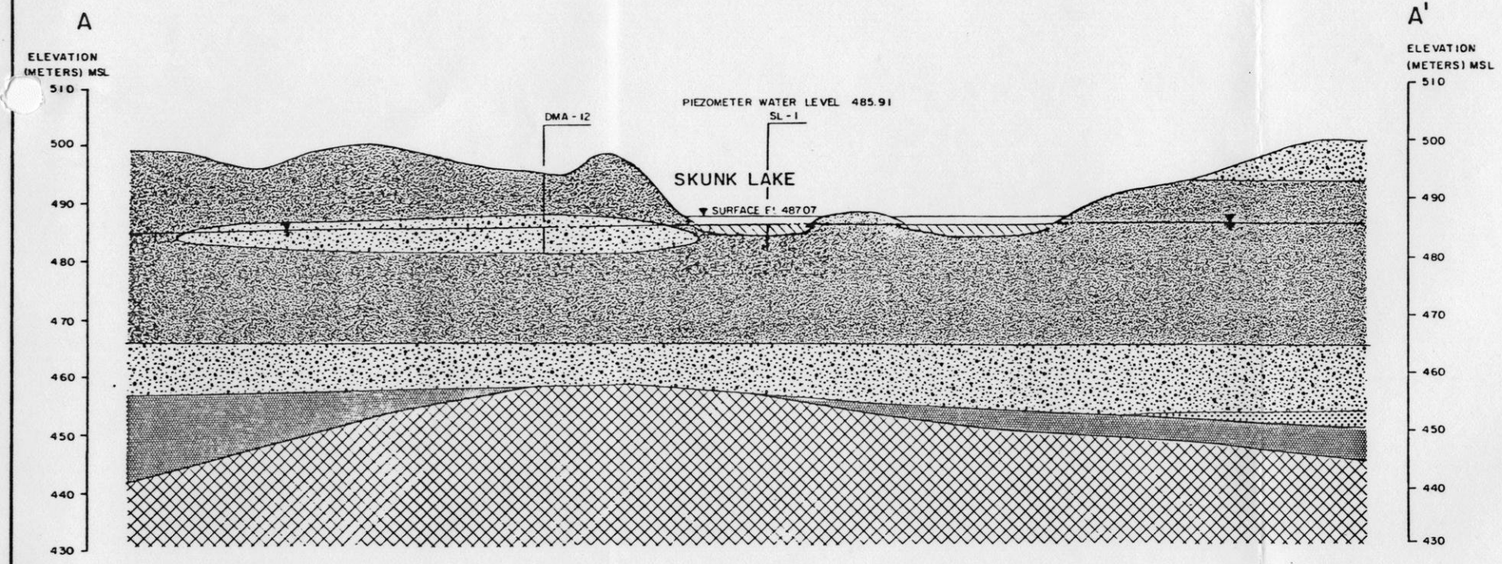
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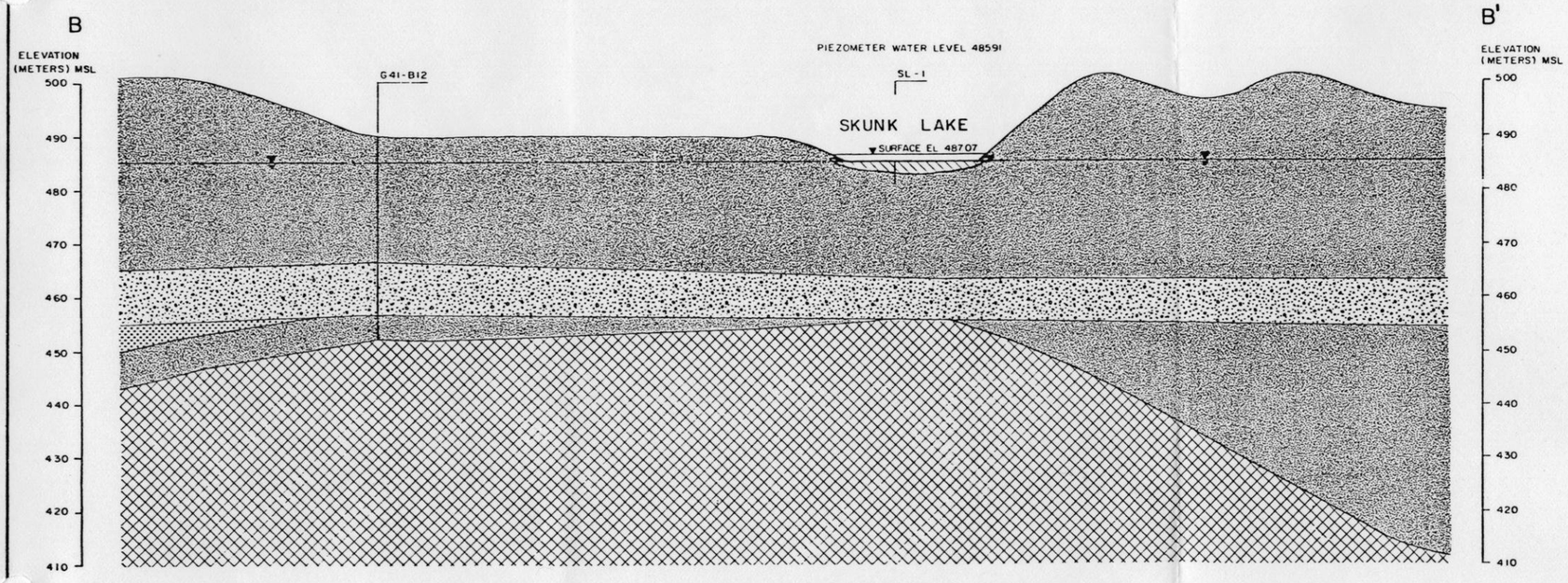
HORIZONTAL SCALE IN METERS
VERTICAL EXAGGERATION 5x

EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE
EXPANDED GEOLOGIC CROSS SECTION OF DEEP HOLE LAKE

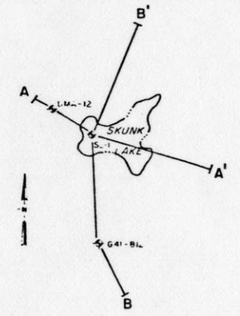
SCALE SHOWN	STATE WISCONSIN		COUNTY FOREST
DRAWING BY STS	DATE 6/84	CHECKED BY	DATE
APPROVED BY	DATE	APPROVED BY	DATE
APPROVED BY	DATE	EXXON	DATE
DRAWING NO. FIGURE 4		SHEET OF	REVISION NO.





LEGEND

- LACUSTRINE
- TILL
- COARSE GRAINED STRATIFIED DRIFT
- FINE GRAINED STRATIFIED DRIFT
- BASAL TILL
- BEDROCK
- LAKE WATER LEVEL
- GROUNDWATER POTENTIOMETRIC SURFACE



CROSS-SECTION INDEX
SKUNK LAKE
NOT TO SCALE

NOTE: PROJECTIONS OF SOIL STRATA BASED ON DATA FROM BORING LOCATIONS. SOIL CONDITIONS BETWEEN BORING MAY VARY

0 50 100 150 200 250
HORIZONTAL SCALE IN METERS
VERTICAL EXAGGERATION 5x

EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE
EXPANDED GEOLOGIC CROSS SECTION OF SKUNK LAKE

SCALE SHOWN	STATE WISCONSIN	COUNTY FOREST
DRAWN BY STS	DATE 6/84	CHECKED BY / DATE
APPROVED BY / DATE		APPROVED BY / DATE
APPROVED BY / DATE		EXXON / DATE
DRAWING NO FIGURE 5		SHEET / OF / REVISION NO

C

ELEVATION (METERS) MSL

510 500 490 480 470 460 450 440 430 420

G40-Y22 G40-Y21 LSL-5 LSL-2 CDM-16

LITTLE SAND LAKE

▼ SURFACE ELEV 485.2

C'

ELEVATION (METERS) MSL

510 500 490 480 470 460 450 440 430 420

A

ELEVATION (METERS) MSL

530 520 510 500 490 480 470 460 450 440 430 420 410

G40-P17 PIEZOMETER WATER LEVEL - DRY OL-1 G40-P10A

OAK LAKE

▼ SURFACE ELEV 497.88

A'

ELEVATION (METERS) MSL

530 520 510 500 490 480 470 460 450 440 430 420 410

A'

G40-P10A

OL-1

OAK LAKE

G40-P17

A

CROSS-SECTION INDEX OAK LAKE

NOT TO SCALE

C'

B

CDM-16 LSL-1 G40-Y15 B'

LSL-2 LSL-3

LITTLE SAND LAKE

G40-S17 LSL-4 A A'

LSL-5

LSL-6 G40-Y21

G40-

C

CROSS-SECTION INDEX LITTLE SAND LAKE

NOT TO SCALE

0 50 100 150 200 250

HORIZONTAL SCALE IN METERS
VERTICAL EXAGGERATION 5x

LEGEND

- LACUSTRINE
- TILL
- COARSE GRAINED STRATIFIED DRIFT
- FINE GRAINED STRATIFIED DRIFT
- BASAL TILL
- BEDROCK
- LAKE WATER LEVEL
- GROUNDWATER POTENTIOMETRIC SURFACE

NOTE: PROJECTIONS OF SOIL STRATA BASED ON DATA FROM BORING LOCATIONS SOIL CONDITIONS BETWEEN BORINGS MAY VARY.

EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE: EXPANDED GEOLOGIC CROSS SECTION OF LITTLE SAND LAKE & OAK LAKE

SCALE: SHOWN | STATE: WISCONSIN | COUNTY: FOREST

DRAWN BY: STS | DATE: 6/84 | CHECKED BY | DATE

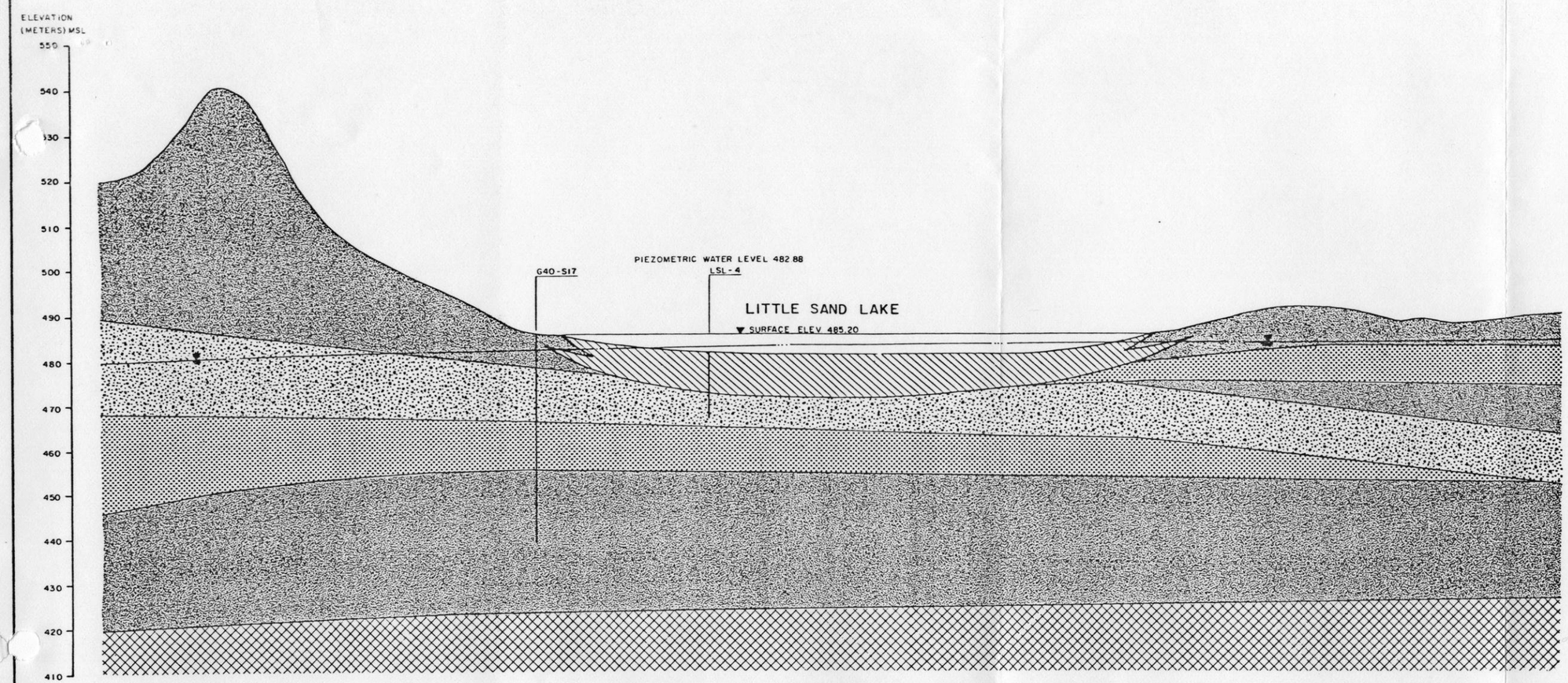
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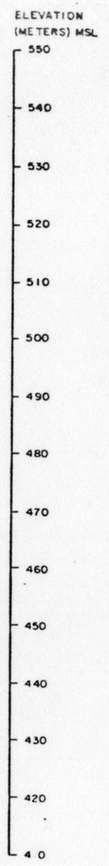
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FIGURE 6

A



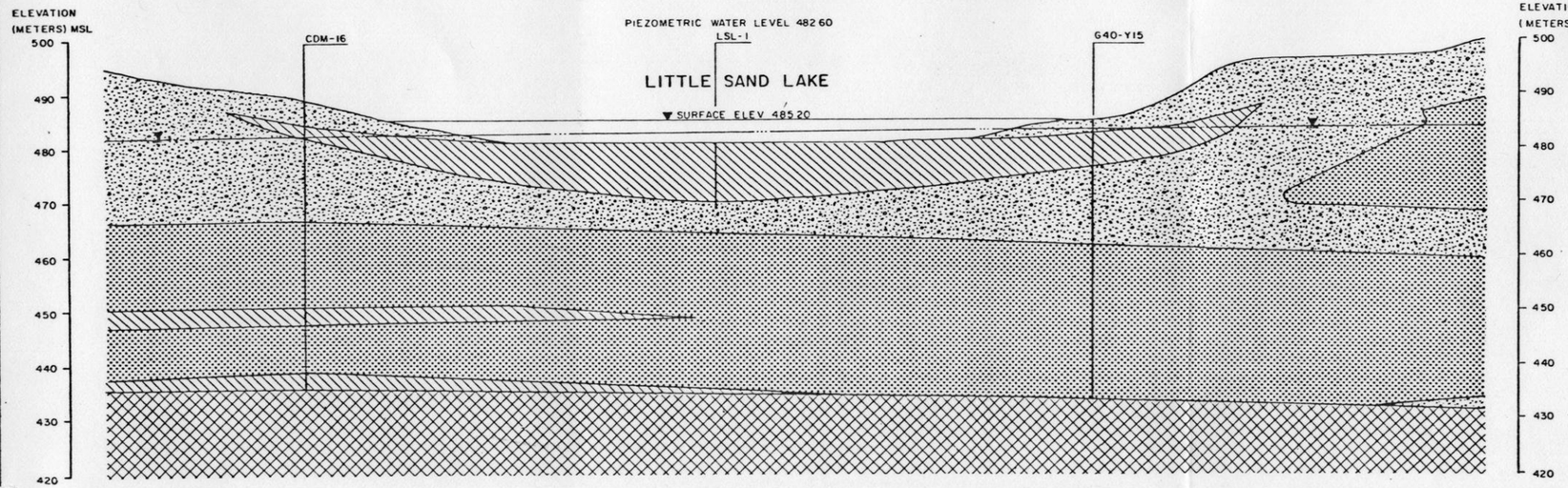
A'



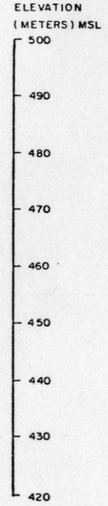
LEGEND

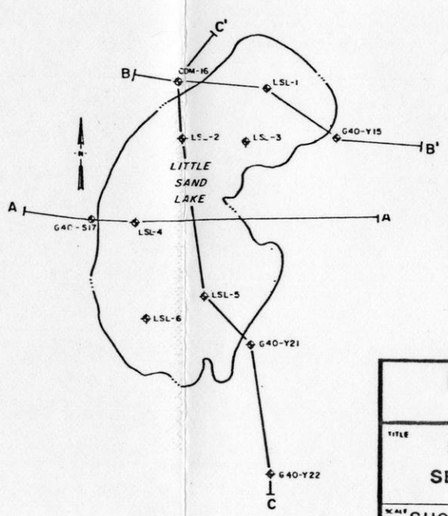
- LACUSTRINE
- TILL
- COARSE GRAINED STRATIFIED DRIFT
- FINE GRAINED STRATIFIED DRIFT
- BASAL TILL
- BEDROCK
- LAKE WATER LEVEL
- GROUNDWATER POTENTIOMETRIC SURFACE

B



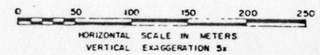
B'





CROSS-SECTION INDEX
LITTLE SAND LAKE
NOT TO SCALE

NOTE: PROJECTIONS OF SOIL STRATA BASED ON DATA FROM BORING LOCATIONS. SOIL CONDITIONS BETWEEN BORINGS MAY VARY.



EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE
EXPANDED GEOLOGIC CROSS SECTION OF LITTLE SAND LAKE

SCALE	STATE	COUNTY
SHOWN	WISCONSIN	FOREST

DRAWN BY	DATE	CHECKED BY	DATE
STS	6/84		
APPROVED BY	DATE	APPROVED BY	DATE
APPROVED BY	DATE	EXXON	DATE
DRAWING NO		SHEET OF	REVISION NO

FIGURE 7

Table 3

LAKE BORING DATA

Lake	Well No. (date)*	Depth to Ground Water From Lake Surface (feet)	Measured Lake Sediment Thickness (feet)	Source
Little Sand	LSL-1 (3-18-82)	8.58 (3-22-82)	37.5	STS, 1982
	LSL-2 (3-22-82)	7.21 (3-23-82)	27.5	STS, 1982
	LSL-3 (3-19-82)		30.5	STS, 1982
	LSL-4 (3-21-82)	8.27 (3-23-82)	30.6	STS, 1982
	LSL-5 (3-20-82)		19.0	STS, 1982
	LSL-6 (3-20-82)	11.05 (3-22-82)	22.0	STS, 1982
Duck	DL-1 (2-08-84)	17.1 (2-28-84)	47.5	STS, 1984
Skunk	SL-1 (2-09-84)	3.8 (2-27-84)	6.3	STS, 1984
	SL-2 (2-27-84)		6.0	STS, 1984
Oak	OL-1 (2-10-84)	Over 44.1 (dry on 2-23-84)	16.9	STS, 1984
Deep Hole	DHL-1 (2-13-84)	18.6 (2-23-84)	26.8	STS, 1984

* Installation date.

5.0 COMPARISON OF SEEPAGE ESTIMATES TO OTHER COMPONENTS OF THE WATER BALANCE

Estimates of seepage times and velocities through the lake sediments for each lake were calculated using Darcy's Law as described in section 2. Travel times are indicative of the time it takes water to leave the lake environment through the sediments. Table 4 shows estimated water seepage travel times for each lake.

Sediments beneath Oak Lake were the most permeable and had the second thinnest sediment sequence and the greatest lake-aquifer head difference (Δh). These combined to produce the fastest seepage travel time, about 2.3 years. Because the complete sequence of bottom sediments of Oak Lake is above the ground water table, any lowering of the table will have no effect on the seepage travel time. Deep Hole Lake had the second fastest seepage travel time (57 years). Duck Lake and Little Sand Lake had seepage travel times of 78 and 95 years, respectively. Skunk Lake had the longest seepage travel time, about 240 years, due to the much lower permeability of the sediments and a very small lake-aquifer head difference, which more than compensated for the thin sediment sequence.

Table 5 shows the amount of seepage for each lake. The relative importance of the numbers can be seen by comparing them to the average annual precipitation in the area (78.16 cm [2.56 feet]). For Little Sand, Duck, Deep Hole, and Skunk lakes, seepage amounts to 11, 24, 18, and 1 percent, respectively, of the precipitation. Since direct precipitation is only one source of water to the lakes, the amount of water lost through seepage is of little importance to the lake levels.

Table 4

SEEPAGE TRAVEL TIME

Lake	Permeability (K) (cm/sec)	Δh (m)	Distance (Sediment Thickness, m)	Time (Years)
Little Sand	1.6×10^{-6}	1.5	8.5	95
Oak*	2.3×10^{-6}	16.0	5.2	2.3
Duck	1.3×10^{-6}	6.6	14.5	78
Deep Hole	6.8×10^{-7}	5.5	8.2	57
Skunk	4.3×10^{-8}	1.0	1.8	240

* For Oak Lake, the potentiometric surface was 3.4 m (11.2 feet) below the bottom of the lake clay. The calculations of Δh used 1.5 m (4.92 feet) instead of 3.4 m (11.2 feet). The 1.5 m (5.0 feet) is a literature estimate of soil suction potential for an unsaturated soil.

Table 5

SEEPAGE INFORMATION

Lake	Barrier Thickness (m)	Lake Area (ha)	Maximum Lake Depth (m)	Seepage			
				Current (cm/yr)	Percent Of Rainfall (%)	Maximum Potential (cm/yr)	Percent Of Rainfall (%)
Little Sand	8.5	100	6.4	8.9	11	95	122
Oak	5.2	21	14.0	230	--	N/C	--
Duck	14.5	11	3.0	19	24	53	68
Deep Hole	8.2	39	3.0	14	18	32	41
Skunk	1.8	2.4	1.8	0.8	1.0	3.7	4.7

N/C = No Change

The maximum seepage possible was also calculated for each lake (Table 5). For this calculation, it was assumed that the ground water would be lowered to a level below the bottom of the lacustrine deposits of each lake. This represents an extreme condition or "worst case" not likely to be encountered in the field. The calculations indicate that seepage will increase for all the lakes with the exception of Oak Lake. The ground water table is below the bottom of the sediments in Oak Lake, and lowering it more does not change the seepage rates. The largest increase in seepage occurs for Little Sand Lake, increasing from 11 to 122 percent of the average annual rainfall. The seepage rates for Duck, Deep Hole, and Skunk lakes remains less than the average annual precipitation, which suggests these lakes will be little affected by even the "worst case." The maximum seepage rates for Little Sand and Oak lakes exceed average annual precipitation by 20 and 152 cm/yr, respectively. Since Oak Lake is presently in equilibrium, it would suggest that the water level in Little Sand Lake would also remain stable.

Lake levels are dependent upon four basic factors: surface flow into the lake, surface flow out of the lake, evaporation, and seepage. A ground water potentiometric surface lower than the lake level indicates that a flow exists through the lake sediment into the ground water. None of the five lakes in the site area has a recorded aquifer potentiometric surface elevation exceeding that of the lake. Dewatering of the mine, and the attendant drawdown of the local aquifer, will only have an effect upon seepage. The other factors are dependent upon environmental conditions unrelated to the project.

Mine dewatering will cause a local lowering of the water table, called a cone of depression. The depth and extent of the cone depend upon

water removal rates, speed of natural aquifer recharge, and interaction between zones of different permeabilities. There is generally less of an effect on the water table upgradient of the removal point than downgradient due to aquifer recharge. Skunk, Duck, and Deep Hole lakes are upgradient of the mine. Little Sand Lake is of generally equal gradient to the mine, and Oak Lake is downgradient.

6.0 CONCLUSIONS

1. Examination of the ground water data indicates that Oak, Duck, Deep Hole, Little Sand, and Skunk lakes are perched (i.e., lake surfaces are positioned above the ground water table). Potentiometric data show clear differences between the various lake surface elevations and the underlying aquifer water table elevations.

2. The permeability of soils beneath all five lakes is classified as very low to practically impermeable. These soils are effective in hydraulically isolating the lakes from the underlying glacial sands and gravels.

3. For existing conditions, water losses through seepage amount to less than 25 percent of the average annual rainfall for each lake, with the exception of Oak Lake.

4. If ground water levels are lowered to below the lake sediments, the amount of water lost through seepage would amount to less than 45 percent of the average annual rainfall for Deep Hole and Skunk lakes, less than 70 percent for Duck Lake, and less than 125 percent for Little Sand Lake. Seepage rates for Oak Lake would be unaffected because the water table is naturally below the lake sediments. Lowering the ground water table will generally have a small effect on lake levels. Seepage losses are small in comparison to other sources of water to the lakes such as rainfall.

5. Oak Lake will not be affected by lowering of the ground water table.

6. A large decrease in ground water levels has the greatest potential to impact Little Sand Lake, particularly during the summer. To determine the magnitude of the impact, a detailed evaluation of data on runoff to the lake is necessary. The impact is not expected to be great.

7.0 REFERENCES

Exxon Minerals Company, 1982, Crandon Project, Environmental Impact Report, Volumes II and VII: Exxon Minerals Company, Rhinelander, Wisconsin.

Hillel, D., 1982, Introduction to soil physics: Academic Press, New York.

Spangler, M.G., and Handy, R.L., 1982, Soil engineering, fourth edition: Harper and Row Publishers, New York.

STS Consultants, Ltd., 1982, Soil boring and laboratory test results of Little Sand Lake drilling project for the Exxon Crandon Project mine development: STS Consultants, Ltd., Green Bay, Wisconsin.

_____, 1984, Lake sediment sampling, Duck, Skunk, Oak and Deep Hole lakes, Crandon Mine Project: STS Consultants, Ltd., Green Bay, Wisconsin.

B. Pamhardt
BEAR/3

State of Wisconsin \ DEPARTMENT OF NATURAL RESOURCES

Carroll D. Besadny
Secretary

BOX 7921
MADISON, WISCONSIN 53707

November 8, 1984 IN REPLY REFER TO: 4400

Mr. Barry Hansen
Permitting Manager
Exxon Minerals Company
P.O. Box 813
Rhinelander, WI 54501

RE: Review of "Hydraulic Relationship Between Site Area Lakes and the Main Groundwater Aquifer, Crandon Project"

Dear Mr. Hansen:

The Department has reviewed the lake report prepared by Dames and Moore for Exxon Minerals, dated September 20, 1984, and received by the Department on September 25, 1984. The Department, in paragraph D on page 7 of its November 14, 1983 letter from Richard Schuff, and in later meetings, has indicated water budgets of the lakes potentially affected by mine dewatering should be prepared. The Dames and Moore report only addresses some of the components of the water budgets. The following comments will provide guidance which should allow Exxon to complete the analysis needed to evaluate mine dewatering impacts upon the project area lakes.

1) A lake water budget consists of determining all inputs and outputs through the lake and any associated changes in storage as reflected in fluctuating lake levels. These components include: incident precipitation, evaporation, lake bottom seepage, surface inflows and outflows. The surface inflows should consider overland flow and channelized stream flow. The Dames and Moore report did not address evaporation, surface inflows, or surface outflows. Seasonal flucuations in the individual components were not discussed. Maximum impacts are likely to occur during a season of high evaporation and low precipitation following a year when lake storage (lake level) is also at low levels. This scenario should be evaluated.

2) The evaporation component should be evaluated using field data gathered from other lakes studied in Northern Wisconsin. Lake level data from the Crandon Project Lakes should be correlated with other lake levels and corresponding evaporation rates. The USGS, WGNHS and University of Wisconsin may be sources of information. Research conducted at Trout Lake Biological Station north of Woodruff may be a source of information. Annual fluctuations as well as average values should be considered.

3) The evaluation of hydraulic connection between the lake and aquifer should have utilized the potentials measured during the two lake boring programs. The Dames and Moore report (Table 1) utilized hydraulic potentials interpolated from groundwater monitoring wells located as much as 1200 feet from the lake. It is unclear why interpolated values were used in Table 1 but actual potentials measured during the lake boring programs were used later in the report in a correct fashion to evaluate seepage rates.

4) The thickness of lake sediments acting as a seepage barrier could be more precisely delineated by examining the boring logs in detail. It is most probable that the firm silt or clay sediments are controlling seepage. The thickness of loose organic rich sediments will be an insignificant barrier to seepage. The sediment thickness, l, used in seepage calculation should be reduced to include only the firm low permeability sediments. (Refer to attachment 1 for guidance in evaluating this parameter).

5) Based on the interpolation procedure discussed in point 3, Dames and Moore conclude Duck, Skunk and Deep Hole Lakes are not hydraulically connected to the groundwater aquifer. The Department believes lake borings made in February 1984 demonstrate the lakes are hydraulically connected. The reasoning is flawed on page 6 of the report, where differences in hydraulic potential between the lake surface and nearby aquifer is said to indicate that a hydraulic discontinuity exists between them. It should be noted that as long as a saturated connection exists between the aquifer and the lake, the potential differences will cause a movement of water between the two ie. a hydraulic connection. The effect of the increase of this potential difference due to mine dewatering is the reason for the Department having requested Exxon prepare a lake water budget.

6) The calculation of seepage travel times as expressed in Table 4 is incorrect. The specific discharge was calculated, not seepage velocity (average linear velocity as used in Freeze and Cherry, 1979). The seepage velocity can be determined by dividing the specific discharge by the effective porosity of the seepage controlling lake sediments. The effectivity porosity of these lake sediments is probably less than 0.25. This would produce seepage velocities four or greater times those used to calculate the travel times listed, thus producing travel times of 25% or less than those listed. It is unclear what the usefulness of the calculation of travel times is in evaluating the water budget of the lakes.

7) Laboratory permeability tests were used to establish values for hydraulic conductivity used in seepage estimates. Disturbance of the soil during sampling and the presence of soil heterogeneities in the field are uncertainties which indicate actual field permeability may be significantly greater than the laboratory values used. It is likely actual in situ permeability is greater than the laboratory measurements. For evaluation purposes a range of hydraulic conductivity values which start at 1×10^{-6} cm/sec (1.03 ft/yr) would be appropriate.

8) The report's conclusion that lake seepage losses during mine dewatering are small in comparison to other sources is unsupported. Even assuming a average non-drought year, the lake seepage was calculated to be as much as 125% of precipitation. If a precipitation value corresponding to a drought year was used the percentage would be increased further. A worst case analysis will probably need to assume no surface water flows into the lakes during drought years. In a lake such as Little Sand Lake, where there would be no surface outflow during periods of drought, the water budget would simplify to the evaporation, precipitation and seepage components. Average and worst case analysis for utilizing the above concepts should be made for Little Sand, Deep Hole, Skunk and Duck Lakes.

   A simplification of the water budgets of Little Sand, Skunk, Duck, Deep Hole, and Oak Lakes should be utilized to determine the maximum lake sediment conductivity value which would still sustain present lake levels. The maximum precipitation and inflow values possible, coupled with minimum evapotranspiration values, should be used to compute a sediment conductivity value which will allow complete seepage of all lake inputs. This conductivity value should then be used to compute the maximum lake seepage which would occur during worst case mine dewatering conditions.

9) The Department has prepared a lake seepage analysis utilizing the existing data which is presented in attachment 1. The Department's consultant, Dr. Kenneth Bradbury of the Wisconsin Geological and Natural History Survey, has also provided criticism of the lake seepage report, which is included as Attachment 2.

10) The Department has conducted field investigations with its hydrogeological consultants in order to evaluate reports of springs along the west edge of Little Sand Lake. The investigation indicates there are no active springs or indications of past spring activity in Little Sand Lake. A description of this investigation is presented in Attachment 3.

11) It has been assumed that Rolling Stone Lake will not be affected by reductions in groundwater discharge due to mine dewatering. Exxon must document this by comparing the modeled reductions in flow to the lake compared to other lake budget components. Seasonal low flows must be considered. The "Inland Lake Renewal Feasibility Study for Rolling Stone Lake, 1977," prepared by Northern Lakes Service Inc. may be of use in this analysis.

12) The impacts associated with the potential reductions in lake levels due to mine dewatering indicates the need for the preparation of hydrographic maps of Little Sand, Skunk, Duck, and Deep Hole Lakes. Exxon shall prepare maps with a one foot minimum contour for the upper five feet of the lake bottom. The contour internal can be increased to larger values for the remaining bottom area. Public access points should be indicated.

13) In order to evaluate the maximum lake storage possible, Exxon shall determine the elevation of lake level controlling structures. These elevations should be surveyed to a 0.1 foot accuracy and a description of the controlling structure should be provided. The lake level controlling structure elevation should be compared to historic lake level maximums.

14) The potential for significant lake level reductions indicates the need for evaluation of appropriate mitigative measures. These should include lake water supplements and the use of a hydraulic barrier formed by reinjection of clean groundwater intercepted before flowing into the lower mine workings. Though there is no assurance of Department approval of such measures at this time the EIR review process could be expedited if the hydraulic barrier concept was evaluated using a varying array of injection wells and pumping rates simulated through use of the GEOFLOW model. Secondary impacts associated with both of these mitigative measures should be addressed.

As can be seen from Departmental analyses, the increases in lake seepage due to mine dewatering are significant. These analyses point out the need for further analysis of the lake water budget and specifically indicates that seepage rates are very sensitive to sediment permeability. Please contact Kenneth Wade, Bureau of Solid Waste hydrogeologist at (608) 267-9387 regarding questions about this letter.

Sincerely,
Bureau of Solid Waste Management

Richard G. Schuff, P.E., Chief
Residuals Mgt. & Land Disposal Section

Bureau of Water Supply

William T. Rock, P.E., Chief
Private Water Supply Section

KW:ms/4489Q

Attachments 1, 2, and 3

cc: A. Wilson - NCD
R. Gerhardt - WS/2
L. Wible - ADM/5
K. Bradbury - WG&NHS
J. Krohelski - USGS
M. Anderson - U.W.-Geology Dept.
B. Ramharter - BEAR/3
C.W. Fetter - U.W.-Oshkosh
Systems Management Section
Bill Krug - USGS
Native American Coord. EE/5
Dale Simon - WR&Z/5
Bob Read - BEAR/3
Bob Sonntag - WR&Z/5
James Derouin - Madison
Kevin Lyons - Milwaukee
Don Zuidmulder - Green Bay
Kathy Falk - DOJ
Gordon Reinke - SW/3
SUSAN STEINGASS - MADISON

Attachment 1

Evaluation of Seepage From Little Sand, Duck Skunk, Oak, and Deep Hole Lakes

The analysis of lake seepage is dependent on the following three parameters: thickness of lake sediments controlling seepage, difference in hydraulic potentials between the lake and the underlying aquifer, and the hydraulic conductivity of the lake sediments.

I. Thickness of Lake Sediments

Examination of the lake borings indicates soft loose black organic silts (OL) make up the upper sediments. The lower sediments tend to be firm silt or silty clay (ML, CL) soils. Only the lower firm soils will be significant in controlling seepage due to their lower conductivity. The table below summarizes the sediment type for the Little Sand Lake (LSL) borings.

Boring Name	LSL1	LSL2	LSL3	LSL4	LSL5	LSL6	Average
Loose, soft sediments (ft)	16.5	12.0	14.5	12.0	3.0	8.0	11.0
Firm sediments (ft)	21.0	15.5	16.0	26.0	26.0	14.0	18.1
Total thickness (ft)	37.5	27.5	30.5	33.0	19.0	22.0	28.25
Difference in lake/aquifer potentials (ft)	8.54	7.21	-	8.27	-	11.24	8.82

Inspection of boring logs gave the following values for thickness of seepage controlling sediments for the other lakes: Duck Lake (20 feet), Skunk Lake (1.5 - 2.0 feet), Oak Lake (13.0 feet), and Deep Hole Lake (17 feet). The existing hydraulic potential differences between the lake and the underlying aquifer are given for those borings for which they could be calculated.

II. Difference in Hydraulic Potentials Between Lake And Underlying Aquifer

The existing potential was taken from the piezometric water levels measured during the lake boring programs and the measured lake surface elevations. The maximum potential difference which could be developed during mine dewatering is determined by examining the boring logs to find the lowest elevation of the seepage controlling lake sediments.

The lowest elevation represents the minimum potential because if groundwater levels are lowered below this point unsaturated conditions should begin to develop below the sediments and hydraulic continuity will be broken between the lake and groundwater aquifer. The following table summarizes the existing and maximum potential differences at the project area lakes:

Lake Name	Little Sand	Duck	Skunk	Deep Hole	Oak
Surface elevation (ft) MSL	1591.91	1611.8	1598.0	1605.4	1633.5
Bottom of seepage controlling sediments (minimum potential)	1542.41	1557.0	1587.5	1570.0	1584.5
Existing aquifer potential	1583.09	1594.7	1594.0	1587.9	1584.5
Existing difference in potentials	8.82	17.1	4.0	17.5	49.0
Maximum difference in potentials	49.5	54.8	10.5	35.4	49.0

III. Hydraulic Conductivity of Lake Sediments

Laboratory measurements of lake sediment samples obtained by the boring programs gave conductivity values ranging from 1.3×10^{-6} cm/sec to 1.6×10^{-8} cm/sec. Obtaining and testing the samples in the laboratory is likely to cause some disturbance of the sample resulting in compaction which can produce conductivity values less than those of the actual in-field material. Heterogeneities within the sediments will probably produce a bulk permeability which is greater than that produced by individual tests of the lab specimens. Because of these uncertainties a range of hydraulic conductivity values from 1×10^{-6} cm/sec (1.03 ft./year) to 1×10^{-7} cm/sec (0.10 ft./year) will be used for analysis of lake seepage.

IV. Lake Seepage Calculations for Existing and Dewatering Conditions

Darcy's Law for saturated flow will be used as follows:

$$q = K \frac{\Delta h}{l}$$

q = specific discharge (seepage rate)
K = hydraulic conductivity
Δh = difference in potentials
l = thickness of seepage controlling sediments

A. Little Sand Lake Seepage Rates

Seepage Conditions	Existing		Maximum/Dewatering	
Sediment Conductivity	low	high	low	high
Sediment Thickness (ft)	18.1	18.1	18.1	18.1
Hydraulic Conductivity (ft/yr)	0.10	1.03	0.10	1.03
Potential Differences (ft)	8.8	8.8	49.5	49.5
Seepage Rate (ft/yr)	0.05	0.5	0.28	2.8
Maximum Potential Seepage Increase During Dewatering%			460%	

B. Duck Lake Seepage Rates

Seepage Conditions	Existing		Maximum/Dewatering	
Sediment Conductivity	low	high	low	high
Sediment Thickness (ft)	20.0	20.0	20.0	20.0
Hydraulic Conductivity (ft/yr)	0.10	1.03	0.10	1.03
Potential Differences (ft)	17.1	17.1	54.8	54.8
Seepage Rate (ft/yr)	0.09	0.88	0.28	2.8
Maximum Potential Seepage Increase During Dewatering%			218%	

C. Skunk Lake Seepage Rates

Seepage Conditions	Existing		Maximum/Dewatering	
Sediment Conductivity	low	high	low	high
Sediment Thickness (ft)	1.5	1.5	1.5	1.5
Hydraulic Conductivity (ft/yr)	0.10	1.03	0.10	1.03
Potential Differences (ft)	4.0	4.0	10.5	10.5
Seepage Rate (ft/yr)	0.18	1.83	0.72	7.2
Maximum Potential Seepage Increase During Dewatering%			293%	

D. Deep Hole Lakes Seepage Rates

Seepage Conditions	Existing		Maximum/Dewatering	
Sediment Conductivity	low	high	low	high
Sediment Thickness (ft)	17.0	17.0	17.0	17.0
Hydraulic Conductivity (ft/yr)	0.10	1.03	0.10	1.03
Potential Differences (ft)	17.5	17.5	35.4	35.4
Seepage Rate (ft/yr)	0.10	1.1	0.21	2.1
Maximum Potential Seepage Increase During Dewatering%			91%	

E. Oak Lake Seepage Rates

Seepage Conditions	Existing and Maximum/Dewatering	
Sediment Conductivity	low	high
Sediment Thickness (ft)	13.0	13.0
Hydraulic Conductivity (ft/yr)	0.10	1.03
Potential Differences (ft)	49.0	49.0
Seepage Rate (ft/yr)	0.39	3.9
Seepage Increase During Dewatering%		0%

It should be noted that the maximum seepage rates possible can only be achieved if the underlying aquifer is lowered to the bottom of the seepage controlling lake sediments. The Department has used the maximum mine inflow computer model simulation (Case II, D'Appolonia, 1982) to evaluate the maximum drawdown relative to the sediment bottom elevation. The following table presents this data:

Lake	Maximum Aquifer Drawdown (ft)	Existing Water Table (ft)	Resultant Water Table (ft)	Elevation of bottom of sediments (ft) (ft)
Little Sand	20-59	1578-1585	1526-1558	1542
Skunk	16-20	1592-1595	1572-1579	1588
Duck	20-23	1590-1594	1570-1572	1557
Deep Hole	13-16	1583-1588	1567-1575	1570

The above analysis shows that the maximum drawdown will lower the water table of Little Sand, Skunk, and Deep Hole Lakes to the bottom of the sediments and so indicates their maximum lake bed seepage could potentially increase to that produced by water table declines to the bottom of their lake bed sediments. The sediments of Duck Lake continues13 to 15 feet below the maximum drawdown elevation so maximum seepage head differences will not occur at this lake. The actual maximum seepage rates for Duck Lake listed in Table B should be reduced by 25% to reflect the fact that the water table will not be lowered below the sediments.

4489Q

ATTACHMENT #2.

WGNHS GEOLOGICAL AND NATURAL HISTORY SURVEY
1815 University Avenue, Madison, Wisconsin 53706 608-262-1705

MEMO

to: Ken Wade, Dept. of Natural Resources date: 11/5/84

from: Ken Bradbury KRB

Re: Review of Report: Hydraulic Relationship Between Site Area Lakes and the Main Ground Water Aquifer, Crandon Project. Prepared by Dames and Moore for Exxon Minerals Company, Dated Sept. 20, 1984.

The Dames and Moore report is a rather brief summary of information already collected about the Exxon Project Area lakes. It provides no new information or data.

Having been present at a series of meetings between Exxon and the DNR during late 1983 and 1984, it is clear to me that this report does not fulfill the request made by DNR at those meetings. DNR requested Exxon to provide detailed water budgets for all lakes near the mine workings. The budget components to be addressed included precipitation, surface water runoff, evaporation, seepage, and surface water inflows or outflows for each lake. My notes from a meeting in Madison on 12/20/83 indicate that Exxon agreed verbally to provide water budgets for all lakes having no unsaturated zone beneath them, and to provide a detailed cross section utilizing the CDM piezometers in the northwest corner of Little Sand Lake. The Dames and Moore report clearly does not fulfill this request.

Detailed comments follow:

Page 1, Par. 1. Lines 5 and 7. The phrases "movement of water...is limited." and "very restricted hydraulic connection." are so vague as to be meaningless. Is groundwater movement "limited" to small amounts, or is it "limited" by some physical barrier? The term "hydraulic connection" (line 7) is vague and undefined.

Page 1, Par. 2., Line 1. "...degree of hydraulic connection..." is an undefined phrase. It is not clear whether "hydraulic connection" refers to continuity of hydraulic gradients, continuous saturation, movement of water, or to something else. For example, water in the unsaturated zone is usually in hydraulic continuity (by rigorous definition) with water in the saturated zone.

Page 2, Last Line. The time-of-travel equation gives specific discharge, not average linear velocity. Average linear velocity (specific discharge divided by porosity) is the appropriate velocity for time-of-travel calculations. In any case it is not clear what bearing time-of-travel calculations have on lake seepage impacts.

Page 3, First Paragraph. Only in Oak Lake is the groundwater potentiometric

surface below the bottom of the lake sediments. The reference to Hillel (1982) is unclear. How was soil suction potential determined? What characteristic curve was used? What water content was used?

Page 4. First Paragraph, lines 11-13. The statement "...difference in water level elevation indicates that these water bodies are hydraulically separated from each other, since connected water sources will seek an equilibrium represented by similar potentiometric elevations." is misleading and generally not true. The difference in water levels indicates only that a hydraulic gradient exists beneath the lakes. If the sediments beneath the lakes are saturated (as they are, with the exception of Oak Lake) then the lakes are hydraulically connected to the groundwater system. The object of Exxon's analysis should have been to quantify the amount of water exchanged through this connection. This amount is expected to be small due to the low permeability of the lake sediments. But the idea that the lakes in the project area are somehow "hydraulically separated" from the rest of the groundwater system is a misconception of the facts.

Pages 4-6 The discussion of "hydraulic connection" based upon differences between lake levels and water levels in wells near the lakes is not relevant. To compare lake levels with well levels up to 140 m away is simply not correct. Such a comparison ignores the effects of horizontal and vertical hydraulic gradients which are often very steep near lakes. A better source of data would have been the lake-bottom piezometric investigations conducted by STS Consultants but not mentioned in this section.

Page 7, Paragraph 2, Line 6. The phrase "...limited hydraulic connectiveness ..." is vague and undefined.

Page 11, Paragraph 1. The statement that Duck, Deep Hole, Little Sand, and Skunk Lake are perched above the water table is incorrect. All the data available to me suggest that the water table (ie, the top of the saturated zone) and the lake surfaces are coincident for these lakes. There is no unsaturated zone beneath these lakes. The observed hydraulic gradients through the fully saturated lake sediments, not to the presence of some hydrologic discontinuity beneath the lakes.

Page 11, Paragraph 6. This statement appears to recommend that a study of runoff to Little Sand Lake is necessary. I believe the DNR intent in requesting this report was for Exxon's consultant to prepare detailed water budgets for all lakes in the project area.

ATTACHMENT #3.

WGNHS GEOLOGICAL AND NATURAL HISTORY SURVEY
1815 University Avenue, Madison, Wisconsin 53706 608-262-1705

MEMO

to: Ken Wade, Jim Krohelski, Roger Gerhardt, Mary Anderson
date: Oct. 22, 1984
from: Ken Bradbury KRB
Re: Summary of field observations at Little Sand Lake, October 1984

On October 9-10, 1984 Ken Wade, Jim Krohelski, and myself conducted a reconnaissance field study of Little Sand Lake. Our objectives were to attempt to locate lake-bottom springs reported by local residents and to determine hydraulic gradients through the lake bed in near-shore areas.

This memorandum is a summary of the observations we made, with some interpretive calculations.

I. Spring Survey

We attempted to confirm reports that springs are present in the bottom of Little Sand Lake. I had received one detailed telephone description of a spring from Mr. Richard Webb who owns a summer cottage on the lake but resides elsewhere. Mr. Webb gave us permission to visit his property, and suggested that springs occurred in a small grassy lake-bottom depression about 40 feet offshore of his property.

We located Mr. Webb's cottage, and did observe two grassy depressions in the sandy lake bed about 40 feet offshore from his property (see attached map). These depressions were circular, about 3 feet in diameter, and in about 2 feet of water. The depressions were about 6 inches deep.

We tested the water in and around these depressions for temperature and electrical conductivity. We were not able to detect any thermal or chemical changes in or near these areas, nor did we see any visual evidence of groundwater discharge (sand boils, for example). We also installed seepage meters and mini-piezometers in the depression area (see below). Data from these devices did not suggest that groundwater discharge was occurring.

We also met Mr. Tom Volmar at the lake. He pointed out an area in the northwest corner of the lake where he remembered encountering very cold water while swimming. Once again we were unable to locate or observe any active springs in this area. Several other residents who we talked to at the lakeshore also said they had often encountered cold areas while swimming, wading, or fishing in the lake, and that they believed these areas were spring discharge points. However, none of these people could direct us to an exact spot, other than again mentioning the general vicinity of Mr. Webb's cabin. In addition, these people stated that they had not observed any physical evidence for groundwater discharge, such as sand boils, in colder areas.

SUMMARY: Although we visited locations where several residents had reported springs in Little Sand Lake, we were unable to locate any active springs in the lake bottom.

II. Seepage Meter Results

We installed four seepage meters in the vicinity of Mr. Webb's cabin to attempt to measure groundwater flow into or out of the lake bottom. The seepage meters were located in 2 to 3 feet of water in the vicinity of the grassy depressions about 40' from shore (see sketch). After installation, each seepage meter bag was filled with 500 ml of lake water and the meters were left overnight, with the following results:

Elapsed Time: 15.25 hours

Meter	Gain or Loss (ml)	Rate ml/hr.
#1 (30 feet from shore)	-140	- 9
#2 (40 feet from shore)	+145	+10
#3 (40 feet from shore)	- 35	- 2
#4 (50 feet from shore)	+100	+ 7
Average	+ 70	+ 5

The seepage meter data are obviously somewhat contradictory. From my personal experience using seepage meters, I conclude that the observed variations are mainly due to errors in measurement and to variations in seepage meter performance, and that the net bottom seepage in the tested area is essentially zero.

III. Vertical Permeability Test

We measured the vertical permeability of the lake bed using a thin-walled metal casing (0.4 feet diameter) driven vertically 1.5 feet into the bottom sediments in about 1 foot of water. Inside the casing we installed a small piezometer with a very short screened opening at the level of the bottom of the outer casing. We filled the casing with water and observed the water level fall for over one hour. This falling-head test gave a vertical permeability of 1.6×10^{-4} cm/sec.. The material tested was a poorly sorted silty till, which occurred about 1 foot beneath the lake bed.

IV. Lake Bed Hydraulic Gradients

We measured hydraulic gradients through the lake bottom using mini-piezometers installed at 6 sites (see map). Each mini-piezometer was allowed to stabilize overnight prior to measurement. Results are as follows:

Mini-piezometer No.	Head (relative to lake) (ft)	Penetration (ft)	Gradient
1 (Northwest Shore)	0.00	1.4	0.00
2 Beach (Outer)	-0.01	3.0	-0.003
3 Beach (inner)	-0.09	0.9	-0.10
4 Webb (outer)	-0.16	2.0	-0.08
5 Webb (inner)	-0.73	2.6	-0.28
6 (South Shore)	-0.23	1.9	-0.12
Average			-0.10

Measured gradients range from 0.00 in the northwest corner of the lake to -0.28 (downward) at the Webb property. The average gradient was -0.10 (downward). The mini-piezometers provide very convincing evidence that Little Sand Lake loses water by downward seepage along its western shore.

V. Nearshore Lake Bed Materials

In order to observe geologic materials present in the lake bed, we bored several shallow holes near the Webb property using a hand auger. The logs at two of these holes are as follows:

Hole #1
Location: 40 feet offshore of Webb cottage, in 3 feet of water

Depth	Material
0 - 3 ft	Sand, fine to medium, well sorted
3 - 3.5 ft	Silt, grey, organic

Hole #2
Location: Onshore, 10 feet from shoreline

Depth	Material
0 - 5 ft	Sand, silty, with pebbles (till)

Hole 2 encountered the water table at 2.2 feet below the land surface and 0.11 feet below the lake water surface, giving a horizontal hydraulic gradient of 0.11/10 = 0.01 westward.

VI. Other Observations

We also measured depth to water in several Exxon wells, with the following results:

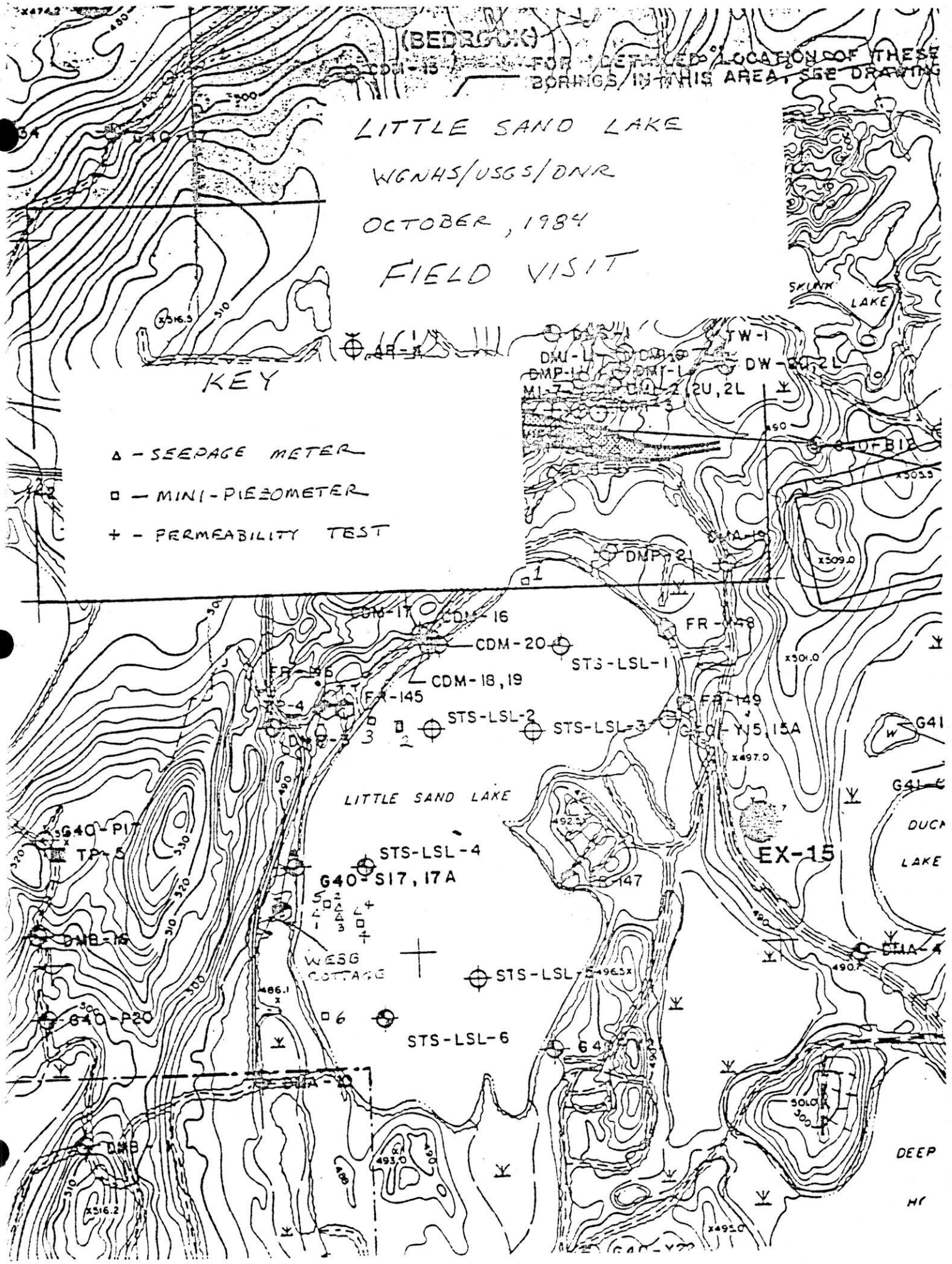
Well	Depth to water
G40-S17	20.98 feet
G40-S17A	19.03 feet
CDM 16	25.54 feet
CDM 18	Dry
CDM 19	15.15 feet
CDM 20	4.00 feet
CDM 17	Dry

In addition, we located the surface water outlet to the lake in a weedy area in the southwest corner of the lake, and followed a well-defined channel several hundred feet back into the woods. We were not able to locate any berm, dam, or obstruction which might be the main surface water level control. We did not detect any flow in the outlet channel.

VII. Summary

On October 9 and 10, 1984 we undertook a rather qualitative survey of groundwater-surface water relationships along the western shore of Little Sand Lake. We found no springs. We measured consistently downward vertical hydraulic gradients through the lake bottom, which indicates that the lake currently loses water through groundwater seepage. Along the western shore, the water table apparently slopes westward, away from the lake.

KRB:kz


(BEDROCK)
CDM-13
FOR DETAILED LOCATION OF THESE BORINGS IN THIS AREA, SEE DRAWING
LITTLE SAND LAKE
WGNHS/USGS/DNR
OCTOBER, 1984
FIELD VISIT
KEY
Δ - SEEPAGE METER
□ - MINI-PIEZOMETER
+ - PERMEABILITY TEST
SKUNK LAKE
AR-1
TW-1
DMW-1
DMP-1
DW-2,2U,2L
MI-7
DMP-2
CDM-16
CDM-17
CDM-20
STS-LSL-1
CDM-18,19
FR-145
FR-146
FR-148
FR-149
STS-LSL-2
STS-LSL-3
G-01-Y5,15A
LITTLE SAND LAKE
G40-P17
TP-5
STS-LSL-4
G40-S17,17A
DMB-16
WESG COTTAGE
STS-LSL-5
G40-P20
STS-LSL-6
64
DMA
DMB
EX-15
DUCK LAKE
DEEP
HI

PRECONSTRUCTION REVIEW AND PRELIMINARY DETERMINATION ON THE PROPOSED CONSTRUCTION, OPERATION AND RECLAMATION OF AN UNDERGROUND ZINC/COPPER/LEAD MINE, ORE PROCESSING MILL AND ASSOCIATED SURFACE FACILITIES FOR EXXON MINERALS COMPANY TO BE LOCATED FIVE MILES SOUTH OF CRANDON FOREST COUNTY, WISCONSIN

This review was performed by the Wisconsin Department of Natural Resources, Bureau of Air Management in accordance with Sections 144.391 to 144.394, Wisconsin Statutes. This review is for the construction of a minor source located in (or significantly affecting) an area which is attainment/unclassified for all pollutants.

THIS IS ONLY A PRELIMINARY DETERMINATION. CONSTRUCTION AND OPERATION CANNOT COMMENCE UNTIL THE APPLICANT HAS RECEIVED A PERMIT FROM THE DEPARTMENT.

NEW SOURCE REVIEW #85-SJK-003

Wisconsin Department of Natural Resources
Bureau of Air Management
P. O. Box 7921
Madison, Wisconsin 53707

April 26, 1985

INTRODUCTION

Sources which are not exempt from the construction or modification and new operation permit requirements as set forth in Section 144.391(4) or (5), Wis. Stats., and Section NR 154.04, Wis. Adm. Code are required to obtain an air pollution control permit prior to construction or modification and operation. Sources subject to the requirements must submit a permit application to the Department of Natural Resources. The application is then reviewed following the provisions set forth in Sections 144.392 to 144.394, Wis. Stats. The criteria for permit issuance vary depending on whether the source is major or minor and whether the source is locating in or significantly affecting an attainment or nonattainment area. The permit must be received by the applicant prior to start of construction and operation.

Subject sources are to be reviewed for the air pollution control technology proposed to be installed and for their impact upon the air quality. This is to insure compliance with all applicable rules and statutory requirements. The plan review will show why the source(s) should be approved, conditionally approved, or disapproved. It will encompass emission calculations and air quality analysis using U.S. EPA models, if applicable. Emissions from volatile organic compound (VOC) sources and small sources whose emissions are known to be insignificant are normally not modelled. As a precautionary note, the emission estimates are based on U.S. EPA emission factors (AP-42) or theoretical data and can vary from actual stack test data.

TYPE OF PERMIT

Permit to construct, operate and reclaim an underground zinc/copper/lead mine, ore processing mill and associated surface facilities.

Dates of Submittal: December 22, 1982; May 23, 1983; January 24, 1984; March 16, 1984; March 30, 1984; May 2, 1984; February 14, 1985.

Owner/Operator: Exxon Minerals Company, a Division of Exxon Corporation
P.O. Box 4508, Houston, Texas 77210

Contact: Barry J. Hansen, Permitting Manager
Exxon Minerals Company - Rhinelander
(715) 369-2800

Principal Executive Officer: Donald B. Achttien, General Manager
Exxon Minerals Company
P.O. Box 813, Rhinelander, WI 54501

Submitted By: Barry J. Hansen, Permitting Manager
Exxon Minerals Company
P.O. Box 813, Rhinelander, WI 54501

SOURCE DESCRIPTION

Exxon Minerals Company proposes to construct, operate and reclaim an underground zinc, copper and lead mine five miles south of Crandon, Wisconsin in Forest County. This project includes surface facilities for processing tne ore into zinc, copper and lead concentrates and disposal of the waste tailings from ore processing. The project does not include plans for an ore smelter.

The project timetable specifies a four year construction period, approximately 22 years of operation, and a 7 year final site reclamation phase. The ore processing mill has a design capacity of 10,535 short tons per day of ore, producing 519 tons per day of copper concentrate, 79 tons per day of lead concentrate, and 1,130 tons per day of zinc concentrate. The mine will operate 24 hours per day, 5 days per week. The mill will operate 24 hours per day, 7 days per week. A description of the air pollution sources associated with the construction, operation and reclamation phase follows.

Construction Phase

Mine Development:

A significant amount of fugitive dust will be generated during the development of the mine and associated surface facilities. Mine construction will require blasting with dynamite to develop the main access, air intake and mine exhaust shafts. Underground mine development will require blasting with ammonium nitrate and fuel oil. Mine construction will also require the use of diesel vehicles for rock handling and transporting personnel, equipment and material, natural gas-fired air heating equipment, and a temporary 2500 kilowatt diesel generator to provide electricity for the first six months of construction until power lines are installed.

Initial emissions from mine construction will be released at ground level until the east and west exhaust raises begin operation in the third and fourth year of construction. These raises will vent dust and other air pollutants generated within the mine. While the majority of the dust generated within the mine is expected to settle out due to gravitational forces before reaching the surface, all gaseous pollutants from fuel combustion and the detonation of explosives are assumed to be released to the ambient air.

The exhaust gas parameters for each of these raises are as follows:

Diameter = 22 feet (6.71 meters)
Height = 12.1 feet (3.7 meters)
Flow rate = 624,000 actual cubic feet per minute (294 cubic meters per second)

Surface Facilities Development:

Construction of the surface facilities will generate a significant amount of fugitive dust from excavation of the mill site, access roads to Highway 55, the potable water supply, east and west exhaust raises, explosive magazines and preproduction ore storage, the wastewater reclaim ponds (R1 and R2), the haul road to the mine waste disposal facility (MWDF), and the MWDF itself.

The MWDF is composed of four mine waste tailings ponds which will receive a tailings slurry by pipeline from the mill. Tailings pond T1, approximately 82 acres, will be constructed during the initial construction phase; the other three ponds will be excavated and developed as needed during the life of the mine. Construction of a tailings pond will include excavation activities and operation of a soil processing and pond liner batch plants. Water spraying and temporary revegetation will be used to suppress dust from excavation activities as needed. The soil plant will process approximately 139,380 short tons (126,445 mt) of material per year using fabric filters to control dust from handling operations. The liner plant will process approximately 102,820 cubic yards (78,004 m^3) per year of material and also will use fabric filters to control operations dust.

The land cleared for the surface facilities will generate a considerable amount of unmarketable brush and tree limbs (13,908 short tons) and stumps (6,419 short tons). Of this, a maximum of 2,545 short tons may be open burned using air curtain destructors if chipping is not feasible.

A concrete batch plant equipped with a fabric filter baghouse on the loading hopper and filters on the silo vents will operate during the construction phase. This will produce 101.3 short tons (91.9 metric tons) per day, and 26,345 short tons (23,900 metric tons) per year of concrete.

Operation Phase

Mine Operation:

During operation of the underground mine, air pollutant emissions will result from drilling and blasting rock to access the ore (drift development) and allow subsequent ore removal (stope production), ore handling and processing activities, diesel vehicle operation and mine air heating equipment.

Production blasting will use ammonium nitrate and fuel oil (ANFO) explosives. Blasting will occur at the end of each shift.

Maximum blasting rates will be:

179,740 short tons (163,059 mt) of ore and waste rock per day
3,991,900 short tons (3,621,428 mt) of ore and waste rock per year
87.9 short tons (79.7 mt) of ANFO per day
2,329.2 short tons (2,113 mt) of ANFO per year

Drilling is expected to be relatively dust free due to water injection to the drill bit. Most dust from blasting is expected to settle out due to gravitational forces before leaving the mine through the east and west exhaust raises. Gaseous pollutants from blasting are assumed to be exhausted to the ambient air.

After being liberated by blasting, the ore and waste rock will be transported to ore and waste rock passes by diesel vehicles, from here the material is fed to rail cars and transported to the primary crusher feed bin. The primary crusher reduces the ore and waste rock to a six inch or less diameter and

discharges to a picking conveyor belt. This transports the material to the loading belt and into the loading pocket. From the loading pocket, ore and waste rock is fed to hoisting skips and pulled to the surface for processing or disposal. Figure 1.2-8 taken from the project environmental impact report summarizes the ore and waste rock handling procedures within the mine.

Dust will be generated during all handling and crushing operations. Most of this dust will settle out due to gravitational forces before reaching the surface. However, to reduce dust in the mine, the primary crusher, discharge to the picking belt, loading belt and loading pocket will be hooded and ducted through a fabric filter dust collection system. The skip loading station will be equipped with water sprays to suppress dust during loading operations.

The capacity of the primary crusher will be approximately 1,323 short tons (1,200 mt) per hour. The maximum amount of ore and waste rock expected to be transported to the surface headframe of the main shaft is 1,213 short tons (1,100 mt) per hour and 24,000 short tons (21,800 mt) per day.

Diesel vehicles within the mine will be used for ore, waste rock, personnel and equipment transport. Most vehicles will use a catalytic scrubber and water-diesel fuel emulsion for fuel in order to minimize pollutant emissions within the mine. All emissions from diesel fuel usage are expected to eventually be exhausted to the ambient air through the exhaust raises. Maximum diesel fuel usage is expected to be 4,185 gallons (15,841 liters) per day, and 965,387 gallons (3,654,000 liters) per year.

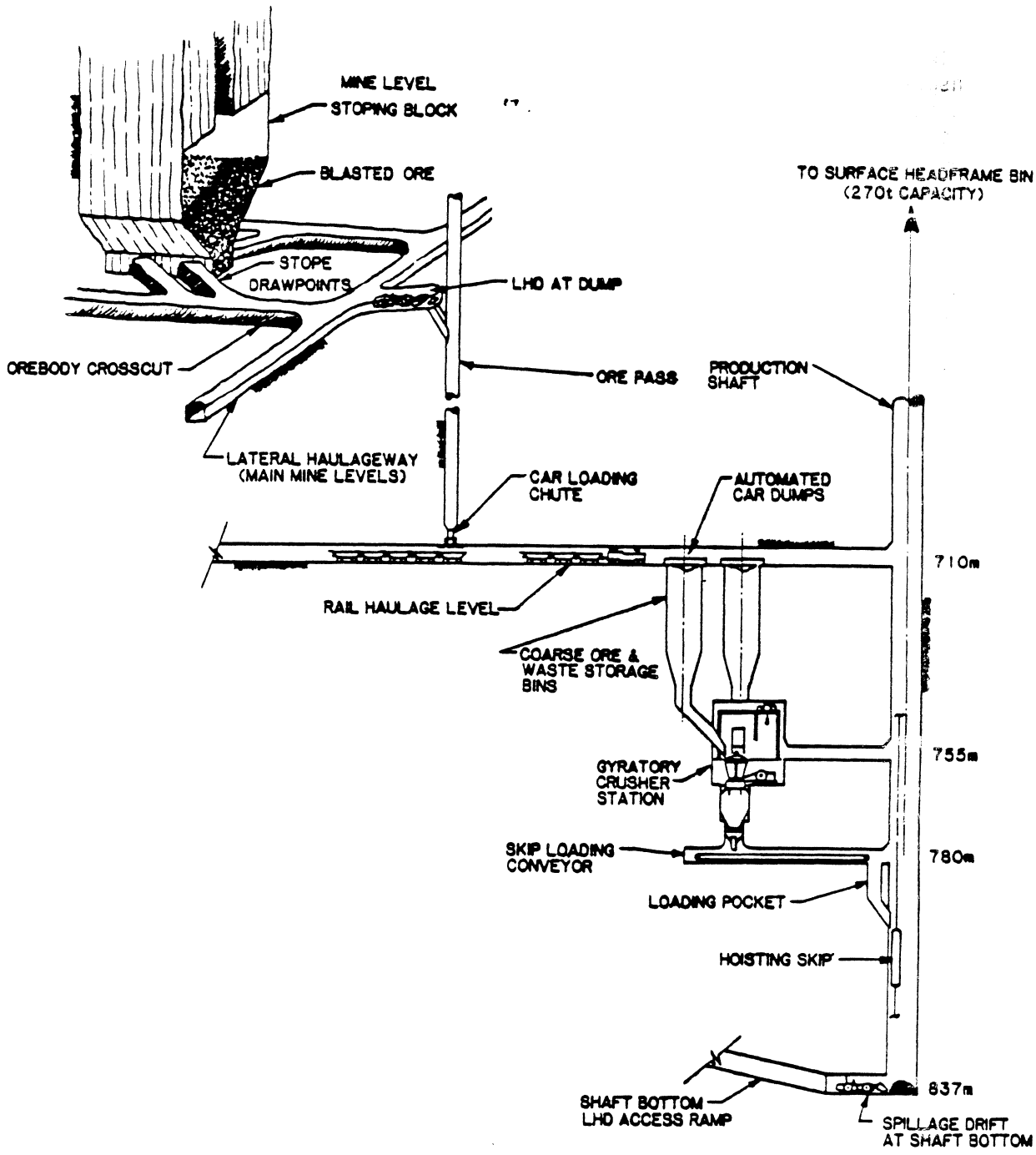
Natural gas-fired air heaters located on the surface will provide heated air when necessary to prevent freezing of water and service pipelines in the intake shafts. Since these directly heat air entering the mine, the combustion gases and pollutants will flow through the mine and are expected to vent through the exhaust raises. The capacity of these heaters is 91,800 cubic feet (2,600 m^3) of natural gas per hour, or 91.8 million BTU per hour @ 1000 BTU per ft^3. Yearly natural gas usage is expected to be 110.6 million cubic feet (3.13 million m^3).

Operation of the underground mine activities will be 24 hours per day, five days per week.

Surface Facilities Operation:

Air pollutant emissions during surface facilities operation will result from ore handling and crushing, fugitive dust and tailpipe emissions from vehicles, fuel transfer and storage, and surface facilities heating. Electrical power to operate the mine and ore processing mill will come from a nearby utility, so an on-site generating facility has not been proposed. Stand-by diesel generators will, however, be used.

Ore from the surface headframe of the main shaft will be transported to the coarse ore storage building by enclosed conveyor. Ore will be stockpiled in the building in massive ore piles (ore containing zinc, copper and lead) and stringer ore piles (ore containing primarily copper and zinc).



Typical Representation: Refinements May Be Made During Final Engineering

EXXON MINERALS COMPANY
CRANDON PROJECT

TITLE
ORE TRANSPORT SYSTEM

SCALE	STATE	COUNTY
NONE	WISCONSIN	FOREST

DRAWN BY	DATE	APPROVED BY	DATE
CHECKED BY	DATE	APPROVED BY	DATE
CHECKED BY	DATE	ENGR.	DATE

DRAWING NO. FIGURE 1.2-8 SHEET OF REVISION NO.

The two piles will feed to a tunnel and conveyor galleries located below, then transported to the fine ore crushing building. Fabric filters will be used when necessary to minimize dust from the ore storage and conveying operations. These filters will vent into the storage building, and not outdoors. The fine crushing building contains secondary and tertiary crushers and screens which reduce the 6 inch diameter ore to 0.75 inches or less. These crushers and screens are hooded and dust is ducted to a wet scrubber which exhausts to the ambient air. The dust/water slurry generated by this scrubber is directed to and recycled in the ore floatation process. The capacity of the fine crushing operation is 683 short tons (620 mt) per hour. The stack parameters for the crushing/screening scrubber is as follows:

Stack height = 85.3 feet (26.0 m)
Diameter = 3.3 feet (1.0 m)
Gas flow rate = 33,617 acfm (15.8 m^3/sec)
Temperature = Ambient

The crushers will feed the ore by conveyor to massive and stringer fine ore bins. Dust generated during the loading and unloading of these bins is also ducted to a wet scrubber before exhausting to the ambient air. The dust/water slurry generated by this scrubber is also recycled in the ore flotation process. The stack parameters for this scrubber are as follows:

Stack height = 124.7 feet (38.0 m)
Diameter = 2.8 feet (0.84 m)
Gas flow rate = 24,042 acfm (11.3 m^3/sec)
Temperature = Ambient

The fine ore bins feed by conveyor to the rest of the ore processing operations which are wet processes and are not expected to produce dust or emissions of concern. These include grinding, flotation, thickening and dewatering, concentrate storage and concentrate loadout. The zinc, copper and lead concentrates (zinc sulfide, copper sulfide and lead sulfide) will contain approximately 8 to 10% moisture when loaded into railroad gondola cars or trucks by telescopic spout. Therefore, no appreciable dust is expected. The gondola cars or trucks will be covered with a plastic cover to keep concentrates from becoming airborne during transport.

A concrete batch plant for miscellaneous uses in the mine and mill will operate during the life of the mine. It will provide a maximum of 50 cubic yards (38.3 m^3) or 101.1 short tons (91.9 mt) of concrete per day, and 26,345 short tons (23,900 mt) per year. Fabric filters will be used to minimize dust emissions to the air from sand, aggregate and cement handling operations. Stack parameters for this batch plant are expected to be as follows:

Stack height = 26.2 feet (8.0 m)
Diameter = 1.4 feet (0.43 m)
Gas flow rate = 3,043 acfm (1.43 m^3/sec)
Temperature = Ambient

Throughout the surface facility buildings, natural gas-fired heaters will be used for space heating, water heating and water treatment (brine crystallizer). Maximum total natural gas usage is proposed to be 33,900 cubic feet (950 m^3) per hour, or 33.9 million BTU per hour @ 1000 BTU per cubic foot, and 171 million cubic feet (4.8 million m^3) per year.

Fuel storage for the surface facilities will consist of two 15,000 gallon fixed roof diesel fuel storage tanks, and a vehicle filling station with a 3,000 gallon diesel tank and 1,000 gallon gasoline tank. Diesel throughput will be a maximum of 6,000 (22,710 liters) gallons per day, and gasoline throughput is 500 gallons (1892.5 liters) per day.

Three emergency generators will be kept on hand, two 2,500 kilowatt and one 1,000 kilowatt units. To verify reliability each unit will be tested weekly and operated for one hour. Emergency operation is estimated to be 2.5 days per year. Maximum yearly diesel fuel usage is estimated to be 163,365 gallons (618,390 liters).

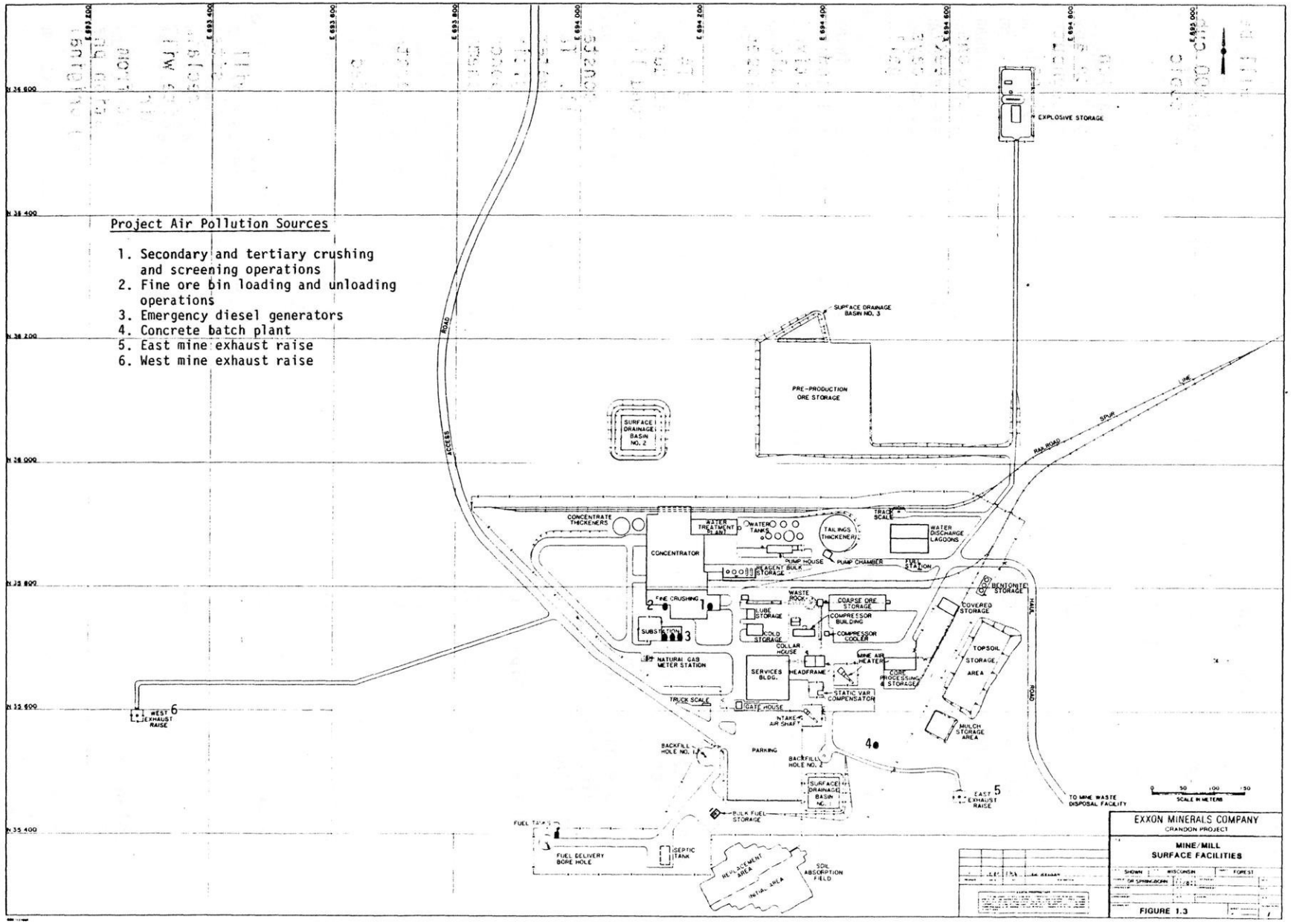
Surface vehicle travel on unpaved roads such as the site access roads and the haul road to the mine waste disposal facility (MWDF) will generate fugitive dust. Water spraying and application of chemical stabilizers such as calcium chloride will be used as necessary to minimize this dust. A truck equipped for this purpose will be kept on-site.

During operation of the surface facilities, tailings ponds T2, T3, and T4 will be constructed as needed. As with T1, fugitive dust will be generated during excavation of these ponds and operation of the soil processing and pond liner batch plants. While in use, the MWDF is not expected to contribute significant dust emissions. The active tailings pond will receive a constant flow of waste tailings slurry by pipeline from the ore processing mill. This will maintain a wet pond surface. Any areas which become dry will be water sprayed as necessary to keep tailings from becoming airborne. Some fugitive dust will be generated by truck activity around the disposal facility ponds. As with other unpaved surface roads, water spraying or longer lasting chemical stabilizers will be used as necessary to suppress vehicle dust.

Ore processing activities at the surface facilities are expected to operate 24 hours per day, 7 days per week. A plot plan of the surface facilities (Figure 1.3) taken from the Revised Mining Permit Application is provided on the following page.

Reclamation Phase

Throughout the life of the mine, reclamation of filled tailings ponds will occur. However, after the useful life of the mine is completed, final site reclamation will begin. Surface facilities will be removed, and the reclaim ponds and remaining tailings ponds will be covered. The mine/mill site will be returned to its original condition as much as it is practical. Air pollution emissions during this phase will primarily be fugitive dust from excavation activities and tailpipe emissions from vehicles. Reclamation phase activities will contribute significantly less emissions than during original site construction or mine/mill operation.


Project Air Pollution Sources
1. Secondary and tertiary crushing and screening operations
2. Fine ore bin loading and unloading operations
3. Emergency diesel generators
4. Concrete batch plant
5. East mine exhaust raise
6. West mine exhaust raise
EXPLOSIVE STORAGE
SURFACE DRAINAGE BASIN NO. 3
PRE-PRODUCTION ORE STORAGE
SURFACE DRAINAGE BASIN NO. 2
ACCESS ROAD
RAIL ROAD SPUR LINE
CONCENTRATE THICKENERS
WATER TREATMENT PLANT
WATER TANKS
TAILINGS THICKENER
TRUCK SCALE
WATER DISCHARGE LAGOONS
CONCENTRATOR
PUMP HOUSE
PUMP CHAMBER
FUEL STATION
REAGENT BULK STORAGE
WASTE ROCK
BENTONITE STORAGE
FINE CRUSHING
COARSE ORE STORAGE
COVERED STORAGE
LUBE STORAGE
COMPRESSOR BUILDING
SUBSTATION
COMPRESSOR COOLER
COLD STORAGE
COLLAR HOUSE
TOPSOIL STORAGE AREA
MINE AIR HEATER
NATURAL GAS METER STATION
SERVICES BLDG.
HEADFRAME
CORE PROCESSING & STORAGE
STATIC VAR COMPENSATOR
TRUCK SCALE
GATE HOUSE
INTAKE AIR SHAFT
MULCH STORAGE AREA
WEST EXHAUST RAISE
BACKFILL HOLE NO. 1
PARKING
BACKFILL HOLE NO. 2
SURFACE DRAINAGE BASIN NO.
EAST EXHAUST RAISE
TO MINE WASTE DISPOSAL FACILITY
BULK FUEL STORAGE
FUEL TANKS
FUEL DELIVERY BORE HOLE
SEPTIC TANK
REPLACEMENT AREA
INITIAL AREA
SOIL ABSORPTION FIELD
SCALE IN METERS
EXXON MINERALS COMPANY
CRANDON PROJECT
MINE/MILL SURFACE FACILITIES
FIGURE 1.3

EXISTING FACILITY EMISSIONS

This is a new facility so there are no existing air pollution emissions.

EMISSIONS FROM NEW EQUIPMENT

Source	Pollutant	Predicted Maximum lbs/hr	Predicted Maximum TPY	Allowable lbs/hr	Allowable TPY
1. Construction phase fugitive dust, open burning, fuel usage and blasting	TSP	-	209.6	Same as maximum	
	SO_2	-	18.8	Same as maximum	
	NO_x	-	79.8	Same as maximum	
	CO	-	186.7	Same as maximum	
	HC	-	74.2	Same as maximum	
	H_2S	-	0.5	Same as maximum	
2. Underground Mine Operations	TSP	93.9	27.4	53.91	27.4
	SO_2	166.8	30.1	Same as maximum	
	NO_x	1344.6	77.6	Same as maximum	
	CO	5669.6	81.3	Same as maximum	
	HC	1.23	1.4	Same as maximum	
	Pb	-	0.11	Same as maximum	
3. Fine ore crushing and screening operations	TSP	3.0	8.8	6.1	26.9
4. Fine ore bin loading and unloading operations	TSP	2.0	5.9	4.4	19.2
5. Concrete batch plant	TSP	0.2	0.2	4.1	17.8
6. Facility heating	TSP	0.3	0.9	Same as maximum	
	SO_2	0.02	0.05	Same as maximum	
	NO_x	4.0	10.3	Same as maximum	
	CO	0.6	1.5	Same as maximum	
	HC	0.1	0.3	Same as maximum	
7. Fuel transfer and storage	HC	NA	1.6	Same as maximum	
8. Emergency diesel generators	TSP	23.0	4.1	10.0	1.8
	SO_2	27.7	4.9	Same as maximum	
	NO_x	230.4	40.8	Same as maximum	
	CO	59.9	10.6	Same as maximum	
	HC	6.5	1.1	Same as maximum	
9. In-plant roads fugitive dust	TSP	-	49.1	Same as maximum	
10. Waste rock handling fugitive dust	TSP	-	9.3	Same as maximum	

Source	Pollutant	Predicted Maximum lbs/hr	TPY	Allowable lbs/hr	TPY
11. Tailings pond construction fugitive dust	TSP	-	112.8	Same as maximum	
12. Access road fugitive dust	TSP	-	3.2	Same as maximum	
13. Reclamation phase fugitive dust	TSP	-	97.0	Same as maximum	

MAJOR APPLICABILITY

Allowable emissions from operation of the air pollution sources at the proposed underground mine/mill are less than 250 tons per year for any pollutant. Therefore, this project is not subject to the requirements for major air pollutant sources under Section 144.393(3), Wisconsin Statutes, or the federal prevention of significant deterioration regulations (40 CFR 52.21).

SITE DESCRIPTION

The proposed underground mine and surface facilities will be located five miles (8 km) south of Crandon, Wisconsin in Forest County. See Figure 3-2 which is taken from the revised project air permit application. Forest County is considered attainment/unclassified for all criteria air pollutants. Ambient air quality monitoring by Exxon in 1977 and 1978 confirmed this area to be in attainment status for the particulates and sulfur dioxide air quality standards.

CONTROL TECHNOLOGY REVIEW

Mine Operation:

Dust and gaseous pollutants created within the underground mine will eventually be vented to the ambient air through the east and west exhaust raises. Several steps will be taken to minimize some of these emissions. Diesel vehicles which will transport personnel, ore and equipment within the mine will burn a water/fuel emulsion which reduces nitrogen oxide and particulate tail pipe emissions. A catalytic scrubber will then be used to reduce carbon monoxide and hydrocarbon emissions which increase due to the use of this water/diesel fuel emulsion.

The primary ore crusher and subsequent ore handling operations will be hooded and equipped with fabric filter modules to collect dust. Though not finalized at this time, these modules will be similar to the Dalamatic Insertable Filers manufactured by DCE Vokes, Inc. These are described in the attachment for Comment No. 1 of the Exxon Minerals air permit submittal dated March 16, 1984. The filters chosen will be felted nylon, orlon or dacron type, have an air to cloth ratio not exceeding 10:1 and be frequently cleaned

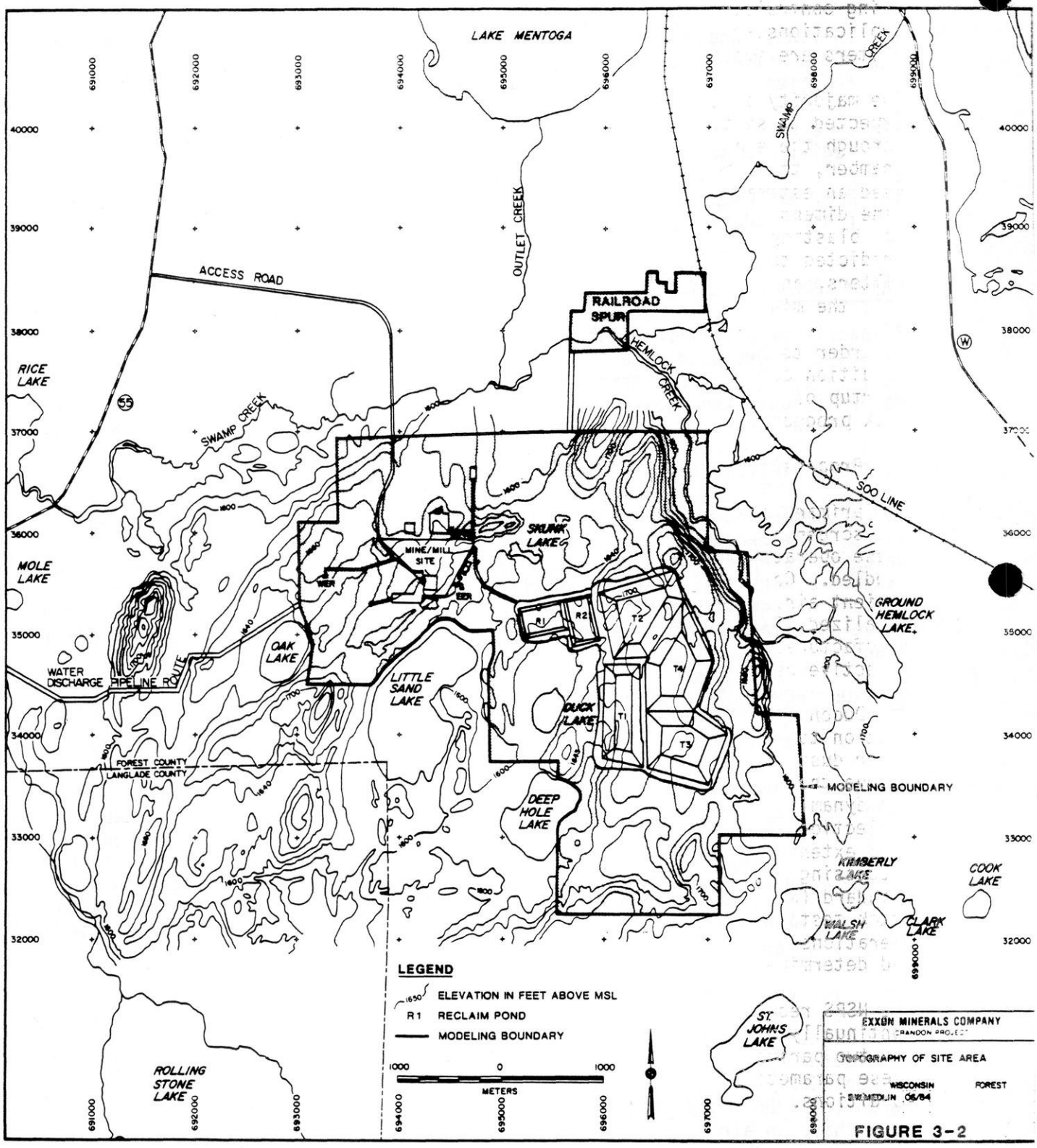


FIGURE 3-2

384-1-136

WER:West Exhaust Raise
EER:East Exhaust Raise
T1:Tailings Pond T1
R1:Reclaim Pond R1

with a pulse of compressed air. These filters would exhaust back into the mine and return collected dust directly to the crusher or ore transfer point being controlled. The DCE Vokes, Inc. filters have been used in similar applications and are expected to be successful if they or similar type fabric filters are employed.

The majority of the dust generated by production blasting and rock handling is expected to settle out due to gravitational forces before exiting the mine through the exhaust raises. In this way, the mine acts as a large settling chamber, the simplest of particulate control devices. Emission calculations used an estimated particle size distribution, particle density, flow rate and mine dimensions to estimate the portion of the dust which would settle out. For blasting and non-filtered rock handling operations, 75-95% of the dust was predicted to settle. All dust from handling operations equipped with fabric filters, and fuel combustion, as well as all gaseous pollutants are assumed to exit the mine through the exhaust raises.

In order to verify the emission estimates, it will be recommended as a condition of permit approval that the exhaust raises be stack tested upon startup of the mine for particulates and records be kept of mine ore and waste rock production and explosive usage.

Ore Processing Mill Operations:

The primary sources of emissions during ore processing are fine ore crushing and screening, and fine ore bin loading and unloading operations. Both of these operations will be hooded at the locations where ore is processed and handled. Collected dust will be vented to wet scrubbers which exhaust to the ambient air. Though the actual collectors which will be used have yet to be finalized, Exxon has proposed to use Ducon Type UW-4, Model IV wet scrubbers manufactured by the Ducon Company, Inc. or collectors of similar control effectiveness.

The Ducon Type UW-4 wet scrubber consists of three stages: a precleaner section to remove larger dust particles, a dynamic fan to turbulently mix the finer dust particles with the scrubbing water, and a cyclonic separator to remove the scrubbing liquid and entrained dust from the exiting stack gases. The dynamic fan serves to both pull the exhaust air through the dust collection system and atomize the scrubbing liquid. This type of scrubber is used extensively in the mining industry as well as for other mineral processing operations. Compliance with the federal new source performance standard (NSPS) for metallic mineral processing (0.05 gm/dscm) is expected. Stack tests upon start-up of the fine ore crushing and bin loading/unloading operations will be recommended to verify compliance with NSPS emission limits and determine necessary operating conditions to assure continued compliance.

The NSPS requires that any wet scrubbers used for control purposes be continually monitored for pressure drop and scrubbing liquid flow rate. These are two parameters affecting scrubber performance. Exxon has indicated that these parameters will be monitored. These will also be recommended as permit conditions.

The pilot plant used for preproduction ore processing will be hooded at dust generating points on the plant and ducted through a fabric filter baghouse. This baghouse is expected to be a DCE Vokes Model DLM 1/3/10 Type W or equivalent. This DCE model is similar to those used in the mine except for a smaller filter area - 315 ft^2. Fabric filters were found by the U.S. EPA to be able to comply with the NSPS for mineral processing plants. No stack test requirement will be recommended since this pilot plant will operate only for short period (6 to 12 months) and process a small amount of ore (2 short tons per hour).

Several dust sources at the mill will use fabric filters to reduce dust in the work areas. The coarse ore transport conveyor to the headframe, coarse ore storage building, surge bins for the fine ore crushing, fine ore crushing transfer tower, lime storage bin loading, sodium carbonate storage bin loading, cement bin loading operations and backfills sand handling will use fabric filters such as the DCE Vokes type used within the mine. These filters, however, will not vent to the ambient air but back into the buildings which house these operations. Minimal emissions to the ambient air are expected from these operations.

The concrete batch plant used for miscellaneous mine/mill site applications will be equipped with a fabric filter baghouse and miscellaneous filters to control dust from sand, aggregate and cement handling operations. This again would be the DCE Vokes type or similar. Generally fabric filters should be more than adequate to control dust to levels allowed by Sec. NR 154.11(3), Wis. Adm. Code (0.2 pounds per 1000 pounds of exhaust gas).

Fugitive dust may contribute significant particulate during facilities operation. Access road and in-plant road usage, tailings pond construction activities, preproduction ore and waste rock handling operations, and reclamation activities are the primary fugitive dust sources that have been identified. Exxon proposes to use regular water spraying (approximately once a shift) or longer lasting chemical dust suppressants such as calcium chloride during extensive dry periods to control fugitive dust. A truck equipped with spray equipment will be available on-site. Trucks carrying crushed rock or fine sized material will be covered or water sprayed as required if long distance transport is necessary. Areas subject to cut and fill operations will be temporarily revegetated after final grading for soil stabilization and wind blown dust control. Frequently traveled in-plant roads and the access road to Highway 55 will be paved early in the construction phase eliminating probable sources of fugitive dust. It will be recommended that a specific fugitive dust control plan be submitted prior to facility startup.

Using generic emission factors from U.S. EPA publication AP-42, it was shown that diesel generators used for emergency power generation would not comply with the emission limit of 0.15 #TSP for MMBTU in Sec. NR 154.11(4)(a)1., Wis. Adm. Code. (See p. 30 of the Appendix.) Particulate emissions due vary depending on the engine used. It will be recommended as a permit condition that prior to the expiration of the project shakedown period, Exxon verify the specific diesel generators to be used and provide sufficient emissions data to prove compliance.

During the construction phase, significant quantities of waste trees and brush will be generated as the site is cleared. Of this, approximately 20,327 short tons of unmarketable brush, limbs, and stumps will need to be disposed of. (B. Hansen - Exxon correspondence to G. Kulibert - DNR, November 9, 1984.) While most will be chipped and likely sold as fuel or fill, approximately 2,545 short tons have been identified in the air permit application for likely open burning. Air curtain destructors have been proposed to improve waste combustion and reduce emissions. Nevertheless, the air quality analysis has shown significant air quality impacts would result if large amounts of material is burned. Therefore, a constraint on open burning will be recommended as a condition of permit approval. Air curtain destructors must be used to burn waste wood and brush, and no more than 500 pounds per hour of waste may be burned.

The Mine Waste Disposal Facility (MWDF) tailings ponds have been suggested as significant sources of fugitive dust. The MWDF Feasibility Report of December, 1982 (p. 3-14) indicates that better than 85% of the tailings will be 30 um or less. As this is the particle size range where wind blown dust remains airborne for extended distances, the potential for significant fugitive dust exists if the tailings are allowed to dry and be exposed to high winds. However, the MWDF tailings ponds are not expected to be significant dust sources. The tailings will be delivered to the active pond by pipeline in the form of slurry. As the tailings settle in the ponds, a pool of water is expected to form at the end away from the discharge pipes. As the ore processing mill will operate 7 days per week, the tailings flow will be continuous. This will maintain a wet pond surface. Some areas not contacted by the incoming tailings slurry, could potentially become dry enough where strong winds would cause tailings to become airborne. The initial depth of the tailings ponds range from 77 to 101 feet. Therefore, the pond embankments may serve to reduce wind speeds in the ponds and contain airborne tailings. As a precaution against fugitive dust, however, Exxon will water spray tailings which have dried as found necessary. Should an extended mine/mill shutdown occur such that the tailings slurry flow is stopped, Exxon will take additional measures such as chemical stabilizer application to avoid airborne tailings. Specific details of the tailings ponds dust control program will be provided in the MWDF Plan of Operation required under Sec. NR 182.09, Wis. Adm. Code. Development of this plan will also be recommended as a condition of air permit approval.

A malfunction prevention and abatement plan for control equipment and methods as required by Sec. NR 154.06(a), Wis. Adm. Code, was not provided with the air permit application submittals. The submittal of this plan will be recommended as a condition of air permit approval to insure continued maintenance of air pollution control equipment.

APPLICABLE LIMITATION

1. Construction of the mine and surface facilities

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(2)	See Note 1
	Sec. 144.394(6), Wis. Stats.	209.6 tons per year*
Sulfur Dioxide	Sec. 144.394(6), Wis. Stats.	18.8 tons per year*
Nitrogen Oxides	Sec. 144.394(6), Wis. Stats.	79.8 tons per year*
Carbon Monoxide	Sec. 144.394(6), Wis. Stats.	186.7 tons per year*
Hydrocarbons	Sec. 144.394(6), Wis. Stats.	74.2 tons per year*
Hydrogen Sulfide	Sec. 144.394(6), Wis. Stats.	0.5 tons per year*

Note 1: Fugitive Dust: No person shall cause, allow, or permit any materials to be handled, transported, or stored without taking precautions to prevent particulate matter from becoming airborne. Nor shall a person allow a structure, a parking lot, or a road to be used, constructed, altered, repaired, sand blasted or demolished without taking such precautions.

(a) Such precautions shall include, but not be limited to:

1. Use, where possible, of water or chemicals for control of dust in the demolition of existing buildings or structures, or construction operations.

2. Application of asphalt, oil, water, suitable chemicals, or plastic covering on dirt roads, material stockpiles, and other surfaces which can create airborne dust, provided such application does not create a hydrocarbon, odor, or water pollution problem.

3. Installation and use of hoods, fans, and air cleaning devices to enclose and vent the areas where dusty materials are handled.

4. Covering or securing of materials likely to become airborne while being moved on public roads, railroads, or navigable waters.

5. Conduct of agricultural practices such as tilling of land or application of fertilizers in such manner as not to create pollution.

6. The paving or maintenance of roadways or parking lots so as not to create air pollution.

*This emission limitation is set in order to document the allocation of the available air resources. It represents the maximum emissions expected during the project construction phase.

2. Underground mine operations venting through the east and west exhaust raises.

For purposes of determining an applicable emission limitation, the underground mine operations are treated as a process. The applicable limitation for a process is either Sec. NR 154.11(3)(a)1.a. based on the process weight rate, or Sec. NR 154.11(3)(b)1.m., Wis. Adm. Code based on the exhaust gas flow rate, whichever is more restrictive. In this case, the former of the two is more restrictive.

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(3)(a)1.a.	$E = 17.31\ P^{0.16}$ and
	Sec. 144.394(6), Wis. Stats.	27.4 tons per year*
Sulfur Dioxide	Sec. 144.394(6), Wis. Stats.	8.0 pounds per hour, 17.7 tons per year**
Nitrogen Oxides	Sec. 144.394(6), Wis. Stats.	40.9 pounds per hour, 79.2 tons per year**
Carbon Monoxide	Sec. 144.394(6), Wis. Stats.	59.8 pounds per hour, 81.1 tons per year**

*E represents the allowable emission rate in pounds per hour, and P, represents the total weight of materials introduced to the process, excluding liquid and gaseous fuels and air, in tons per hour. In this case, for a maximum process weight rate of 1,213 short tons per hour, (the weight of material brought to the surface by the hoisting skip), the allowable TSP emission rate is 53.9 pounds per hour. A yearly emission limit of 27.4 tons per year is set in order to document the allocation of available air resources. This is the maximum particulate emissions expected from mine operation.

**These emission limits are set in order to document the allocation of available air resources. These represent the maximum emissions expected from mine operation.

3. Fine ore crushing and screening operations

As this is a process, the applicable limitation is either Sec. NR 154.11(3)(a)1.a, or Sec. NR 154.11(3)(b)1.m., Wis. Adm. Code. In this case, Sec. NR 154.11(3)(b)1.m. - 0.2 pounds per 1000 pounds of exhaust gas - would be more restrictive.

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(3)(b)1.m.	0.05 grams per dry
	Sec. 144.394(3), Wis. Stats.	standard cubic meter*
Visible Emissions	Sec. NR 154.11(6)(a)1.	20% opacity

*This process would normally be subject to the emission limit of Sec. NR 154.11(3)(b)1.m., Wis. Adm. Code of 0.2 pounds per 1000 pounds of exhaust gas (0.24 grams per dry standard cubic meter). However, this process is also subject to the federal New Source Performance Standards (NSPS) for metallic mineral processing plants of 40 CFR Part 60 Subpart LL. In anticipation that these standards will be adopted into Chapter 440, Wis. Adm. Code, the more restrictive NSPS of 0.05 grams per dry standard cubic meter is applied.

4. Fine ore bin loading and unloading operations

As this is a process, the applicable limitation is either Sec. NR 154.11(3)(a)1.a., or Sec. NR 154.11(3)(b)1.m., Wis. Adm. Code. In this case, Sec. NR 154.11(3)(b)1.m. - 0.2 pounds per 1000 pounds of exhaust gas - would be more restrictive.

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(3)(b)1.m. Sec. 144.394(3), Wis. Stats.	0.05 grams per dry standard cubic meter*
Visible Emissions	Sec. NR 154.11(6)(a)1.	20% opacity

*This process would normally be subject to the emission limit of Sec. NR 154.11(3)(b)1.m., Wis. Adm. Code of 0.2 pounds per 1000 pounds of exhaust gas (0.24 grams per dry standard cubic meter). However, this process is also subject to the federal New Source Performance Standards (NSPS) for metallic mineral processing plants of 40 CFR Part 60 Subpart LL. In anticipation that these standards will be adopted into Chapter 440, Wis. Adm. Code, the more restrictive NSPS of 0.05 grams per dry standard cubic meter is applied.

5. Concrete batch plant

As this is a process, the applicable limitation is either Sec. NR 154.11(3)(a)1.a., or Sec. NR 154.11(3)(b)1.m., Wis. Adm. Code. In this case, Sec. NR 154.11(3)(b)1.m. - 0.2 pounds per 1,000 pounds of exhaust gas - would be more restrictive.

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(3)(b)1.m.	0.2 pounds per 1,000 pounds of exhaust gas.
Visible Emissions	Sec. NR 154.11(6)(a)1.	20% opacity

6. Facility heating

This includes all fuel usage for surface facility space heating, water heating and water treatment.

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(4)(a)1. Sec. 144.394(3), Wis. Stats. Sec. 144.394(6), Wis. Stats.	0.01 pounds per million BTU heat input* 0.9 tons per year**
Sulfur Dioxide	Sec. 144.394(6), Wis. Stats.	0.10 tons per year**
Nitrogen Oxides	Sec. 144.294(6), Wis. Stats.	10.3 tons per year**
Carbon Monoxide	Sec. 144.394(6), Wis. Stats.	1.5 tons per year**
Hydrocarbons	Sec. 144.394(6), Wis. Stats.	0.3 tons per year**

* This alternate limitation represents the maximum emissions expected. Sec. NR 154.11(4)(a)1., Wis. Adm. Code, allows 0.15 pounds per million BTU heat input.

**These emission limitations are set in order to document the allocation of the available air resource. Yearly emissions are based on a maximum total natural gas usage of 171,032,000 standard cubic feet per year.

7. Diesel and Gasoline Storage

This includes diesel and gasoline storage and handling emissions from the two 15,000 gallon diesel fuel oil storage tanks, the 3,000 gallon diesel and gasoline storage tanks at the facility service station, and the mine diesel storage tanks.

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Organic Compounds	Sec. NR 154.13(3)(f)2.	See Note 1
	Sec. 144.394(6), Wis. Stats.	1.6 tons per year*

Note 1: The gasoline storage tank shall be equipped with a permanent submerged fill pipe with a discharge opening which is entirely submerged when the liquid level is 6 inches above the tank bottom.

* This limitation is set in order to document the allocation of the available air resource. Yearly emissions are based on a maximum diesel fuel oil usage of 2.1 million gallons per year, and gasoline usage of 175,000 gallons per year.

8. Emergency diesel generators

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(4)(a)1.	0.15 lbs/MMBTU heat input
	Sec. 144.394(6), Wis. Stats.	1.8 tons per year*
Sulfur Dioxide	Sec. 144.394(6), Wis. Stats.	27.7 pounds per hour,
	Sec. 144.394(6), Wis. Stats.	4.9 tons per year*
Nitrogen Oxides	Sec. 144.394(6), Wis. Stats.	230.4 pounds per hour, 40.84tons per year*
Carbon Monoxide	Sec. 144.394(6), Wis. Stats.	59.9 pounds per hour, 10.6 tons per year*
Hydrocarbons	Sec. 144.394(6), Wis. Stats.	6.5 pounds per hour, 1.1 tons per year*
Visible Emissions	Sec. NR 154.11(6)(a)1.	20% opacity

*These emission limits are set in order to document the allocation of the available air resource. Yearly emissions are based on a maximum diesel fuel usage of 163,365 gallons per year for testing and emergency operation.

9. Surface facilities operation fugitive dust

Pollutant	Applicable Wis. Adm. Code	Limitation/Requirement
Particulates	Sec. NR 154.11(2)	See Note 1
	Sec. 144.394(6), Wis. Stats.	174.4 tons per year*

Note 1: Fugitive Dust. No person shall cause, allow, or permit any materials to be handled, transported, or stored without taking precautions to prevent particulate matter from becoming airborne. Nor shall a person allow a structure, a parking lot, or a road to be used, constructed, altered, repaired, sand blasted or demolished without taking such precautions.

(a) Such precautions shall include, but not be limited to:

1. Use, where possible, of water or chemicals for control of dust in the demolition of existing buildings or structures, or construction operations.

2. Application of asphalt, oil, water, suitable chemicals, or plastic covering on dirt roads, material stockpiles, and other surfaces which can create airborne dust, provided such application does not create a hydrocarbon, odor, or water pollution problem.

3. Installation and use of hoods, fans, and air cleaning devices to enclose and vent the areas where dusty materials are handled.

4. Covering or securing of materials likely to become airborne while being moved on public roads, railroads, or navigable waters.

5. Conduct of agricultural practices such as tilling of land or application of fertilizers in such manner as not to create pollution.

6. The paving or maintenance of roadways or parking lots so as not to create air pollution.

*This emission limitation is set in order to document the allocation of the available air resources. It represents the maximum emissions expected from access road and in-plant road usage, tailings pond construction activities and waste rock handling operations, or reclamation activities.

AIR QUALITY REVIEW

To evaluate the effects of this project on the air quality in the surrounding area, two analyses were done using U.S. EPA approved dispersion models. The first was that completed by Exxon Minerals and submitted with their revised air permit application of February 1985 (Section 4 - Air Quality Assessment). The second was that done by the Air Impact Analysis and Planning Section of the DNR Bureau of Air Management. (See the April 4, 1985 DNR Memorandum from Larry Bruss to Steve Klafka, Exxon Crandon Mine - Air Quality Analysis.)

The Exxon Minerals analysis evaluated the project impacts during the fourth year of mine/mill operation. This is the year when annual particulate emissions would be greatest. (See emission summary Table 4.1 of the permit application.) The results of this analysis are presented in Table 4.7 which follows are taken from the revised air permit application. This table summarizes the predicted maximum pollutant concentrations, representative background concentrations, and the total pollutant concentrations as they relate to the national ambient air quality standards (NAAQS). The pollutant of greatest consequence (consuming the largest percentage of the applicable standard) is total suspended particulates. The maximum 24 hour average predicted impact or concentration is 36.5 ug/m^3. Combined with a background of 77 ug/m^3, the predicted worst case total is 113.5 ug/m^3. This is below the secondary particulate standard of 150 ug/m^3, and well below the primary (health related) standard of 260 ug/m^3. The predicted annual average impact was 3.8 ug/m^3. Combined with a background of 17.9 ug/m^3, the total is

TABLE 4.7

COMPARISON OF STATE AND FEDERAL AMBIENT AIR QUALITY STANDARDS[a] WITH ISC MODEL PREDICTED CONCENTRATIONS FOR THE PROJECT AT THE MODELLING BOUNDARY
(All Concentrations in ug/m3)

	PREDICTED CONCENTRATION FROM CRANDON PROJECT	BACKGROUND CONCEN-TRATION	SUMMED CONCEN-TRATION	PRI-MARY NAAQS	SECON-DARY NAAQS
SULFUR DIOXIDE					
Annual	0.1b(2.1)c	25.0	25.1 (27.1)	80	--
24-Hour	1.8 (24.9)	25.0	26.8 (49.9)	365	--
3-Hour	7.1 (186.0)	25.0	32.1 (211.0)	--	1,300
PARTICULATE MATTER (TSP)					
Annual	3.8	17.9	21.7	75	60
24-Hour	36.5	77.0	113.5	260	150
NITROGEN DIOXIDE					
Annual	3.8	19.4	23.2	100	100
CARBON MONOXIDE					
8-Hour	1802.5	N/Ad	1802.5	10,000	10,000
1-Hour	2025.3	N/A	2025.3	40,000	40,000
HYDROCARBONS (HC)					
3-Hour	31.8	N/A	31.8	160	160
LEAD (Pb)					
3-Month Average	0.04	N/A	0.04	1.5	1.5

a. All short-term limits (24-hour and less) can be exceeded once a year.

b. Stationary sources only (see the December 1982 Air Permit Application, Appendix C - Tables C.10, C.11 and C.12 [Exxon Minerals Company, 1982]).

c. Includes temporary mobile source emissions and are the highest, second-highest predicted concentrations (see the December 1982 Air Permit Application, Appendix C - Table C-9 [Exxon Minerals Company, 1982]).

d. N/A = Not applicable.

21.7 ug/m^3. This, again, is well below the secondary and primary standards of 60 and 75 ug/m^3, respectively. Background particulate concentrations were obtained from an ambient monitoring program conducted by Exxon in 1977 and 1978.

All other criteria pollutants - sulfur dioxide, nitrogen oxides, carbon monoxide and lead - had predicted impacts which when combined with background concentrations were well below their applicable standards. The hydrocarbon standard no longer exists so this part of the Exxon analysis is not addressed.

Emission rates used for the air pollution sources modeled were: state and federal allowable emission limits for short-term particulate impacts, predicted actual emissions for long term particulate impacts, predicted actual emissions for both short and long term SO_2, NO_x, CO and Pb impacts. Thus, short term particulate impacts may be lower than predicted since actual short term emissions may be lower than allowable emission rates. (See April 30, 1984, April 16, 1984 and March 30, 1984 air permit correspondence for the derivation of modeling emission rates.)

An air quality impact analysis was conducted by the Air Impact Analysis and Planning Section of the DNR Bureau of Air Management. This analysis was conducted to both verify and expand the analysis submitted by Exxon Minerals with the revised air permit application. Project impacts were again evaluated for the fourth year of mine/mill operation. This year has the largest particulate emissions due to the excavation of tailings pond T2. Additionally, the first year of the project construction phase was evaluated. This year has significant particulate emissions from excavation activities, fugitive dust, and significant gaseous pollutant emissions due to diesel generator operation and open burning of cleared trees and brush. (See emission summary Tables D-1, D-2, D-3 and D-4 of the revised air permit application.)

The results of the analysis are presented in Tables 9 through 14 taken from the aforementioned April 4, 1985 DNR memorandum. Table 9 summarizes the predicted maximum criteria pollutant concentrations, representative background concentrations, and the total pollutant concentrations as they relate to the national ambient air quality standards. As with the Exxon analysis, the pollutant of greatest consequence is particulates. The maximum 24 hour average predicted impact for project operation is 48.9 ug/m^3. Combined with a background of 77 ug/m^3, the predicted worst case total is 125.9 ug/m^3. This, again, is below the particulate standard of 150 ug/m^3, and well below the primary standard of 260 ug/m^3. The predicted annual average impact was 5.7 ug/m^3. Combined with a background of 17.9 ug/m^3, the total is 23.6 ug/m^3. This is well below the secondary and primary air quality standards.

The particulate impacts predicted for the construction phase were lower than the operation phase for the short term, 24 hour averaging period, but higher for the annual averaging period. The 24 hour average was 41.4 ug/m^3, and the annual average was 7.9 ug/m^3. Combined with a background of 17.9 ug/m^3, the total annual concentration is 25.8 ug/m^3. This still is well below the applicable particulate standards. This is summarized in Table 9.

Table 9

Air Quality Summary

Pollutant	Period	NATIONAL AIR QUALITY STANDARD Concentration Primary ug/m3	Secondary ug/m3	CONCENTRATION Maximum Concentration Due to Exxon Sources Construction ug/m3	Operation ug/m3	Background ug/m3	Total Construction ug/m3	Operation ug/m3
TSP	24-hour	260	150	41.4	48.9	77.0	118.4	125.9
	Annual	75	60	7.9	5.7	17.9	25.8	23.6
SO_2	3-hour	No Standard	1300	***	160.9	25.0	***	185.9
	24-hour	365	No Standard	***	52.2	25.0	***	77.2
	Annual	80	No Standard	1.5	0.1	25.0	26.5	25.1
NO_2	Annual	100	100	3.0	0.7	19.4	22.4	20.1
CO	1-hour	40,000	40,000	***	7805	0.0	***	7805
	8-hour	10,000	10,000	***	1654	0.0	***	1654
Lead	Calendar Quarter	1.5	No Standard	***	0.019	0.0	***	0.019

***No air quality modeling done - insignificant emission rates.

As with the Exxon analysis, all other criteria pollutants had predicted impacts which are not expected to cause violations of the applicable ambient air quality standards.

The ore which will be mined contains a vast array of metals besides the copper, zinc and lead which will be recovered. These metals are expected to be found in the particulate emissions resulting from ore processing and waste tailings disposal. Emission rates for any given metal were calculated by assuming it comprised the same percentage of the particulate emissions as its concentration in the ore or tailings handled at the source. The sources of metals included in the analysis were underground mine operations venting through the exhaust raises, fine ore crushing and screening operations, fine ore bin loading and unloading operations and wind blown tailings off of an active tailings pond. Emissions from a tailings pond were estimated to produce a more conservative analysis. All other significant particulate sources do not involve ore handling.

Table 10 summarizes the maximum predicted concentrations for mercury, arsenic, cadmium, chromium, cobolt, copper, manganese and zinc. As no national ambient air quality standards yet exist for these pollutants, concentrations were predicted for averaging periods of currently available guideline levels of acceptability. A 30-day average guideline level of 1 ug/m^3 for mercury is provided in Section NR 154.19, Wisconsin Administrative Code. The maximum predicted calendar quarter average (nearly equal to the 30-day average) was 1.02 pg/m^3 (1.02 x 10^{-6} ug/m^3). Assuming no background concentrations, this mercury impact is insignificant with regard to the guideline.

The other metals were compared with short-term guideline levels derived from the occupational exposure threshold limit values or TLV's published by the American Conference of Governmental Industrial Hygienists. 2.4% of eight hour TLV's and 1% of ceiling limit TLV's are the guideline levels currently used by the DNR Bureau of Air Management while noncriteria pollutant rules are developed. As shown in Table 11, each metal investigated had predicted short-term concentrations well below the TLV derived quideline values.

Lastly, those metals which the US EPA Carcinogenic Assessment Group (CAG) have identified as potential human carcinogens were evaluated using CAG potency slopes. These potency slopes convert long-term pollutant concentrations into potential risk of developing cancer. As a result, cadmium, chromium and manganese were found to have extremely small predicted risk (i.e. 4.40 x 10^{-8} or less). This is summarized in Table 12. These are far below the 1 x 10^{-6} risk level currently used as a guideline of acceptability by most regulatory agencies. Arsenic was found to have a predicted risk of 1.24 x 10^{-6}. This is greater than the 1 x 10^{-6} level, but in this case is not considered high enough to merit some regulatory action (e.g. more stringent particulate control requirements). No action is considered necessary because 1) emission estimates are likely to be conservatively high and overpredict the impact, 2) no individual is likely to be exposed to the maximum long-term concentration for his or her lifetime, and 3) not all arsenic inhaled will be absorbed.

Thus, based on current guidelines, airborne concentrations of metals resulting from ore and tailings processing are not expected to pose a significant hazard.

Table 10

Mercury Concentrations

Standard NR 154.19, Wis. Adm. Code	Predicted Concentrations: Max. Con. Due to Exxon Sources ug/m3	Background ug/m3	Total ug/m3
1 ug/m^3 30 day average	1.02×10^{-6} (calendar quarter)	0.0	1.02×10^{-6} (calendar quarter)

Table 11

Toxic Metal Concentrations

Pollutant	Threshold Limit Valve ## % of TLV	Concentration	Maximum Predicted Concentration # For The Period (ug/m^3)
Arsenic	2.4% - 24 Hour	4.8 ug/m^3	6.8×10^{-2}
Cadmium	2.4% - 24 Hour	1.2 ug/m^3	7.8×10^{-4}
Chromium	2.4% - 24 Hour	1.2 ug/m^3	1.3×10^{-3}
Cobolt	2.4% - 24 Hour	2.4 ug/m^3	4.8×10^{-7}
Copper	2.4% - 24 Hour	24. ug/m^3	0.67
Manganese	1% - 1 Hour (ceiling)	50. ug/m^3	0.29
Zinc	2.4% - 24 Hour	120. ugm^3	2.83

Calculated from a percentage of TSP impact using allowable TSP emission rates.

Threshold Limit Values for Chemical Substances and Physical Agents in the Work Environment and Biological Exposure Indicies with Intended Changes for 1984-1985, American Conference of Governmental Industrial Hygienists, Cincinnati, OH, 1984.

Table 12

Cancer Potential

Pollutant	Unit Risk Factor* m^3/ug	Annual Maximum Concent. ug/m^3	Cancer Risk
Arsenic	4.0×10^{-3}	3.1×10^{-4}	1.24×10^{-6}
Cadmium	1.9×10^{-3}	3.5×10^{-6}	6.65×10^{-9}
Chromium	1.2×10^{-2}	6.2×10^{-6}	7.44×10^{-8}
Manganese	4.0×10^{-4}	1.1×10^{-4}	4.40×10^{-8}

*Source: Standards Applicable to Owners and Operators of Hazardous Waste Treatment, Storage, Disposal Facilities Under RCRA Proposed Additions to Standards for Hazardous Waste Incinerators, U.S. EPA, Research Triangle Park, NC, NTIS PB81-193021, 1981.

Table 13

Amounts of Metals Emitted (Stacks Only)

Pollutant	Amounts Emitted g/year	Amounts Emitted pounds/year
Lead	164334.	362.0
Arsenic	38454.	84.7
Cadmium	427.	.9
Chromium	756.	1.7
Cobolt	.26	5.7×10^{-4}
Copper	460135.	1013.5
Manganese	13475.	29.7
Mercury	99.	.2
Zinc	1906274.	4198.8

Table 14

Toxic Metal Deposition

Pollutant	Maximum Deposition, Off-Site Seasonal mg/m2	Annual mg/m2
Arsenic	1.8×10^{-2}	5.3×10^{-2}
Cadmium	2.0×10^{-4}	6.0×10^{-4}
Chromium	3.5×10^{-4}	1.0×10^{-3}
Cobolt	1.2×10^{-4}	3.6×10^{-3}
Copper	2.2×10^{-1}	6.3×10^{-1}
Lead	7.7×10^{-2}	2.3×10^{-1}
Manganese	6.3×10^{-3}	1.9×10^{-2}
Mercury	4.6×10^{-5}	1.4×10^{-4}
Zinc	8.9×10^{-1}	2.6

Table 14 provides a summary of predicted metal deposition rates. No air quality standards or guidelines are currently in use to evaluate deposition of metals. Thus, this data was provided to DNR personnel addressing project impacts on surface waters and terrestrial ecology. Their conclusions will be considered in the final determination on air permit issuance.

To verify pollutant emissions used for the air quality analysis, stack tests will be required as a condition of air permit approval for significant sources. Additionally, an ambient air quality monitoring program has been proposed by Exxon as required under the mining permit requirements of Chapter NR 132, Wisconsin Administrative Code. The preliminary monitoring plan includes the operation of four particulate monitors during the construction phase and three particulate monitors during the operation phase. Asbestiform fiber monitors will be operated during the fourth and fifth year of operation of the tailings pond T1 measure if any asbestiform fibers are present in the waste tailings. This monitoring plan, like the stack tests, will verify if the air quality impact analysis of the project was accurate and necessary particulate control measures are being undertaken.

OFFSET ANALYSIS

The proposed underground mine and surface facilities are not located in a nonattainment area, so offsets are not required.

EMISSION REDUCTION OPTION

This is not applicable.

PRECLUDE OTHER SOURCES

The operation of this facility will not preclude the construction of other air pollution sources in this area. The predicted air quality impact of this facility is well within current air quality standards.

CRITERIA FOR PERMIT APPROVAL

Sec. 144.393, Wisconsin Statutes sets forth the specific language for permit approval criteria. The following conclusions have been made:

1. The source will meet emission limitations.

2. The source will not cause or exacerbate a violation of an air quality standard or ambient air increment.

3. If the source is operating or seeks to operate under an emission reduction option, the required permit application for other sources participating in that emission reduction option are approvable. (Not applicable)

4. The source will not preclude the construction or operation of another source for which an application has been received.

OTHER IMPACTS

This project has been reviewed under the Wisconsin Environmental Policy Act Chapter NR 150, Wis. Adm. Code, to determine the short-term and long-term environmental and economic effects of this project upon the quality of the human environment. It has been determined that an environmental impact statement is required. This is now being prepared by the Department, coordinated by the Bureau of Environmental Analysis and Review. The decision on the permit for this project will be held pending the completion and approval of the statement.

DETERMINATION

The Bureau of Air Management, Wisconsin Department of Natural Resources has reviewed the materials submitted by the Exxon Minerals Company for an air pollution control permit for an underground mine, ore processing mill, and associated facilities and has made a preliminary determination that this project is approvable subject to the following conditions:

a. This permit does authorize an initial operation period of 180 days for equipment shake-down, testing and Department evaluation of operation to assure conformity with the permit conditions. Permanent operation of the source(s) covered by this permit after the initial operation period is prohibited until a release has been issued by the Department.

b. Source performance tests shall be conducted within 90 days after the start of initial operation to prove compliance with the particulate limitations for the underground mine operations, fine ore crushing and screening operations, and fine ore bin loading and unloading operations while operating at 80% or greater capacity and using U.S EPA tests methods identified in 40 CFR 60 Subpart LL. The Department shall be informed at least 10 working days prior to the tests so a Department representative can witness the testing. At the time of notification, a stack test plan following the provisions set forth in Section NR 154.06(5), Wisconsin Administrative Code, shall also be submitted for approval.

   Two copies of the report on the tests shall be submitted to the Department for evaluation within 30 days after the tests or at least 15 working days prior to the expiration of the initial operation period. Release for permanent operation will be issued only upon proof of compliance.

c. The wet scrubbers used to control particulate emissions from the fine ore crushing and screening operations, and the fine ore bin loading and unloading operations shall be equipped with monitoring devices for pressure drop across the scrubber and scrubbing liquid flow rate as required under the new source performance standards for metallic mineral processing plants (40 CFR 60 Subpart LL). The pressure drop monitoring device must be certified by the manufacturer to be accurate within ± 1 inch water (± 250 pascals) gage pressure and must be calibrated on an annual basis in accordance with manufacturer's instructions. The scrubbing liquid flow rate monitoring device must be certified by the manufacturer to be accurate within ± 5% of design scrubbing liquid flow rate and must be calibrated on at least an annual basis in accordance with manufacturer's instructions.

Quarterly reports shall be submitted to the Department of occurrences when the scrubber pressure drop and scrubbing liquid flow rate differ more than ± 30% from those measurements recorded during the most recent performance test.

d. Records shall be kept indicating daily ore and waste rock production rates, and explosive and diesel fuel oil usage rates for the mine. Production rates should be for those materials removed from the mine.

e. Prior to expiration of the project shakedown period, a malfunction prevention and abatement plan shall be submitted to and approved by the Department. This shall include a specific plan for control of fugitive dust during surface facilities operation.

f. All open burning of cleared trees and brush shall use air curtain destructors. Burning rates shall not exceed 500 pounds per hour.

g. Not later than 180 days after initial start-up, no ore processing related fugitive emissions shall exhibit an opacity greater than 10 percent using U.S. EPA Method 9 as required under the NSPS for metallic mineral processing plants (40 CFR 60 Subpart LL).

h. Diesel fuel oil usage by the mill backup generators shall not exceed 163,365 gallons per year.

i. Prior to expiration of the project shakedown period, emissions data acceptable to the Department shall be submitted which verifies the emergency diesel generators used for this project comply with the applicable particulate emission limitation.

Review Engineer: Steven Klafka SKlafka 26 April 1985

Checked by: DBR

Approved by: DBR

cc: M. Debrock - NCD
R. Ramharter - EA/3
R. Herbst - Exxon

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