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INVESTIGATION OF FEASIBILITY OF DEWATERING AND OTHER ALTERNATIVES FOR OPEN PIT MINE OPTION, NEAR CRANDON, WISCONSIN

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May 20, 1977

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Job No. 8837-051-07

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DAMES & MOORE
DAMES & MOORE

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INTRODUCTION

GENERAL

In this investigation we have evaluated several conventional dewatering methods and how they apply to the large possible open pit mine near Crandon for Exxon Company, U.S.A. The main dewatering methods considered for use were peripheral dewatering wells, horizontal collectors within the excavation, well points, and a combination of all these methods. In accordance with a verbal request from Mr. Robert Russell of Exxon, we have also considered alternative methods of maintaining a dry excavation and pit. These methods include slurry trenches, chemical grout curtains, and freezing.

The possible open pit considered for the Crandon project would be located over the east-west trending ore body. The general pit outline is shown on Figure 1. This upper pit outline was transmitted to Dames & Moore by Exxon on March 1, 1977. The approximate areal dimensions of the elongate pit are about 6000 by 2500 feet at the surface. The pit would extend through the total thickness of overburden

Not Exxon. Should be ≈550 ft Jim [illegible] of Mining Engineer.

and about 400 feet into the ore body. According to Exxon Company, U.S.A., the excavated walls in the pit would have 3 horizontal to 1 vertical slopes through the overburden and 1 to 1 in the crystalline rock.

We have estimated that about 85 to 100 million cubic yards of overburden would be excavated. The overburden thickness at the site ranges from about 90 to over 225 feet. The overburden is generally thicker at the western and central portions of the site and thinnest at the east end of the pit.

Although the possible open pit would extend into the crystalline bedrock and ore, we have limited this study to the overburden dewatering as per our proposal. For the purposes of this study, we have assumed that the bedrock in the area is impermeable and will not transmit water to the excavation by upwelling.

PURPOSE

The purposes of the dewatering investigation were the following:

1. Evaluate the geohydrology of the unconsolidated materials overlying the ore body;
2. Make a preliminary design of the dewatering system for the site, including the size and number of wells needed, approximate pumping rates, spacing of wells, and estimated costs;
3. Assess the potential environmental effects of the local ground and surface water systems and measures that could be used to mitigate any adverse effects; and
4. Make a conceptual design of several alternative methods of maintaining a dry pit and assess their feasibility and costs.

SCOPE OF WORK

The field work completed for this investigation included the drilling of four test borings and installing seven piezometers, drilling and construction of two 8-inch ID test wells and conducting one 24-hour pumping test and one 18-hour recovery test, and one 48-hour pumping test and one 24-hour recovery test. One 24-hour or longer and one short-term (4 to 8 hour) pumping test remain to be conducted; the results will be sent to Exxon in an addendum letter report.

Office work performed for this report included analysis of the pumping test results, evaluation of site geology, a computer modeling and analysis of the several dewatering systems, a cost analysis of the various methods of dewatering, and an assessment of the environmental effects of an open pit dewatering system.

Field Investigations

Four borings were made for the purpose of evaluating the overburden geology and to install isolated piezometers tapping each of the main aquifer zones and the intermediate till (DW-1U). Two borings (DW-1, DW-1A) and three piezometers were installed near Water Well -2 near the middle of the ore body (see Figure 1). This 8-inch ID well was originally drilled by Exxon to obtain water for drilling. One boring (DW-2U,L) with two piezometers was made near the northeast edge of the ore body (Figure 1). The third location was near the southwest edge of the ore body (Figure 1). One boring with two piezometers (DW-3U,L) was made at this location. The construction features of the piezometers and the lithologic logs for these borings are given on Figures 2 through 4, cutting Logs? Split Spoons?

Two test wells were constructed at the site area (see Figure 1): one near the northeast piezometers and one near the southwest piezometers. The test wells consisted of 8-inch ID for steel casing (ASTM-538) and 10 and 15 feet of 8-inch stainless steel screens installed in 12-inch diameter borings. Each hole was drilled with a Schramm rotary drilling rig. After placing the 8-inch ID casing in the hole, the 8-inch nominal screen equipped with neoprene upper seal and a bottom plate was set through the casing to the desired depth. The casing was then pulled back to expose the screen. Then pea gravel was placed around the several feet above the screen. Details of the two test wells are shown on Figure 5.

Two pumping tests were conducted at the site. A 24-hour pumping and 18-hour recovery test of Test Well -1 was conducted April 1 through April 3, 1977. The well was pumped with an airlift pump at an average rate of 80 gallons per minute.

Water Well -2 at the site was pumped continuously for 48 hours and recovered for 24 hours from April 11 through April 14, 1977. The average pumping rate was 48 gallons per minute using the existing submersible pump.

SITE CONDITIONS

GEOLOGY

The unconsolidated deposits that overlie the ore body and bedrock in the site and vicinity consist of glacial deposits ranging from about 90 to over 200 feet in thickness. Generally, the overburden can be classified as outwash deposits, till, or lacustrine deposits. The outwash was generally deposited in a high energy environment as the glacier meltwaters flowed and sorted existing till or other deposits. Outwash materials are mostly sands and gravels with some cobbles and boulders, as finer-grained silts and clays were likely held in suspension in the meltwater streams and were not deposited simultaneously with the coarser fraction.

The till is a heterogeneous mixture of sand, silt, gravel, and boulders with a trace of clay. Till was deposited directly during either the advance or recession of a glacier and was not water sorted.

Lacustrine deposits occur mostly in swamps and lowland areas and include peat, fine sands, and silts. These materials were generally deposited in relatively calm bodies of water. Figure 6 shows the location of geohydrologic sections shown on Figures 8 and 9. Figures 7 through 9 show the geologic characteristics of the unconsolidated deposits near the site.

The glacial deposits at the site and vicinity overlie crystalline igneous bedrock. The upper portion of the bedrock is a weathered zone, commonly a red clay. The weathered zone of rock varies in thickness over the site. Weathered rock grades into unweathered rock at greater depths.

GEOHYDROLOGY

The geohydrologic setting in the overburden in the site area is basically two sand and gravel aquifers with confining beds of till. The main aquifer (hereafter termed upper aquifer) is a sand and gravel deposit that is continuous over much of the site. A till deposit overlies the upper aquifer over most of the site.

The upper aquifer is underlain by another till, which in turn overlies a thinner discontinuous sand and gravel aquifer (lower aquifer). The lower aquifer is present at the west end of the site. Cross sections (Figures 7 to 9) show the relative subsurface position of the geohydrologic units present in the site area.

The upper aquifer appears to be continuous over much of the site. This aquifer ranges from fine to coarse sand with varying amounts of gravel and often contains large boulders. The unit ranges in thickness from about 25 to 60 feet near the exploration site. The bottom of this aquifer ranges from an elevation of about 1540 at the east end of the site to a high of about 1620 near the center of the site and elevation 1560 at the west end. Figure 10 is a contour map showing the approximate elevation of the base of the upper aquifer in the site area. The ground water levels measured in piezometers tapping this aquifer indicate that the aquifer is artesian at the east end and under water-table conditions at the west end of the site. Water levels in this unit vary from about 1580 to 1590 feet elevation (MSL).

The lower aquifer occurs below the upper aquifer and is separated by as much as 80 feet of silty sand till. This aquifer appears to be a lens as it is not present over part of the site. It ranges from about 5 to 20 feet thick where present, which is mostly under the western half of the site. This aquifer is overlain, underlain and confined by till. Although the till is a confining unit, it is assumed to be the main source of recharge to the lower aquifer. Ground water in the lower aquifer is under artesian conditions with potentiometric levels similar to those in the upper aquifer (1580 to 1590 feet elevation, MSL).

In addition to the above noted two aquifers, it is likely that several thin, discontinuous sand and gravel lenses occur within the till at the site. These lenses may be expected to be of limited thickness and horizontal extent.

Till deposits in the overburden have relatively low permeability and tend to restrict ground water flow. The till is generally a heterogeneous mixture of sand and silt with some gravel and boulders, is mostly quite dense and contains varying relative amounts of sand and silt. The upper till at the site appears to be continuous and ranges in thickness from about 15 to 45 feet. This deposit acts as a confining bed over the upper aquifer at the east end of the site and probably restricts the amount of natural recharge reaching the underlying aquifer.

Another till occurs below the upper aquifer and appears to extend to bedrock except for where the lower aquifer is present. The lower till is generally reddish-brown in color and similar in composition to the upper till. The unit ranges in thickness from about 30 to 130 feet.

The weathered and unweathered bedrock units at the site will generally act as an impermeable barrier to ground water flow. The weathered material above the rock is mostly a silty clay material with rock particles. The un-weathered bedrock is dense and would have a very low natural permeability.

AQUIFER PARAMETERS

The first pumping test was conducted at Test Well -1, which taps the upper aquifer at the northeast end of the proposed pit. The aquifer is artesian in this area and test results indicate the aquifer transmissivity ranges from about 30,000 to 64,000 gallons per day per foot of aquifer (gpd/ft). With approximately 50 feet of saturated material at piezometer DW-2U, and an average transmissivity of 50,000 gpd/ft, the average permeability of the upper aquifer is about 1,000 gallons per day per square foot of aquifer (gpd/ft^2). The coefficient of storage calculated from piezometer DW-2U data collected during the pumping test of Test Well -1 is about 2×10^{-4}. Since much of the upper aquifer is under water-table conditions, we have assumed an average storage coefficient of 0.05 for the aquifer.

A pumping test of the lower aquifer was run using Water Well -2 as the pumping well, and Observation Well DW-1L as the primary monitoring point. Results indicate that the average transmissivity is about 8,000 gpd/ft and the storage coefficient about 2×10^{-4}. Based on the test results and an average aquifer thickness ranging from 8 to 15 feet, the average permeability of the lower aquifer probably ranges from 600 to 1000 gpd/ft^2. The vertical permeability of the lacustrine silts and till deposits were estimated to range from 0.1 to 0.01 gpd/ft^2. The basic data for the pumping tests are in the Appendix.

SURFACE WATER - GROUND WATER REGIMES

The main surface water bodies in the site area (see Figure 6) are Swamp Creek, Hemlock Creek, Little Sand Lake, Oak Lake, Skunk Lake, Duck Lake, Rice Lake, and Mole

Lake. Swamp Creek is the primary drainage basin near the site. Hemlock Creek flows north-northwest into Swamp Creek about 1.5 miles northeast of the site. Swamp Creek flows west to Rice Lake and is located about 1 mile north of the site.

Little Sand Lake is located immediately south of the site and at its northernmost point is about 400 feet from the edge of the proposed pit outline. Oak Lake is south of the west end of the possible pit. This lake is smaller than Little Sand Lake but deeper. Skunk Lake is immediately northeast of the northeast end of the site, while Duck Lake is about 1 mile south of Skunk Lake. Mole Lake and Rice Lake are located about 2 miles directly west of the site.

The surface water regime in the site vicinity is somewhat complex and variable. The average annual precipitation in the region is about 31 inches and the average runoff is about 12 inches. Thus of the 31 inches total, about 19 inches are lost by evaporation and transpiration; the remaining 12 inches make up the flow in the rivers and streams. Swamp and Hemlock Creeks are fed by surface water outflow from lakes and swamps in their drainage basins. In addition, ground water seepage reaches these streams where permeable deposits intersect the streams.

The lakes in the site vicinity appear to be related to the overall ground-surface water regime to some degree. The Little Sand Lake water elevation was about 1591 feet above sea level (MSL) May, 1977. Ground water elevations northeast and north of the lake were (April, 1977) between 1588 and 1590 feet MSL at piezometers DMA-19 and DW-2U, respectively. The water level in piezometer DMA-10, located south of Little Sand Lake, is about 10 or more feet below lake level. These data indicate that Little Sand Lake may slowly feed the ground water regime and not vice-versa. Ground water elevations near Oak, Skunk, and Duck Lakes are presently lower than the lake levels, indicating that these lakes are not directly connected to the underlying aquifers.

It is likely that the materials underlying the lakes near the site are low permeability deposits of silt, fine sand, and till, and the interconnection between these lakes and the ground water regime is mostly by slow movement through these confining beds.

Based on ground water levels at piezometers DMA-16, DMA-20, and DMA-47, it appears that Swamp Creek north and northwest of the site is being fed by ground water inflow. It is not clear whether the upper aquifer at the site is connected with Swamp Creek here, but as shown on Figure 8, it seems that the upper aquifer thins and pinches out before it reaches the valley. Some seepage through the till or through springs where the aquifer does intersect the valley wall probably does serve to maintain baseflow in Swamp Creek.

Ground water flow in the upper aquifer at the site was found to be under very low gradients. As shown on Figure 7, it appears that the aquifer is not saturated near the middle of the site. This indicates that there is no ground water flow in this aquifer in this area. West of this barrier ground water movement is probably to the north and south in the upper aquifer. Ground water seems to be moving towards the south in this aquifer at the east end of the site.

Ground water movement in the lower aquifer is not well defined due to lack of control data, but at the western portion of the site, ground water elevations in the lower aquifer are about the same as in the upper aquifer and flow is probably in the same general directions except where the aquifer is absent.

The main source of natural ground water recharge is precipitation, which infiltrates the ground and percolates to the saturated zone in the upper aquifer. Some ground water recharge is probably by leakage from nearby lakes. The lower aquifer at the site probably receives recharge by leakage through the overlying till.

The main ground water discharge areas appear to be the major streams including Hemlock Creek, Swamp Creek, and the Wolf River as well as some of the large lakes and swamp areas.

LOCAL GROUND WATER USE

The primary off site use of ground water in the vicinity is for domestic purposes. Although a well inventory was not done during this study, the main center of year-round ground water users is about 1.5 to 2 miles west of the site at Mole Lake and the Mole Lake Indian Reservation. There are also about 7 to 10 permanent residences along Little Sand Lake Road between State Route 55 and the site. It is assumed that these homes use ground water supplies. Several cottages or summer homes around Little Sand Lake likely utilize ground water for limited times during the year, primarily during the summer.

ALTERNATIVE DEWATERING METHODS AND PRELIMINARY COST ESTIMATES

We have evaluated the following methods of dewatering or ground water control for use at the possible open pit of the Crandon site: slurry trenches, grout curtain, freezing methods, deep peripheral wells, well points, horizontal collectors, and combination systems using two or more of the above methods. The cost estimates given in the sections that follow serve only as general guidelines that could vary considerably depending upon the timing, the contractor used, and more specific site information.

SLURRY TRENCHES

The slurry trench method involves excavating a deep narrow trench through the permeable zones and backfilling the trench with a very low permeability slurry such as a bentonite mud. The trench walls should be as near vertical as possible and should be excavated by conventional methods such as drilling or dragline. Boulders in the overburden would cause additional problems in terms of trench excavation. The required depth is at about the upper limit of present day equipment. A bentonite slurry is most commonly used to make the impermeable curtain.

The use of slurry trenches to stop the movement of ground water toward an excavation would require trenches ranging from 80 to 175 feet deep at the site.

If slurry cutoff trenches were used as the primary ground water control system, we estimate that about 12,000 lineal feet of the perimeter would require the slurry trench. This distance assumes no trench needed where the aquifer base is dry.

The cost of a slurry trench cutoff system at the site was estimated by assuming an average of 100-foot-deep trench 12,000 feet long around much of the perimeter of the possible pit. Based on estimates from experienced contractors, such a system would cost about $7.50 per square foot (cross sectional area) or about $9,000,000 for a trench 12,000 feet by 100 feet deep. The estimated capital and annual operating costs for this system are given in Table 1. It has been assumed that an interior sump and ditch system would be required in addition to the slurry trench to assure the integrity of the slopes. The slurry trench cannot be assumed to be 100 percent efficient without increasing the costs significantly.

CHEMICAL GROUTING

Chemical grouting methods of ground water control involve drilling a line of small diameter holes into the permeable zones that require control. Chemical grout is injected under pressure, into the permeable zones forming a "wall" of very low permeability soils.

As the primary system at the site, the chemical grout curtain that would be required is estimated to be about 12,000 lineal feet with holes on 4-foot centers. This assumes that no curtain is needed where the aquifer is dry. The average hole depth would be an estimated 100 feet but the average curtain height would be about 50 feet. The hole cost is estimated at $5/foot.

The rough estimated cost is about $12 to $14 per square foot of cutoff area. For a 12,000-lineal-foot by 50-foot high curtain or cutoff area, and for 450,000 feet of drilling, the system would cost about $10,000,000. The estimated capital and annual operating costs for this system are given in Table 2. We have again assumed that an interior sump and ditch system would be required in addition to the grout curtain.

FREEZING METHODS

Freezing methods to control ground water movement generally involve pumping brine-type solutions through piping installed in borings. The brines freeze the water and soil near the borings, thus causing a barrier to ground water movement. These type systems are usually used for temporary control of ground water during construction of shafts and smaller pits when other methods fail. Very large numbers of holes and large amounts of piping are required. High energy input is required to operate the overall system.

The feasibility of using freezing methods to control ground water at the possible pit appears to be poor.

The systems are generally used only for temporary localized conditions; to utilize a freezing system as the primary method of ground water control at the Crandon site would not be cost effective. In addition, high energy inputs would be required over the life of the project. We feel costs for a freezing system would be several times higher than for the slurry trench or grout curtain methods.

SUMMARY OF ALTERNATE DEWATERING METHODS

It was found that in general, the large size of the pit and thickness of the overburden would likely prohibit the use of slurry trenches, grout curtains and the freezing methods. These methods would cost much more than conventional dewatering methods, and the overburden soils, gravel and boulder content would probably further increase the installation time of these systems. The use of dewatering wells, horizontal gravity collectors and well points appears to be more feasible in terms of economics, time, and flexibility as will be explained in subsequent sections.

DEWATERING COMPUTER MODEL

A computer program was developed after Prickett (1971) to simulate the hydrogeologic conditions in the vicinity of the possible open pit mine. In the program, the upper aquifer, which was discussed previously, is replaced by an equivalent set of discrete elements. The digital computer treats both space and time variables as discrete parameters. Also, the equations governing the flow of ground water in the simulation model are rewritten in finite difference form. The resulting set of finite difference equations is solved numerically by means of a computer.

A finite difference variable grid system was designed to concentrate on the head distribution in the pit area due to dewatering, and to determine the effects of pumping on nearby Little Sand Lake. Although preliminary data indicate that there is little exchange of water between the ground water region and the lake, the leakage factor was modeled for conservation. The intersections of grid lines are called nodes, which are referenced with a column (i) and row (j) coordinate system. The variable grid system consists of 25 columns and 29 rows of grid lines, covering an area of about 170 square miles. Figure 11 shows the variable grid system used in the dewatering computer model.

The spacing between nodes at and near the open pit was set at 500 feet. This value was selected as an estimate of the probable well spacing or a multiple of the well spacing. However, with increasing distance, convergence in the model can be achieved at a greater grid spacing without sacrificing accuracy of the solution. A detailed description of the head distribution at great distances from the proposed excavation was not of major concern in this report. The variable grid system was utilized to minimize core storage and reduce computer execution time and costs.

After studying and interpreting field boring logs and pumping test data, a model aquifer representing the upper aquifer (see Figures 7 to 9) was developed. Figure 12 is a typical north-south cross section of the model aquifer. From field data average values of 30,000 gpd/ft and 0.05 were chosen for the aquifer transmissivity and storage coefficient, respectively. The vertical permeability (P') of the semipervious sediments below Little Sand Lake was estimated to be 0.04 gpd/ft^2. The model aquifer has a thickness of 30 feet, is bounded below by impermeable strata, is completely saturated initially and is under water-table conditions. In reality, the aquifer is partially saturated at the west end and under a slight artesian head at the east end of the pit. Figure 11 shows the barrier boundaries that were placed in the model by setting values of transmissivity equal to zero along Swamp Creek and Hemlock Creek. These barrier boundaries were incorporated into the model because boring log and field reconnaissance information indicate that the upper aquifer is pinched out by an overlying till unit south of Swamp Creek and west of Hemlock Creek (see Figure 8), and the upper aquifer is hydraulically connected to Swamp and Hemlock Creeks only through seepage and springs north of the site area. At these locations, ground water seepage at the surface moves downhill to recharge Swamp Creek and probably Hemlock Creek.

Field data from borings in the immediate vicinity of the site also indicate that the upper aquifer is mounded in the center of the pit area. Within this mounded area, the water levels are at an elevation of about 1590 (see Figure 7) and are below the base of the upper aquifer. Therefore, the upper aquifer is completely dewatered in this area and a ground water barrier exists. Figure 10 indicates that this "high" region probably extends for some distance to the north and south of the pit area. A barrier (no flow) boundary (see Figure 11) was incorporated into the computer dewatering model, and values of transmissivity were again set to zero at the appropriate nodes to represent this region where no flow is occurring.

Because of the close proximity of Little Sand Lake to the possible open pit mine, and its apparent hydraulic connection with the upper aquifer, dewatering of the upper aquifer may induce surface water from the lake to recharge the upper aquifer. During dewatering operations, water levels in the upper aquifer will be lowered below the surface level in Little Sand Lake creating a potential for induced infiltration through the lake bottom. The quantity of induced infiltration from Little Sand Lake from dewatering of the upper aquifer was calculated with the model based on estimates of the vertical permeability of the lake bottom. The vertical permeability (P') of the silts and clays beneath the lake bottom was estimated to be 0.04 gpd/ft^2. Although no field samples of the lake bottom were obtained, this low value of vertical permeability is probably representative of typical lake bottom sediments from this region. Induced infiltration was calculated at each node representing Little Sand Lake (see Figure 11 and the sum of these values approximates the total amount of recharge from Little Sand Lake at any given time. Figure 12 shows the hydrologic relationship between the model aquifer and Little Sand Lake.

Initially, the model aquifer is in equilibrium with no flow occurring anywhere within the grid system and all heads are set equal to a zero reference level. When dewatering operations begin, water levels in the aquifer decline, and induced infiltration from Little Sand Lake occurs. The barrier boundaries and the induced infiltration will affect the rate of water level decline throughout the upper aquifer. The model output shows the drawdown effects at each node due to pumping. Drawdown effects at pumping nodes have to be adjusted to account for converging flow in the model.

The average adjustment factor was calculated to be approximately 8 feet.

A model aquifer representing the lower aquifer (see Figures 7 through 9) was also developed. Figure 12-A is an east-west cross section of the model aquifer. From the field data, average values of 8000 gpd/ft and 0.0002 were chosen for the lower aquifer transmissivity and storage coefficient, respectively. The vertical permeability (P') of the confining till unit was chosen as 0.02 gpd/ft. The model aquifer has a thickness of 15 feet, is overlain by a semipervious till unit with a thickness of 75 feet, is bounded below by impermeable strata, and is under artesian conditions. Field data indicate that the lower aquifer is pinched off near the middle of the pit. Therefore, the barrier boundary bisecting the pit was used as a no-flow boundary representing the discontinuity of the lower aquifer. Available field data suggest that the lower aquifer is not hydraulically connected with any surface water bodies in the vicinity of the site.

Initially, the model aquifer is assumed to be in equilibrium with no flow occurring anywhere within the grid system. Water levels are set at the top of the confining till unit. When dewatering operations begin, water levels in the lower aquifer will decline until the contributions from leakage through the aquitard balances the well discharge. However, in reality, water levels will continue to decline because the source bed will be virtually removed locally by excavation of the pit.

CONCEPTUAL DEWATERING SYSTEM

The physical dimensions of the possible open pit are about 6000 by 2500 feet and are shown on Figure 1. Conceptually, the pit is bisected by a barrier boundary (naturally unsaturated part of the aquifer) that trends in a north-south direction through the area. The upper and lower aquifers will have to be dewatered near the face of and within the pit itself to permit operation of the pit and to protect against slope instability.

UPPER AQUIFER

A total of 20 wells, each 6 to 8 inches in diameter, will likely be needed for the upper aquifer dewatering operation with some of the wells on standby. Most of the wells on the eastern side of the pit will be concentrated near Little Sand Lake to collect induced infiltration and to maintain water levels at required depths near the face of the pit. Figure 13 shows the proposed locations of wells that will be needed to effectively dewater the upper aquifer.

Two pumping schemes were used to describe the drawdown effects for short- and long-term pumping periods. Firstly, 12 of the 20 dewatering wells were pumped at rates from 100 to 150 gpm for a period of 240 days. Figure 14 is a contour map of the drawdown effects for short-term pumping in the upper aquifer. At these pumping rates, the upper

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An approximate schedule of pumping rates for the upper aquifer is given in Table 3. Here 15 pumping wells are suggested to optimize the system. Another 5 wells would be on standby.

LOWER AQUIFER

Figure 13 also shows the proposed locations of wells that will be needed to reduce the artesian head in the lower aquifer, eventually dewatering a major portion of the aquifer. This aquifer is probably discontinuous (see Figure 8). A total of six wells, each 6 to 8 inches in diameter, will be needed, with two of these wells acting in a standby capacity. Four of these wells will be pumped at an approximate rate of 200 gpm.

The dewatering wells in the model are pumped continuously, resulting in an initial total discharge of about 1.2 mgd.

As the source bed above the lower aquifer is dewatered by dewatering wells and horizontal collectors, the lower aquifer will be deprived of recharge. Therefore, rates of water level decline will increase until water-table conditions prevail in the lower aquifer. Additional pumping will rapidly dewater the remaining saturated thickness. When the depth of excavation penetrates the lower aquifer, using a bench system similar to the one shown on Figure 16, horizontal wells will be installed to minimize seepage at the excavation face, and a ditch will be made to capture any remaining seepage and divert it to a sump system for discharge to the surface.

Figure 17 is a contour map of the drawdown distribution in the lower aquifer after pumping for a period of 900 days at a rate of 1.2 mgd, indicating that drawdowns up to 80 feet could be achieved. Actual drawdowns would probably be much greater once the upper aquifer and till is lost as a source bed for the lower aquifer in the vicinity of the excavation. Thus, water levels would decline at a faster rate. When water levels reach the top of the lower aquifer, water-table conditions will prevail. After a short period of time the relatively thin lower aquifer (about 15 feet) will be dewatered because of rapid reductions in saturated thickness and transmissivity.

HORIZONTAL COLLECTOR WELLS

It is physically impossible to completely dewater both the upper and lower aquifers with wells. It will be necessary to reduce the water levels in all materials to a considerable depth beneath the base of the aquifer and to the top of the bedrock. Thus, an additional dewatering system, such as horizontal collector wells, sumps, well points or a combination of these systems, will be needed to effectively dewater the remaining saturated areas, near the face of the excavation where slope instability conditions could occur. One possible companion system could be a series of well points located on benches cut into the face of the excavation. Although well points could possibly be used locally within the pit during the initial excavation, especially prior to the installation of horizontal collectors, their limited efficiency and pumping lift would preclude their effective use on a permanent basis.

A more feasible companion system is a series of horizontal collector wells, and a header system around the perimeter of the excavation at some depth. A schematic diagram of a combined dewatering system consisting of dewatering wells, horizontal collector drains, ditches, and sumps is shown on Figures 13 and 16. Horizontal collector drains would be about 150 feet in length, and spaced 50 to 100 feet apart. Using Darcy's law, where $Q = KiA$, the amount of water flowing to the horizontal drains along the entire perimeter of the pit was estimated to be about 600,000 gpd after 500 days of dewatering from about 200 drains tapping the upper aquifer. Another 50 drains yielding 300,000 gpd would tap the lower aquifer. Therefore, each drain in the upper and lower aquifers would discharge at a rate of about 2 and 3 gpm, respectively.

Water in storage between the dewatering wells and the face of the excavation would be collected at some distance into the exposed face of the aquifer and would move under gravity drainage to the header system. Subsequently, these waters would be removed from the excavation by pumping from the collector header or sump. Also, some horizontal collectors will have to be placed in the upper and lower till units to capture remaining waters that might conceivably reach the face of the excavation, creating slope instability problems. A ditch along the bench would capture any remaining seepage and transport it to a sump system for discharge to the surface.

PRELIMINARY COST ESTIMATE OF CONCEPTUAL COMBINATION DEWATERING SYSTEM

DEWATERING WELLS

Figure 12 is a schematic of the dewatering well system including standby wells, header collection system, sumps, and ditches. A total of 20 wells, 6 inches in diameter, will be utilized during the dewatering operation for the upper aquifer with 5 to 8 of these acting in a standby capacity. The dewatering well system will pump approximately 2.2 mgd during the first 4 to 6 months. As pumping rates are decreased over the period of dewatering to maintain the desired water levels, power costs will also be reduced. Table 3 gives an estimate of one possible pumping schedule for dewatering the upper aquifer. In addition, 4 to 6 wells will tap the lower aquifer at the western end of the excavation. These wells will be necessary to reduce the existing artesian head of approximately 80 feet and eventually dewater the lower aquifer. A total dewatering system similar to that proposed for the upper aquifer will also be used for dewatering the lower aquifer. The cost of the dewatering well system is itemized in Table 4. Capital costs for the initial installation of the dewatering well system are estimated at $1,260,000. Operation and maintenance costs, much of which will be shared with the horizontal collector system are estimated at $275,000. The power costs in Table 4 were estimated assuming an average dynamic head of 200 feet, an average pumping rate of approximately 2.2 mgd, months, and a pump efficiency of 50 percent. However, initial pumping rates may be somewhat greater to achieve maximum drawdown as soon as possible, without reducing the efficiency of the pumping equipment.

HORIZONTAL COLLECTORS, DRAINS, AND SUMPS

Figure 17 is a section through the various units at the site delineating the horizontal collector system which consists of horizontal drains, header pipes, ditches and sumps. Inflow to the drains and any additional seepage along the face of the excavation will be collected to preclude the possibility of slope instability conditions occurring. The horizontal collector system for both the aquifer and the confining units will include approximately 200 drains spaced at 50 to 100 foot intervals or as conditions dictate, about 16,000 feet of header pipe (6-inch PVC) to collect drainage, continuous ditches to capture additional seepage, sumps to accumulate the ground water inflow and pumps to discharge these waters at the surface. The cost of the horizontal collector system is itemized in Table 4. Capital costs for the horizontal collector system are estimated at $711,000. After the collector system is in operation an additional pumpage of 900,000 gpd (both aquifers) will have to be removed from the header, ditch and sump system. When the horizontal collector system is installed in the upper aquifer, the total pumping rate will be 1.7 mgd for the entire upper aquifer dewatering system (see Table 3). The electrical cost associated with the increased discharge rate of 0.9 mgd from the horizontal collects is estimated at $15,000/year; however, at this time, the initial electrical costs for the dewatering wells will be reduced to about $25,000. Therefore, after 4 to 6 months the total electrical cost associated with pumping will be reduced from $52,000 to $40,000.

The total capital cost of the dewatering system, including dewatering wells, horizontal collectors, header pipes, pumps, ditches and sumps is estimated at $1,971,000. The total cost for operation and maintenance of the combined dewatering system is estimated at $275,000/year. It is assumed that the operation and maintenance on both the dewatering well system and the horizontal collector system can be accomplished simultaneously with the suggested manpower. Obviously, electrical costs for pumping will fluctuate depending on the pumping rates for the total system in operation at that time. Therefore, the figures shown in Table 3 should be considered as best estimates for that discharge rate.

ENVIRONMENTAL EFFECTS ON DEWATERING

NEARBY WELL SUPPLIES

The drawdown distributions from the dewatering operation indicate that wells tapping the upper aquifer in the area will be affected by the dewatering operations at the possible mine pit site. Figures 15 through 17 show that drawdowns up to 5 feet occur at distances over 1 mile southeast of the site after 6 months of pumping. The known well supplies near the northern end of Little Sand Lake will experience increased drawdowns because of the concentrated pumping at the southeast end of the excavation. Drawdowns up to 12 feet could occur in this area (see Figures 15 through 17). Well supplies north of Swamp Creek and east of Hemlock Creek will not be affected by the dewatering operation because the upper aquifer is pinched out to the north and east. This pinch out (see Figures 7 through 9) acts as a no-flow boundary; also the creeks would hydraulically separate any shallow ground water effects opposite the mine.

Drawdown effects in the lower aquifer will be substantial (see Figure 17) during dewatering operations. However, no wells are known to tap the lower aquifer in the vicinity of the possible open pit mine.

WATER LEVELS IN NEARBY WATER BODIES

After interpreting boring logs and studying the geohydrology of the area, including the apparent barrier boundaries, we believe that Little Sand Lake, Hemlock Creek and Swamp Creek probably will be either directly or indirectly affected by the dewatering operation. Higher water levels in the other lakes surrounding the area, such as Oak Lake and Skunk Lake (see Figure 6), indicate that these surface water bodies are probably not hydrologically interconnected with the upper and lower aquifers. These lakes have water levels higher than those found in the underlying aquifers.

When water levels in the upper aquifer decline during dewatering, induced infiltration through the lake bed increases until water levels in the aquifer drop below the base of the lake bed confining unit. At this point, maximum recharge to the upper aquifer through induced infiltration will occur. Using an estimated vertical permeability (P') of 0.04 gpd/ft^2, the amount of water lost to the aquifer from Little Sand Lake was calculated to be approximately 330,000 gpd (about 225 gpm) after 180 days of pumping. This is viewed as very conservative because preliminary data indicate that the ground water regime may be fed by the lake and not vice-versa.

The upper aquifer is indirectly connected with Swamp Creek and Hemlock Creek, thus, contributing some ground water discharge to the creeks in the form of seepage both north and east of the site. Although the amount of seepage could not be quantified with the dewatering computer model, the presence of springs, such as Hoffman Springs, indicates that indirect recharge is occurring. The reduction in seepage discharge to these springs due to dewatering could be mitigated by monitoring major spring discharges before dewatering operations and maintaining those discharges (i.e., water levels in springs) during dewatering operations with some type of recharge mechanism.

In order to mitigate any possible environmental effects of dewatering on Swamp Creek, Hemlock Creek and Little Sand Lake, several recharge pits could be located as shown on Figure 19. Discharge waters from dewatering operations could be diverted to these pits to maintain water levels in affected surface water bodies. In addition, these recharge pits would maintain water levels in the swampy lowlands that would likely decline during dewatering operations from either direct or indirect causes.

The initial quantity of pumpage from upper aquifer dewatering wells will be approximately 2.2 million gallons per day (mgd), thence to 1.6 mgd after about 200 days. After about 400 days of continuous pumpage, when water levels in the upper aquifer are at lower levels, a substantial reduction in this quantity of pumpage, down to 1.1 mgd, will be made by pumping at reduced rates (see Table 3) using automatic water-level activated relays. The additional drainage from horizontal collectors and small seeps, estimated at 0.6 mgd, in the various sand and till units will be collected by means of a header system, ditches and sumps. The total quantity of discharge waters from the dewatering operation will be used to maintain water levels in those water bodies probably affected by the dewatering operation, specifically Little Sand Lake, Swamp Creek, Hemlock Creek and surrounding wetland areas. Water from net precipitation falling on the bottom of the pit, estimated at about 340,000 gpd, cannot be used as recharge because this water would have contacted the ore and would likely contain trace metals. Thus, the accumulation of net precipitation would be collected in sumps in the mine and pumped to the tailings pond or used as process water.

An important environmental consideration is the eventual abandonment of the open pit. If the pit is not partially backfilled with waste rock, it will have a volume of about 62,000 acre feet. The pit would eventually fill with water to the natural ground water level. This would take about 45,000 to 50,000 acre feet of water. Approximately 600 acre feet/year would come from net precipitation and another 650 acre feet from controlled infiltration. The controlled infiltration is estimated at 50 percent of the long-term withdrawal rate of 1.7 mgd , because part (estimated at 50 percent) of the water would probably have to be recharged to the aquifer to avoid possible draining effect on

Little Sand Lake to fill the pit. Thus, reduced pumping would have to continue even after open pit mining ceased. Conceivably, it could take several tens of years to fill up the pit to equilibrium water levels.

RECOMMENDATIONS

Based upon this feasibility study, we believe that the combined dewatering system described in this report, utilizing peripheral dewatering wells, horizontal collectors, ditches and sumps, would be the most suitable dewatering plan if the open pit alternative is chosen.

The relative costs for the slurry trench, grout curtain, and freezing methods for ground water control are prohibitive compared with costs for the combined dewatering system, consisting of dewatering wells and horizontal collectors.

The total capital costs for the combined dewatering system are estimated at approximately $1,971,000. Annual costs for operation and maintenance of the system are estimated at $275,000.

We believe that dewatering of the upper and lower aquifers could be effectively accomplished using the combined dewatering schemes discussed previously. It must be pointed out, however, that the results of this investigation would not be adequate to properly design the dewatering-recharge system nor to fully describe the environmental impacts associated with the system. It is likely that permits for the withdrawal of water would have to be obtained from the Wisconsin Department of Natural Resources. In all likelihood, providing the necessary assurance for obtaining the withdrawal permits would be a very rigorous excercise.

TABLE 1

PRELIMINARY COST ESTIMATE FOR SLURRY TRENCH

ITEM	QUANTITY	UNIT COST	TOTAL COST
Capital Costs:			
Trench	1,200,000 sq. ft.	7.50	$9,000,000
Interior sumps	6	12,000	72,000
Sump pumps	12 (100 gpm)	2,000	24,000
	6 (250 gpm)	3,000	18,000
Pump installation	6 sumps	15,000	90,000
Discharge piping (8 and 12 inch ID)	6,000 ft.	22/ft	132,000
Electrical system	lump sum	80,000	80,000
			$9,416,000
Operation and Maintenance (annual):			
Labor (12 man-hours/day)	3,066 hr/yr	$15/hr	46,000
Ditch maintenance equipment	500 hr/yr	$50/hr	25,000
Pumping cost (0.3 mgd)	109 mill. gal.	$64/mill gal.	7,000
Equipment replacement	estimate 5 percent of capital for replaceable equipment	--	21,000
Estimated Annual Cost:			$99,000

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TABLE 2

PRELIMINARY COST ESTIMATE FOR CHEMICAL GROUT CURTAIN

ITEM	QUANTITY	UNIT COST	TOTAL COST
Capital Costs:			
Drilling	300,000 feet	$ 51/ft	$1,500,000
Grout curtain	600,000 sq. feet	$ 14/ft	8,400,000
Interior sumps	6	12,000	72,000
Sump pumps	12 (100 gpm)	2,000	24,000
	6 (250 gpm)	3,000	18,000
Pump installation	6 sumps	15,000	90,000
Discharge pipe (8 and 12 inch ID)	6,000 feet	$ 22/ft	132,000
Electrical system	lump sum	80,000	80,000
Estimated Total Capital Cost:			$10,300,000
Operation and Maintenance (annual):			
Labor (1 man @ 12 hrs/day)	3,066 hr/yr	$ 15/hr	46,000
Ditch maintenance equipment	500 hr/yr	$ 50/hr	25,000
Pumping cost (0.3 mgd)	--	--	7,000
Equipment replacement	estimate 5 percent of capital for replaceable equipment	--	21,000
Estimated Annual Cost:			$99,000

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TABLE 3

POSSIBLE PUMPING SCHEDULE FOR DEWATERING THE UPPER AND LOWER AQUIFERS

PERIOD OF PUMPING (days)	NUMBER OF WELLS		INDIVIDUAL PUMPING RATE (gpm)		WELL DISCHARGE (MGD)		TOTAL DISCHARGE
	upper aquifer	lower aquifer	upper aquifer	lower aquifer	upper aquifer	lower aquifer	
0-200	15	4	100	200	2.2	1.2	3.4
200-400	15	4	75	200	1.6	1.2	2.8
400-800[a]	15	4	50	200	1.1[b]	1.2	2.3
800-2000	15	4	25	50	0.5[b]	0.3[b]	1.7

[a] Horizontal collector well system installed.

[b] Horizontal collector system yields an additional 0.6 mgd.

[c] Horizontal collector system yields an additional 0.3 mgd.

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TABLE 4

PRELIMINARY COST ESTIMATE OF COMBINATION DEWATERING SYSTEM

ITEM	QUANTITY	UNIT COST	TOTAL COST
DEWATERING WELL SYSTEM			
Capital Costs:			
Dewatering wells	26	600/ea	156,000
Header pipe and connector pipe between wells and recharge pits	36,000	22/L.F.	792,000
Submersible pumps and pumps (10 standby)	36	3,500/ea	126,000
Recharge pits & ditch	6	20,000/ea	120,000
Drop pipe for wells	26	135/ea	3,500
Electrical	lump sum	60,000	60,000
Estimated Capital Cost:			$1,260,000
HORIZONTAL COLLECTOR SYSTEM			
Capital Costs:			
Drains (150 ft in length)	37,500	6/L.F.	225,000
Mobilization & demobilization	lump sum	10,000	10,000
Header pipe and connector pipe between drains (6 inch PVC)	17,500	12/L.F.	210,000
Sumps (25 yds. concrete)	6	1,200/ea	72,000
Pumps	12 @ 100 gpm	2,000/ea	24,000
Installation	6 @ 250 gpm	15,000/ea	90,000
Electrical system; cables, poles & alarm	lump sum	80,000	80,000
Estimated Capital Cost:			$711,000
COMBINED OPERATION AND MAINTENANCE			
Labor (3 shifts x 1.5 men/shift)	13,140/hr/yr	15/hr	200,000
Ditch maintenance equipment	500/hr/yr	50/hr	25,000
Pumping costs* Q x H (1/11) x 1/eff. x $0.05/KWH	3 cfs (2 MGD)	1.8 KWH / cfs	52,000
Estimated Annual Cost:			$275,000

*Assuming an average dynamic head of 200 feet, an average rate of 2 MGD and an efficiency of 50 percent.

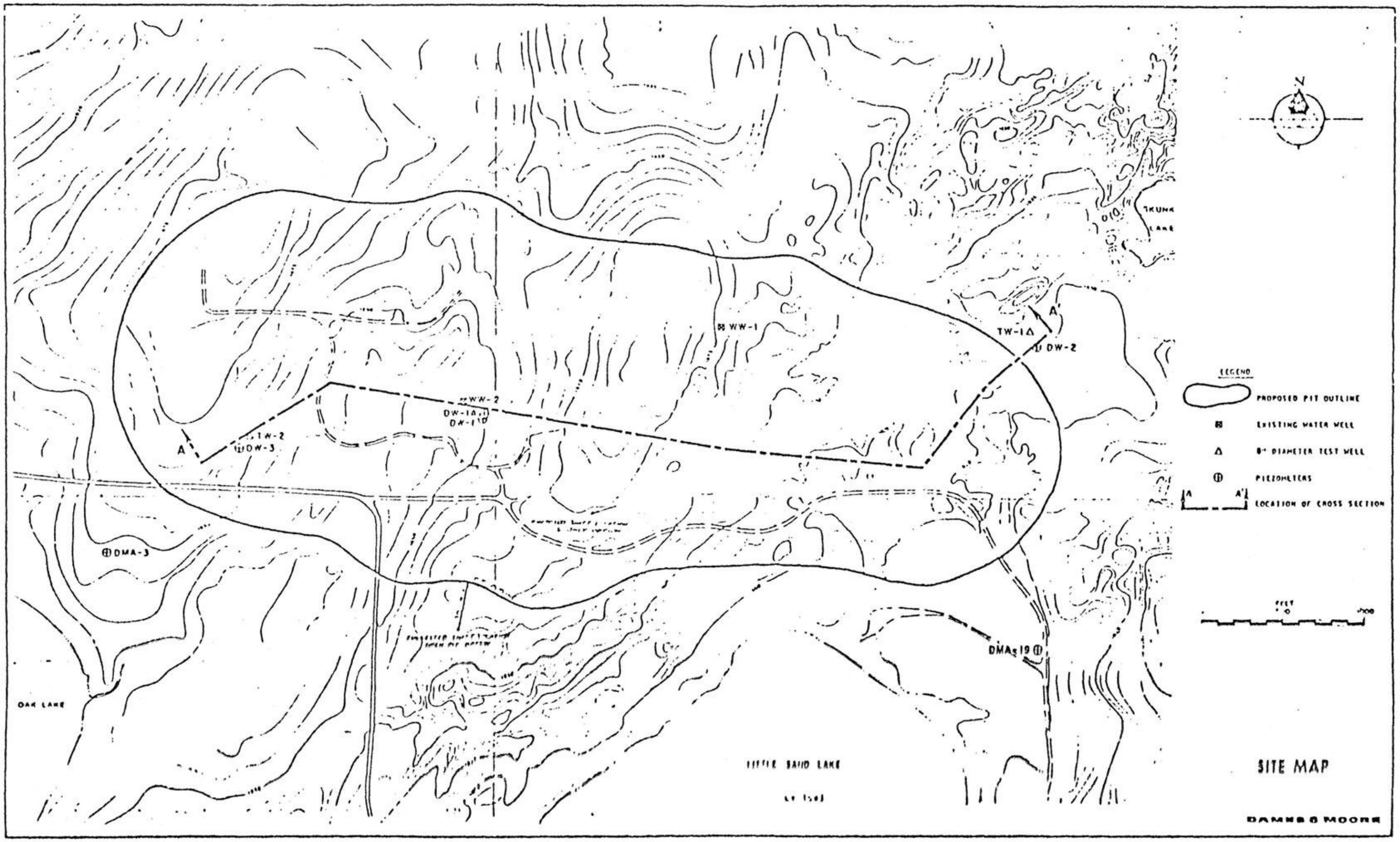

FIGURE 1

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FIGURE 2

GENERALIZED GEOLOGIC LOGS AT PEIZOMETER LOCATIONS

DW-1 - Land Elevation - 1648.32

	Depth-ft	
T	0-18	Brown fine to medium sand, some silt, some gravel
UA	13-63	Brown fine to coarse sand, some gravel, several cobbles, none to trace silt
T	63-139	Red-brown fine to medium sand and silt, little gravel, few sand lenses
LA	139-147	Brown fine to coarse sand, some gravel and cobbles
T	147-212	Red-brown fine sand and silt, some gravel and cobbles, few boulders
	212	Weathered bedrock - black and green

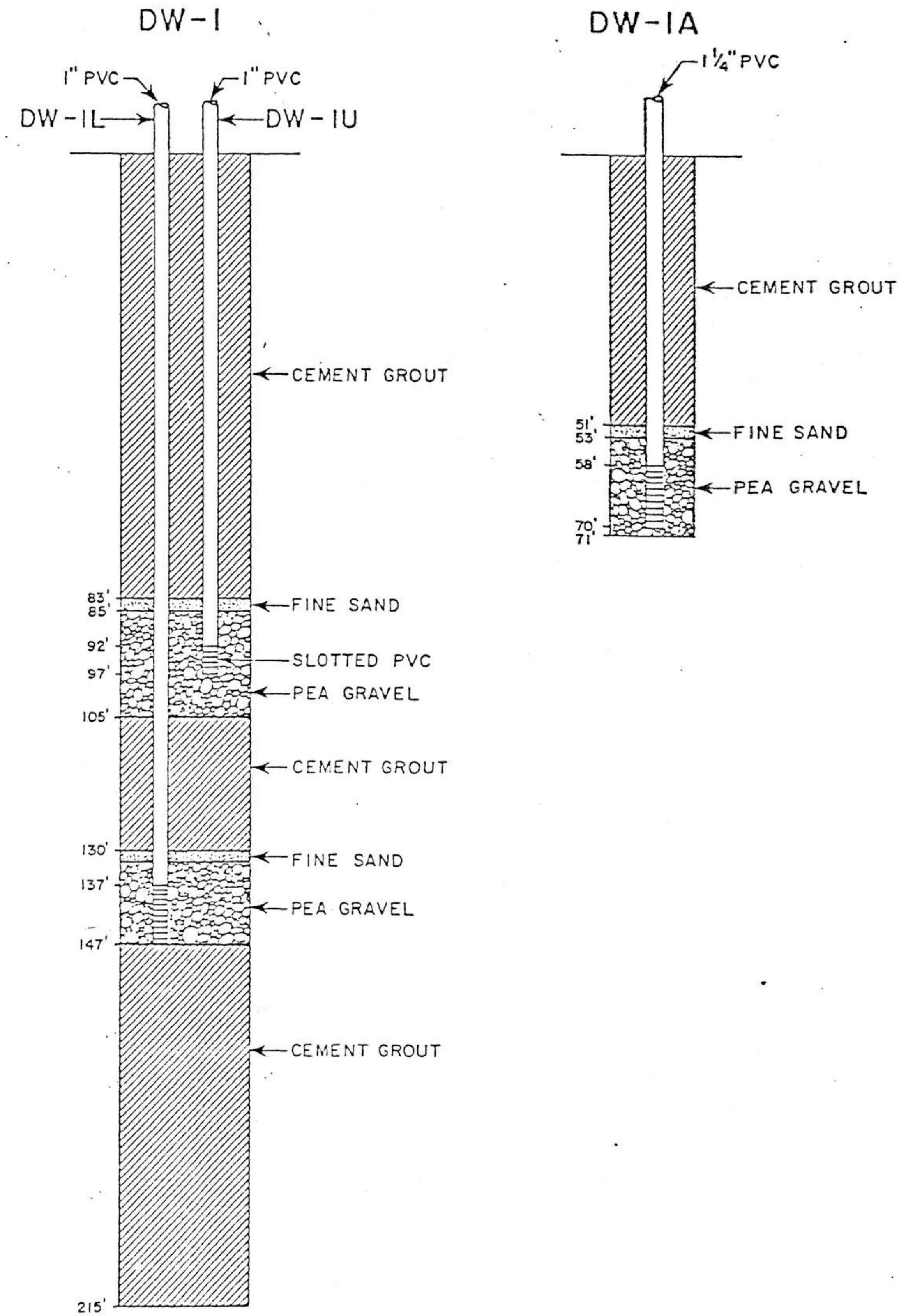
DW-2 - Land Elevation - 1600.74

	Depth-ft	
L	0-2	Light brown fine-medium sand (fill)
L	2-5	Black peat
L	5-7	Yellow-brown and gray fine sand, some silt, mottled
L	7-17	Light gray fine to coarse sand, trace silt, trace gravel
L	17-19	Light gray silt, some fine sand
UA	19-71	Brown and orange-brown fine to coarse sand, some gravel, trace silt, varying amounts of sand and gravel
T	71-84	Red-brown fine to coarse sand and silt, little to some gravel, many weathered pebbles.
LA	84-92	Brown fine to medium sand and coarse gravel, trace to little silt
T	92-95	Red weathered bedrock
	95	Weathered bedrock

DW-3 - Land Elevation - 1657.07

	Depth-ft	
T	0-38	Brown fine sand, gravel, and silt, boulders
UA	38-98	Brown fine to medium sand, some gravel, none to trace silt
T	98-129	Brown fine to coarse sand, some silt, little gravel
LA	129-133	Light brown fine to medium sand, some gravel
T	133-162	Red-brown fine to medium sand, some silt, little gravel
	162-164	Red weathered bedrock
	164	Weathered bedrock

T = Till
UA = Upper Aquifer
LA = Lower Aquifer
L = Lacustrine deposits



PIEZOMETER DETAILS

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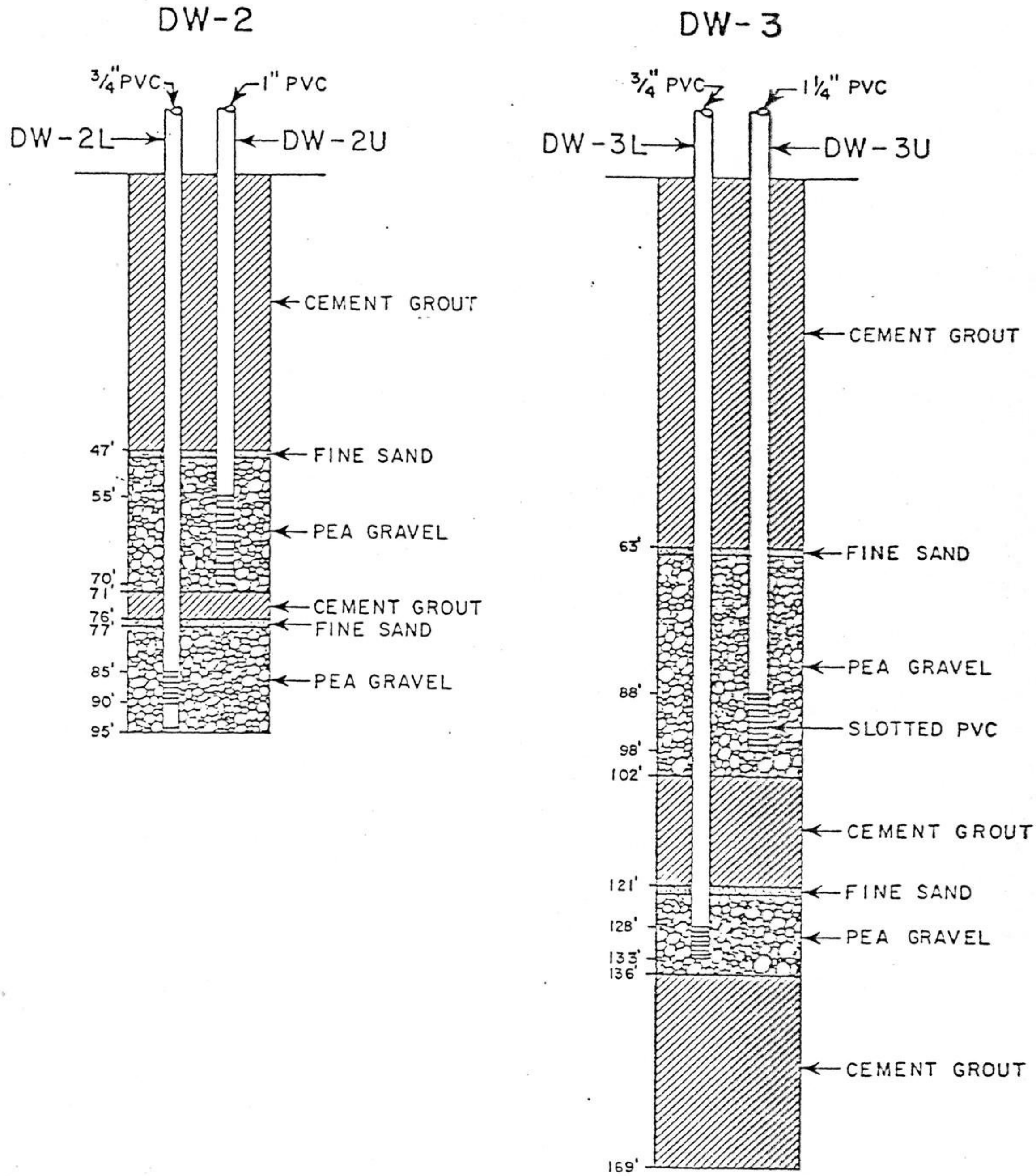
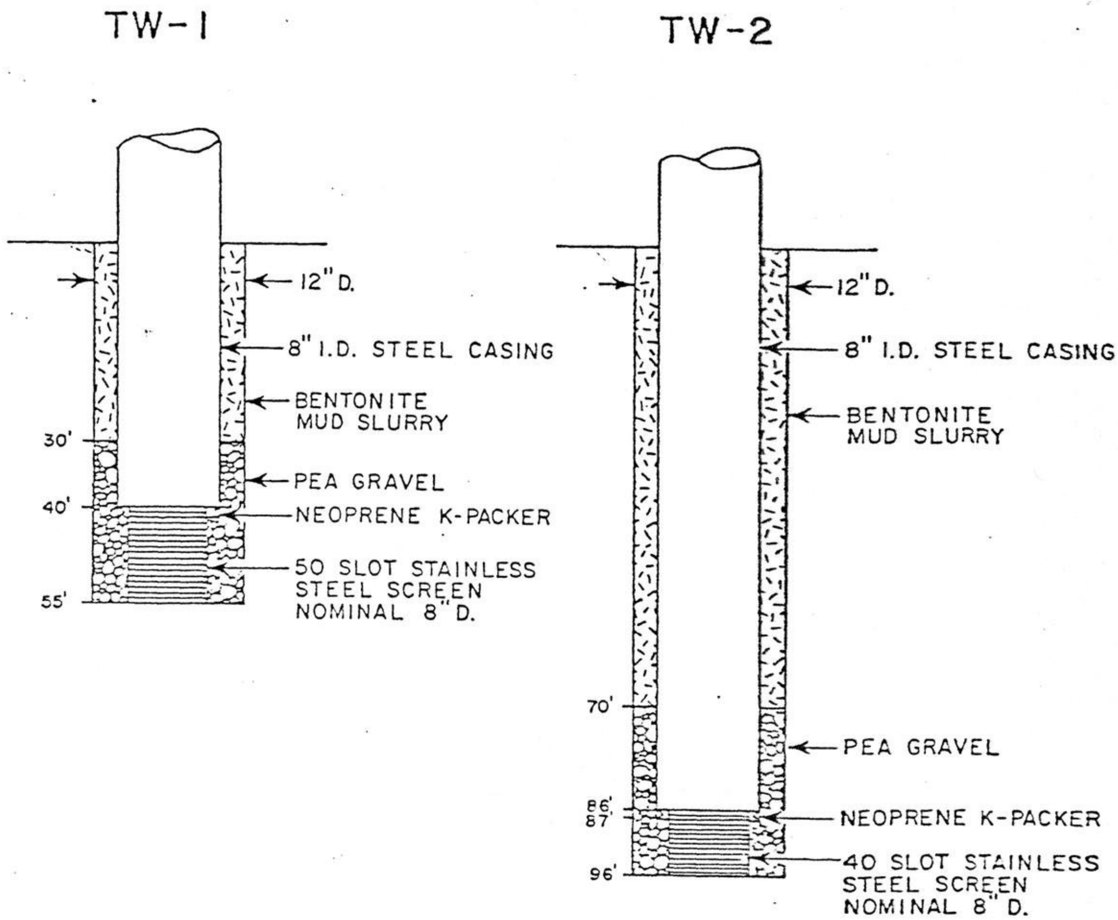


PIEZOMETER DETAILS

DAMES & MOORE



TEST WELL
CONSTRUCTION DETAILS

DAMES & MOORE

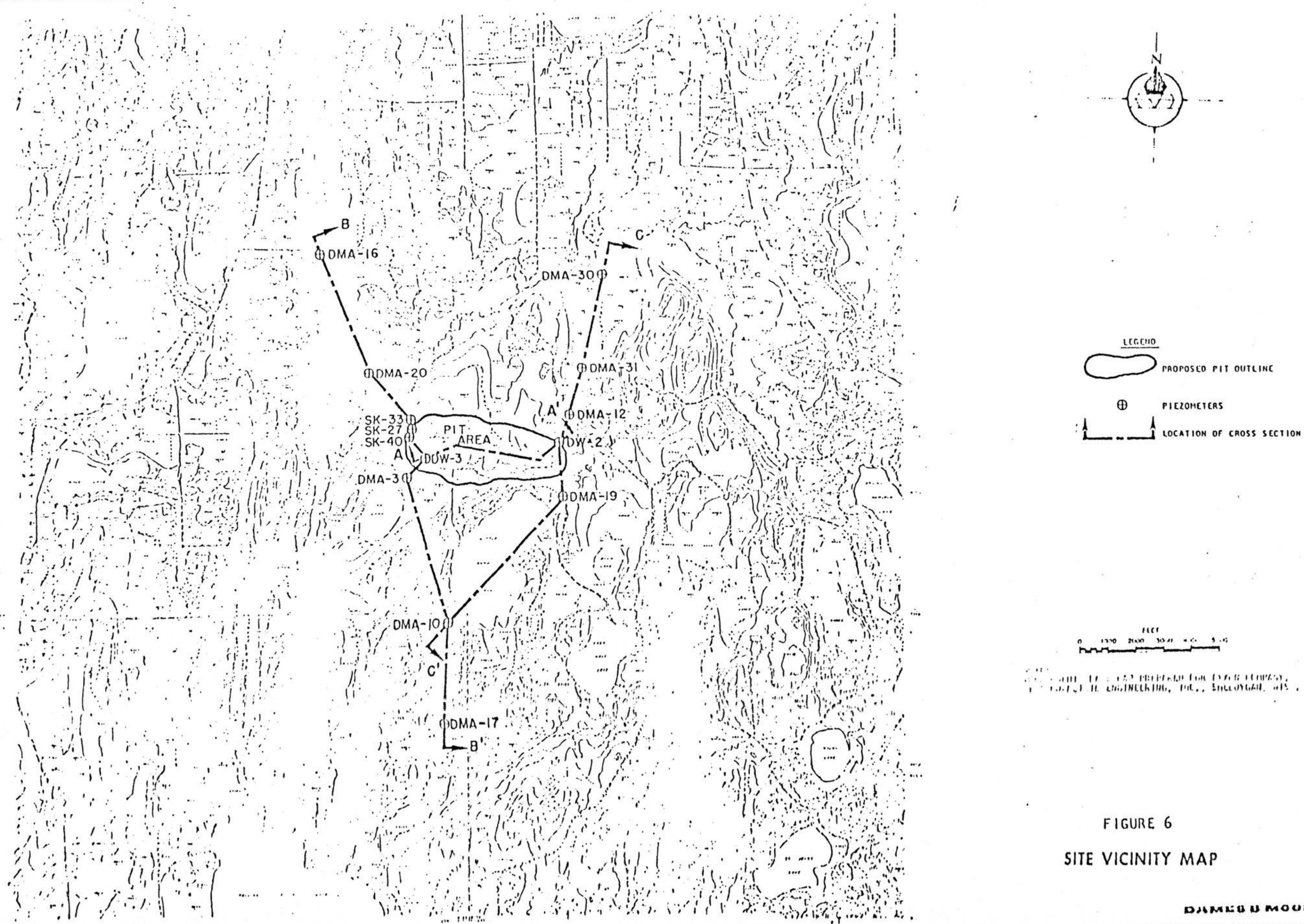


FIGURE 6

SITE VICINITY MAP

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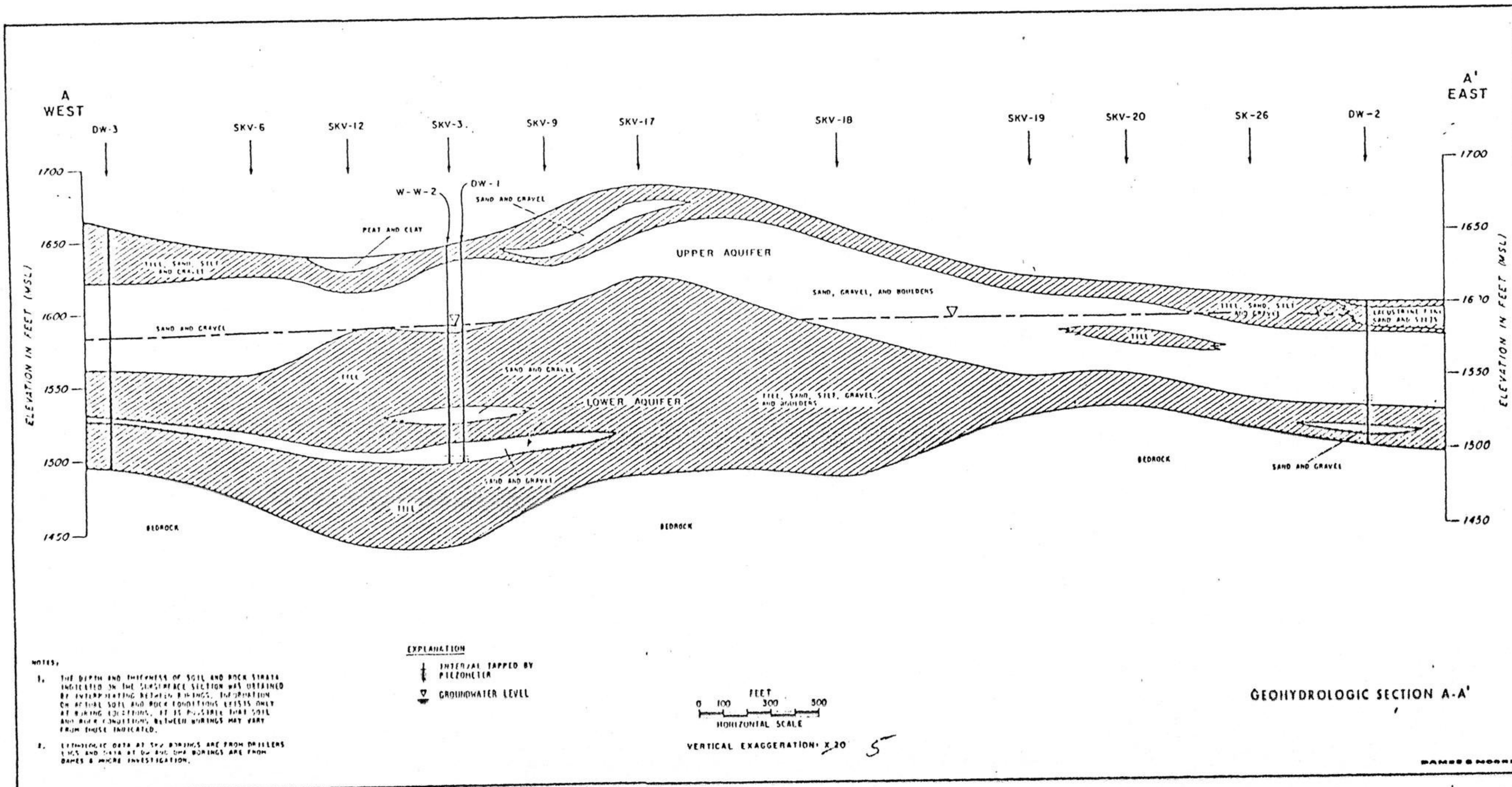
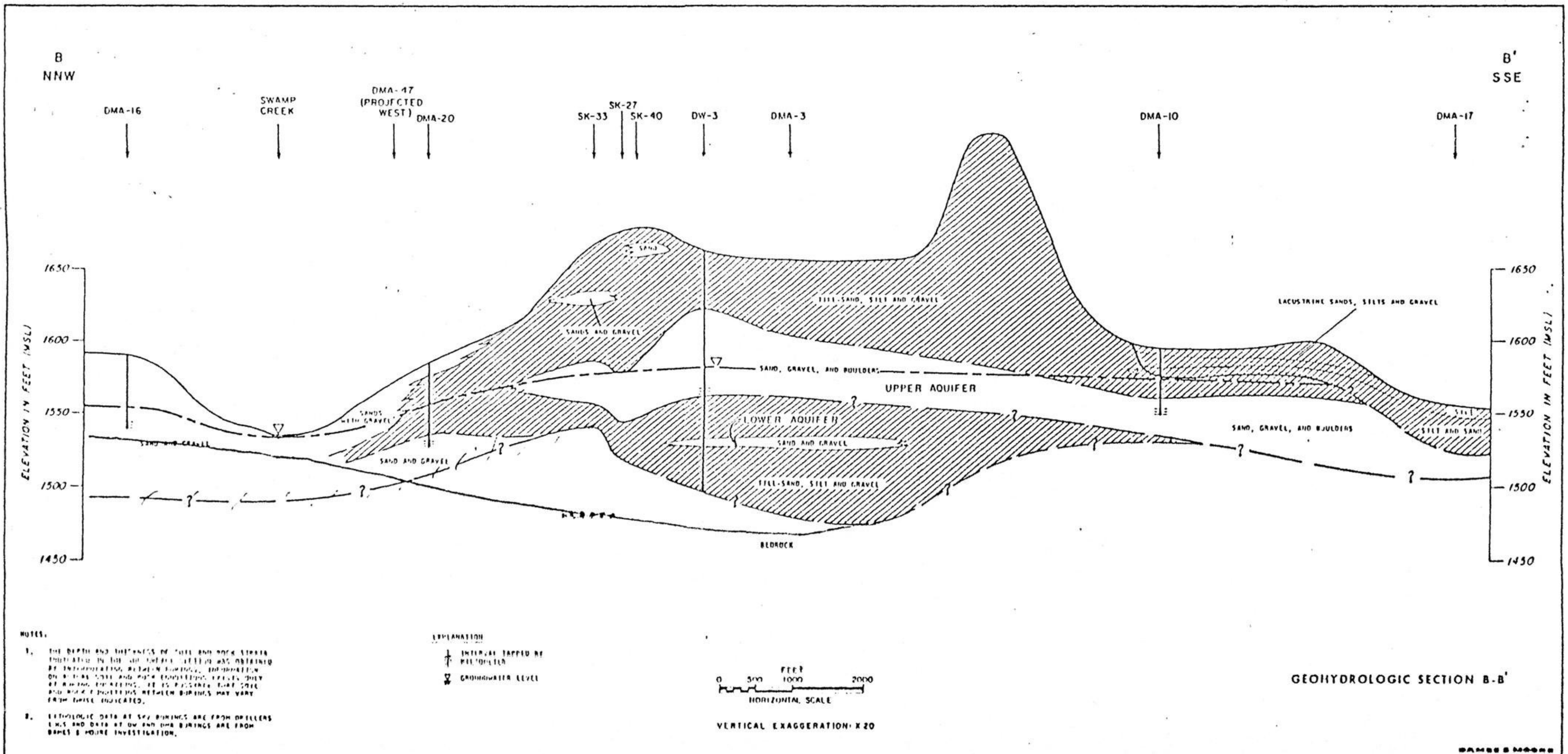
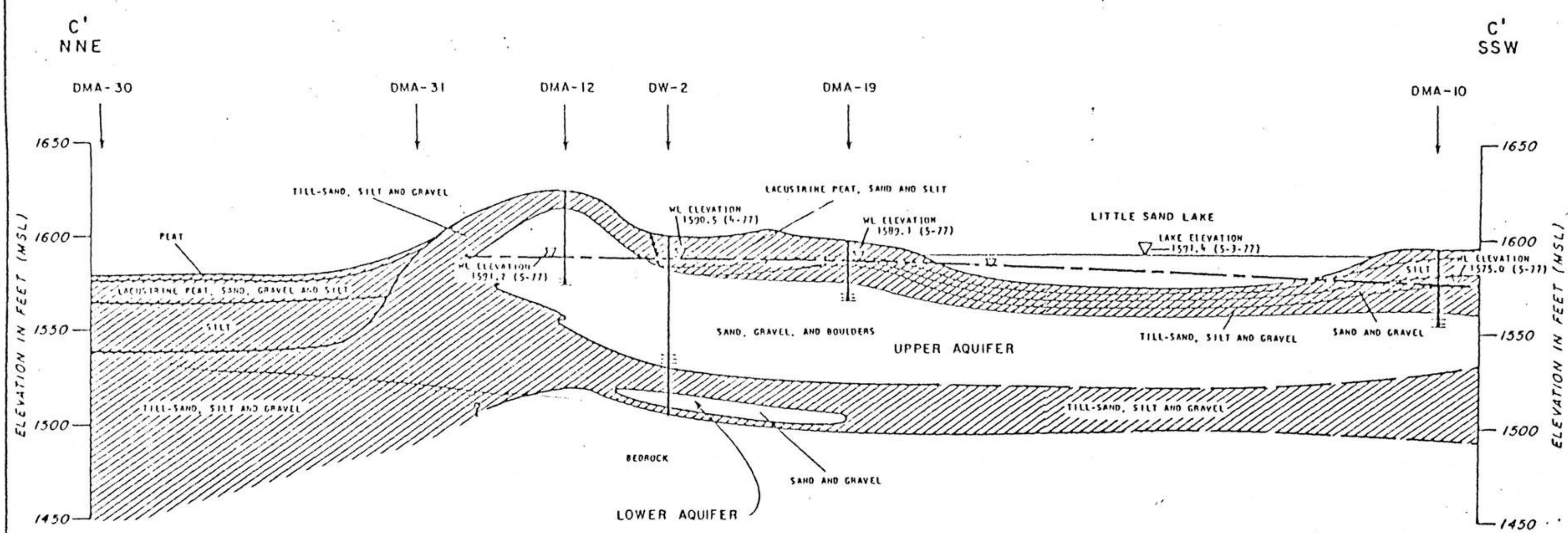

A
WEST
A'
EAST
DW-3
SKV-6
SKV-12
SKV-3
SKV-9
SKV-17
SKV-18
SKV-19
SKV-20
SK-26
DW-2
W-W-2
DW-1
1700
1650
1600
1550
1500
1450
ELEVATION IN FEET (MSL)
SAND AND GRAVEL
PEAT AND CLAY
TILL, SAND, SILT AND GRAVEL
UPPER AQUIFER
SAND, GRAVEL, AND BOULDERS
LACUSTRINE FINE SAND AND SILTS
TILL
LOWER AQUIFER
TILL, SAND, SILT, GRAVEL, AND BOULDERS
BEDROCK
NOTES:
1. THE DEPTH AND THICKNESS OF SOIL AND ROCK STRATA INDICATED ON THE SUBSURFACE SECTION WAS OBTAINED BY INTERPOLATING BETWEEN BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT BORING LOCATIONS. IT IS POSSIBLE THAT SOIL AND ROCK CONDITIONS BETWEEN BORINGS MAY VARY FROM THOSE INDICATED.
2. LITHOLOGIC DATA AT SKV BORINGS ARE FROM DRILLERS LOGS AND DATA AT DW AND DMA BORINGS ARE FROM DAMES & MOORE INVESTIGATION.
EXPLANATION
INTERVAL TAPPED BY PIEZOMETER
GROUNDWATER LEVEL
FEET
0 100 300 500
HORIZONTAL SCALE
VERTICAL EXAGGERATION: X 20 5
GEOHYDROLOGIC SECTION A-A'
DAMES & MOORE

FIGURE 7



B
NNW
B'
SSE
DMA-16
SWAMP CREEK
DMA-47 (PROJECTED WEST)
DMA-20
SK-27
SK-33
SK-40
DW-3
DMA-3
DMA-10
DMA-17
ELEVATION IN FEET (MSL)
1650
1600
1550
1500
1450
SAND
SANDS AND GRAVEL
TILL-SAND, SILT AND GRAVEL
LACUSTRINE SANDS, SILTS AND GRAVEL
SAND, GRAVEL, AND BOULDERS
UPPER AQUIFER
LOWER AQUIFER
SAND AND GRAVEL
SANDS WITH GRAVEL
SILT
SILT AND SAND
BEDROCK
NOTES:
EXPLANATION
GROUNDWATER LEVEL
FEET
0
500
1000
2000
HORIZONTAL SCALE
VERTICAL EXAGGERATION: X 20
GEOHYDROLOGIC SECTION B-B'

FIGURE 8



NOTES:

1. THE DEPTH AND THICKNESS OF SOIL AND ROCK STRATA INDICATED ON THE SUBSURFACE SECTION WAS OBTAINED BY INTERPOLATING BETWEEN BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT BORING LOCATIONS. IT IS POSSIBLE THAT SOIL AND ROCK CONDITIONS BETWEEN BORINGS MAY VARY FROM THOSE INDICATED.

2. LITHOLOGIC DATA AT SKY BORINGS ARE FROM DRILLERS LOGS AND DATA AT DW AND DMA BORINGS ARE FROM DAMES & MOORE INVESTIGATION.

EXPLANATION

INTERVAL TAPPED BY PIEZOMETER

GROUNDWATER LEVEL

FEET
0 1000 2000
HORIZONTAL SCALE

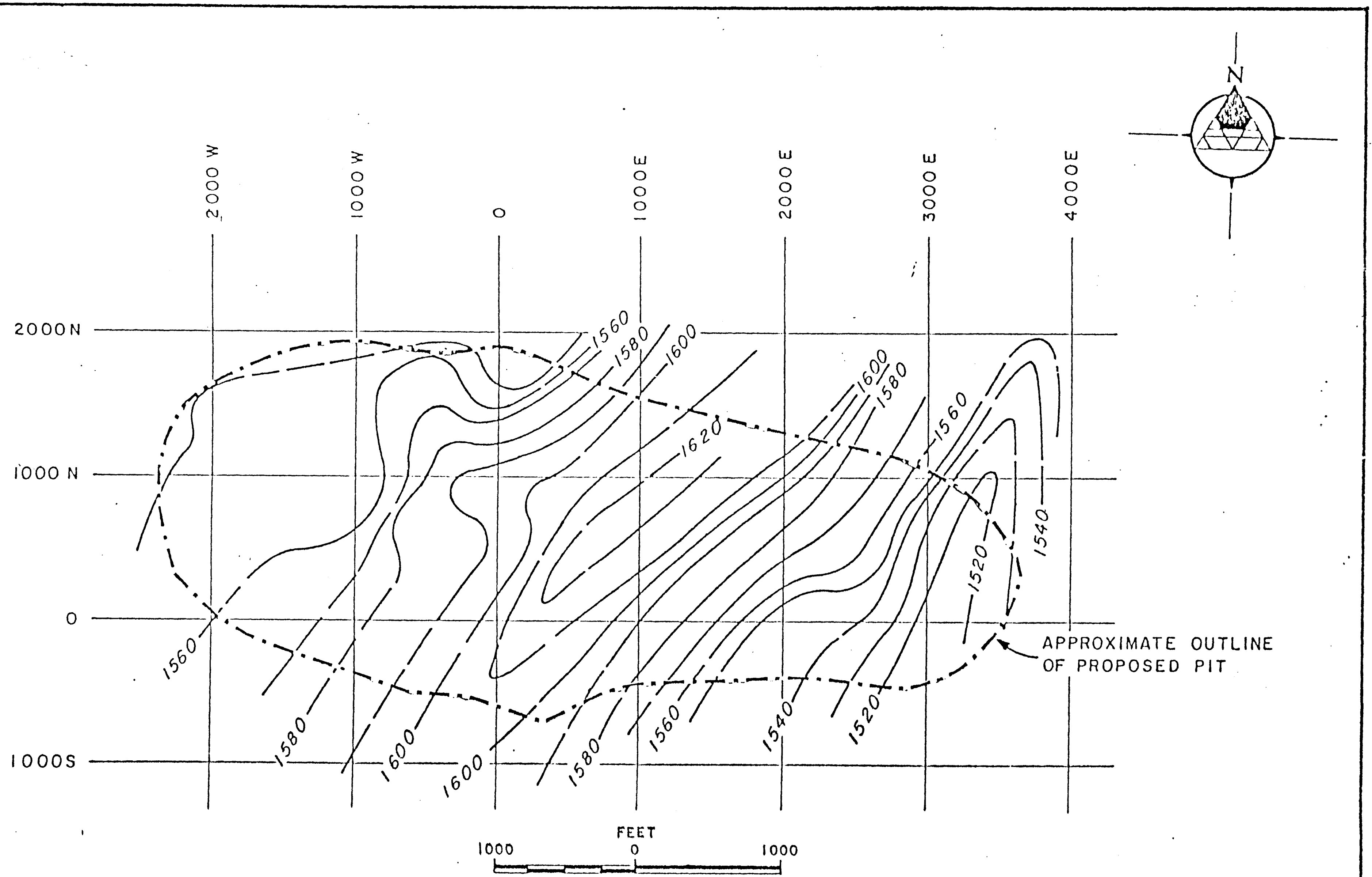
VERTICAL EXAGGERATION: X 20

GEOHYDROLOGIC SECTION C-C'

DAMES & MOORE

FIGURE 9

8837-044-07



CONTOUR MAP SHOWING APPROXIMATE ELEVATION OF BASE OF UPPER AQUIFER

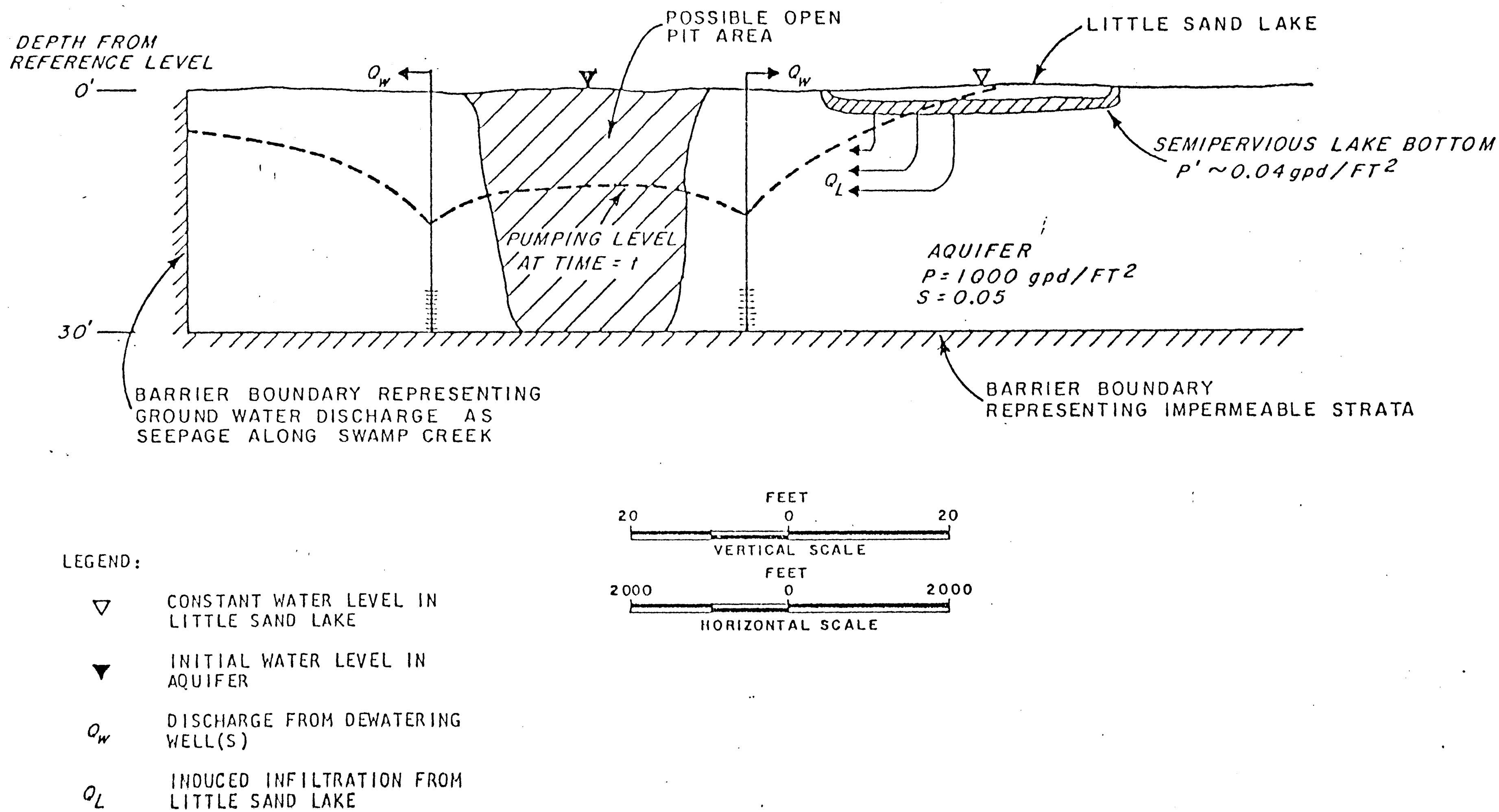
DAMES & MOORE

NOTE:
TOPOGRAPHIC BASE MAP PREPARED FOR EXXON COMPANY,
BY AERO-METRIC ENGINEERING, INC., SHEBOYGAN [illegible],

SCHEMATIC OF COMPUTER MODEL
WITH BOUNDARIES AND WELL CONFIGURATION
FOR DEWATERING

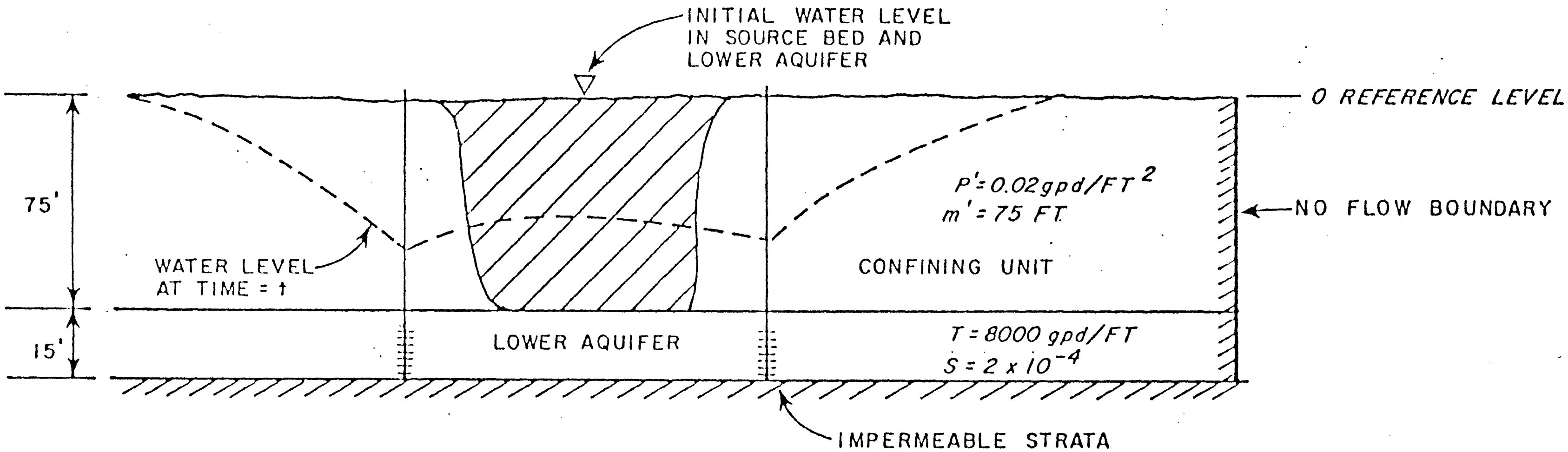
DAMES & MOORE

8837-044-07



NORTH - SOUTH SECTION THROUGH MODEL AQUIFER ALONG COLUMN 11

DAMES & MOORE



EAST - WEST SECTION THROUGH LOWER AQUIFER MODEL

FIGURE 12A

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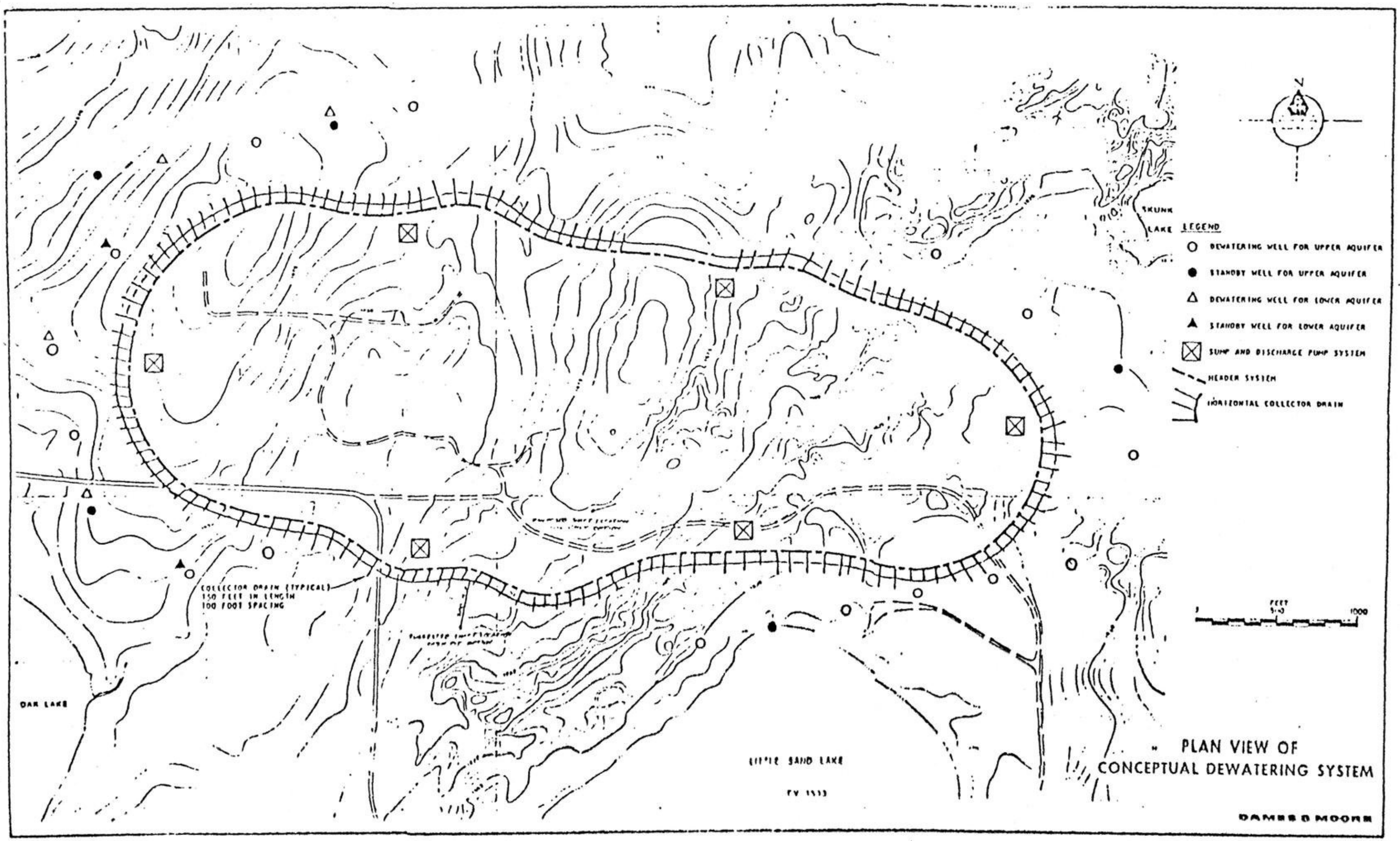

SKUNK LAKE
LEGEND
DEWATERING WELL FOR UPPER AQUIFER
STANDBY WELL FOR UPPER AQUIFER
DEWATERING WELL FOR LOWER AQUIFER
STANDBY WELL FOR LOWER AQUIFER
SUMP AND DISCHARGE PUMP SYSTEM
HEADER SYSTEM
HORIZONTAL COLLECTOR DRAIN
COLLECTOR DRAIN (TYPICAL)
150 FEET IN LENGTH
100 FOOT SPACING
OAK LAKE
LITTLE SAND LAKE
1000
PLAN VIEW OF
CONCEPTUAL DEWATERING SYSTEM
DAMES & MOORE

FIGURE 13

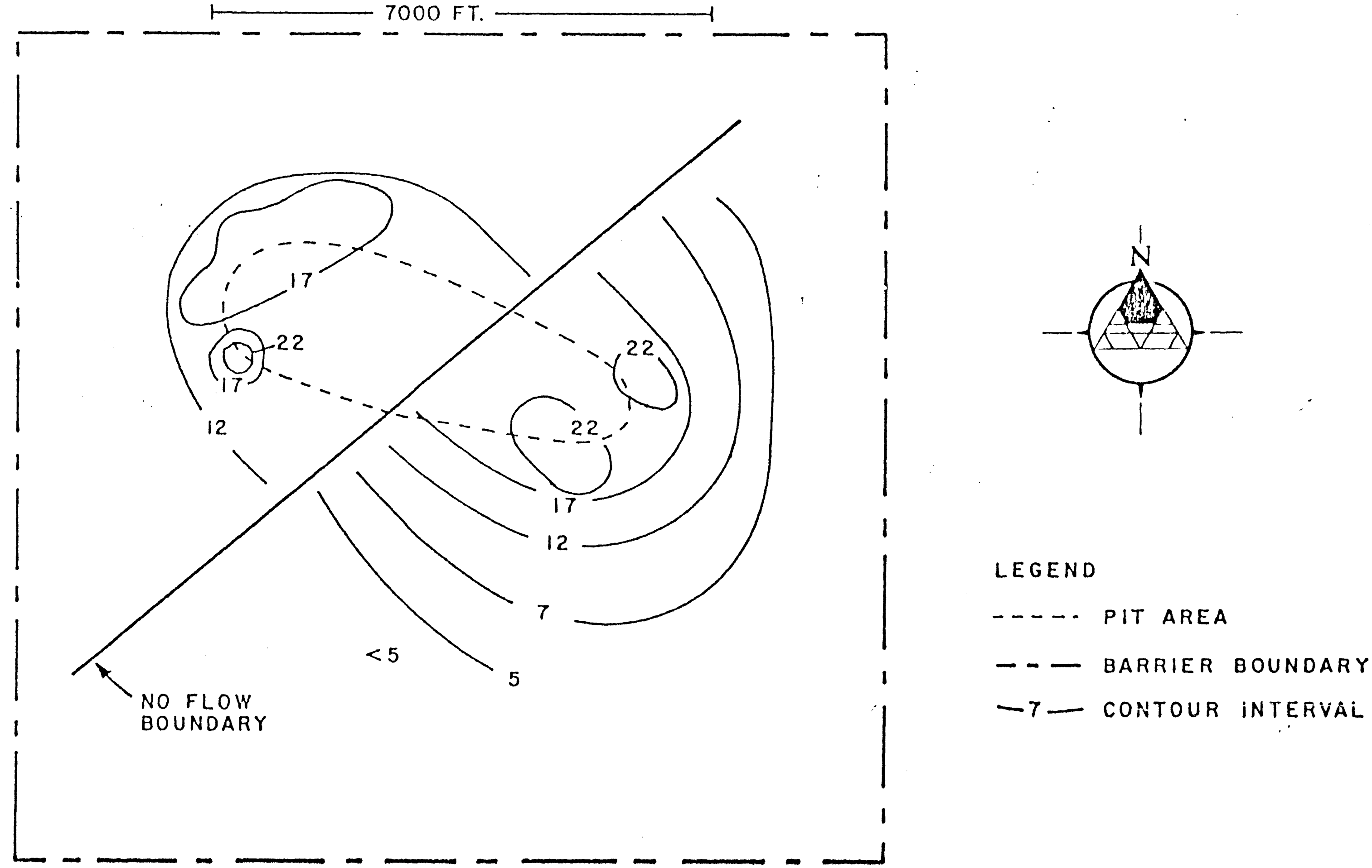


FIGURE 14

CONTOUR MAP SHOWING DRAWDOWN DISTRIBUTION IN THE UPPER AQUIFER AFTER 180 DAYS WITH 12 WELLS PUMPING FROM 100 TO 150 gpm

DAMES & MOORE

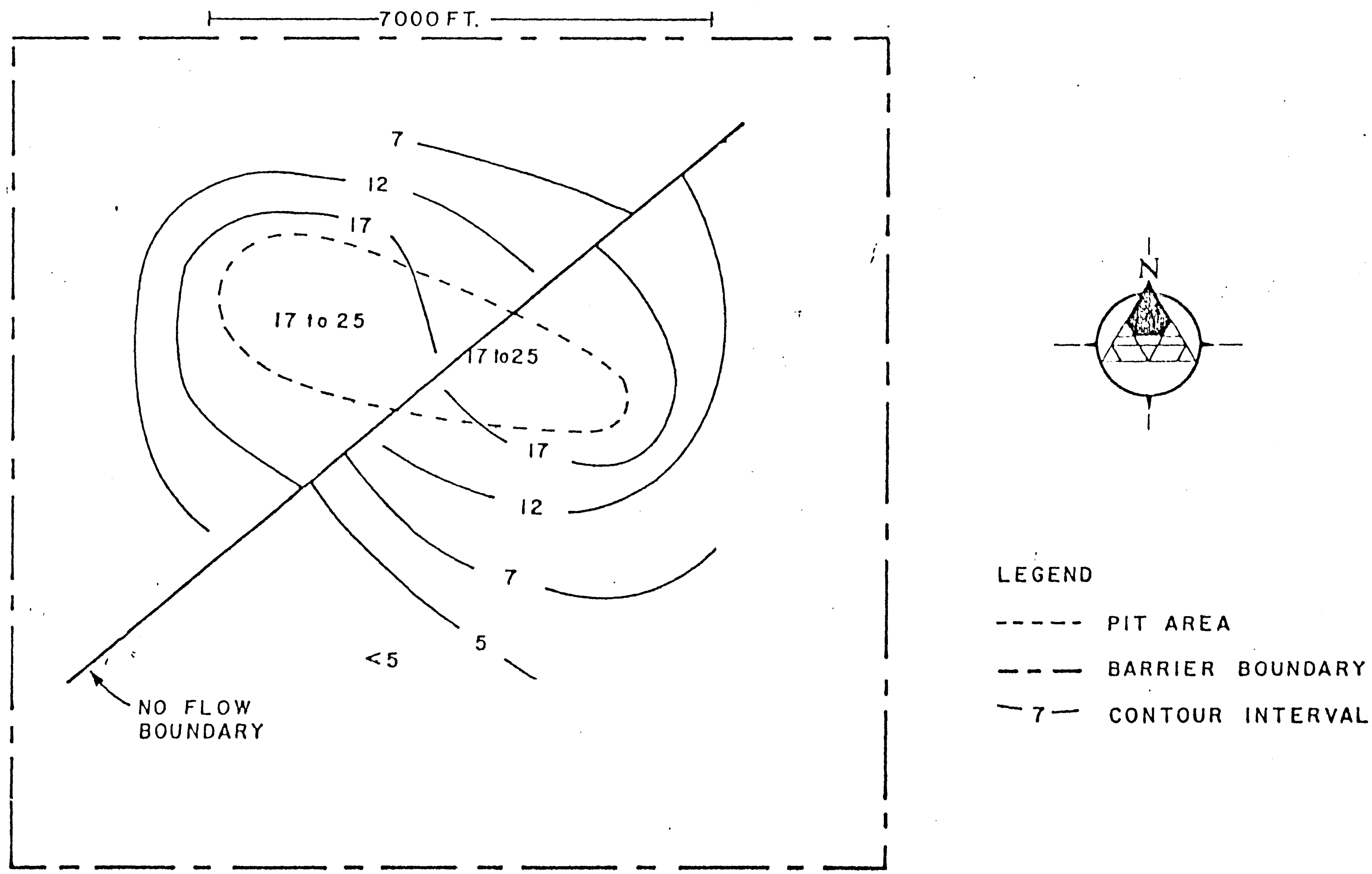
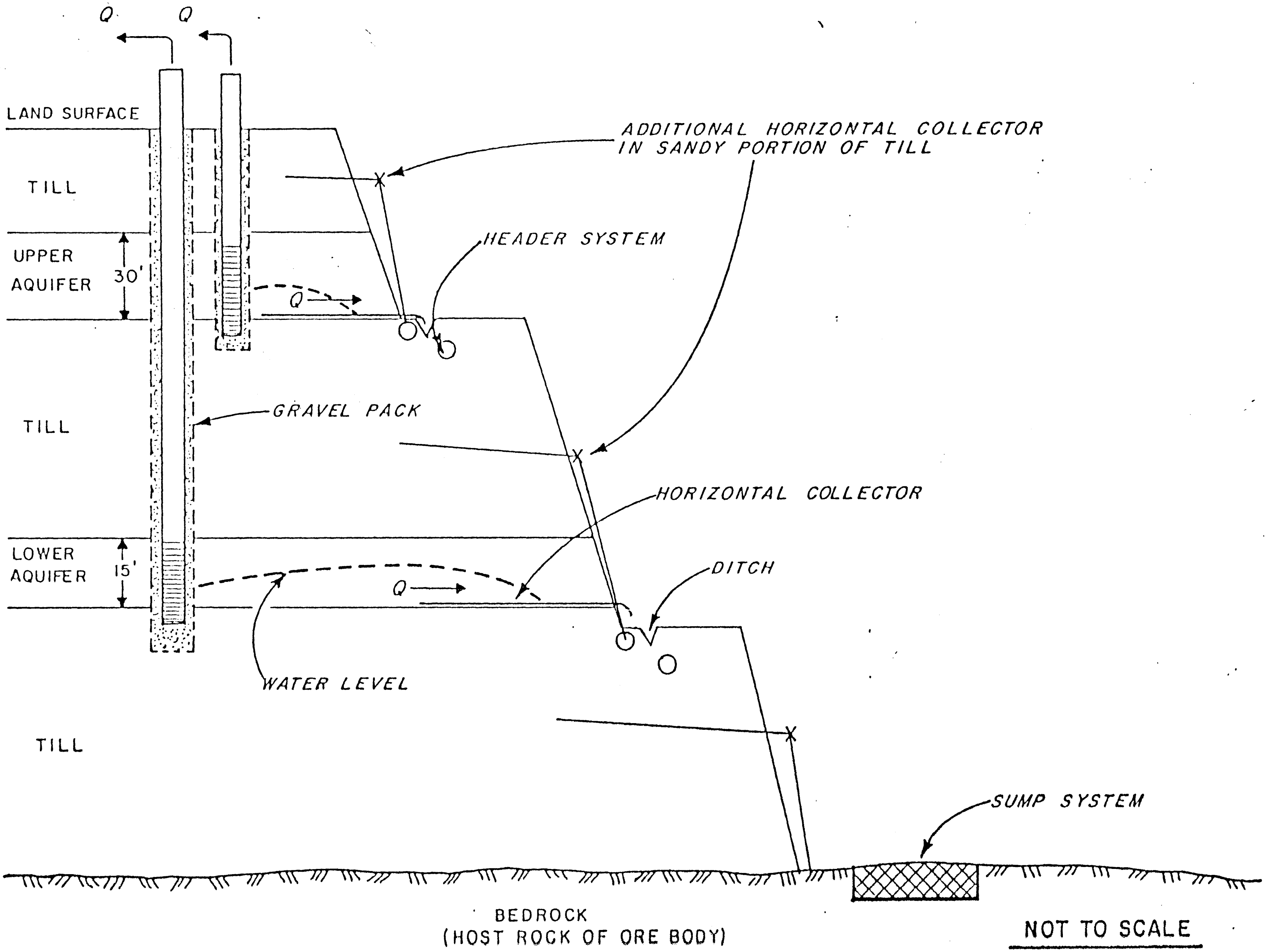


FIGURE 15

CONTOUR MAP SHOWING DRAWNDOWN DISTRIBUTION IN THE UPPER AQUIFER AFTER 600 DAYS WITH 20 WELLS PUMPING AT 50 gpm

DAMES & MOORE



SECTION SHOWING CONCEPTUAL DEWATERING SYSTEM

DAMES & MOORE

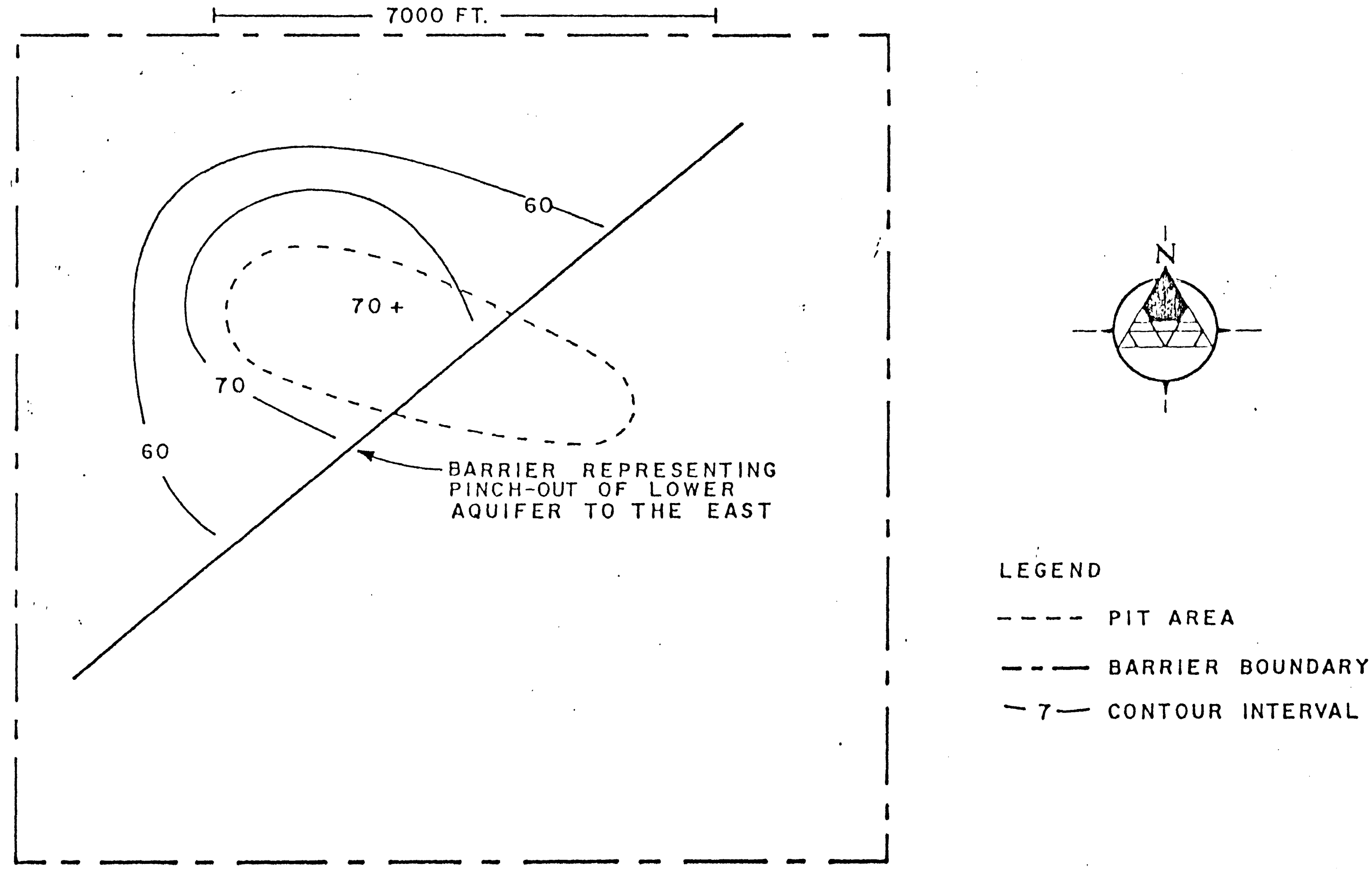


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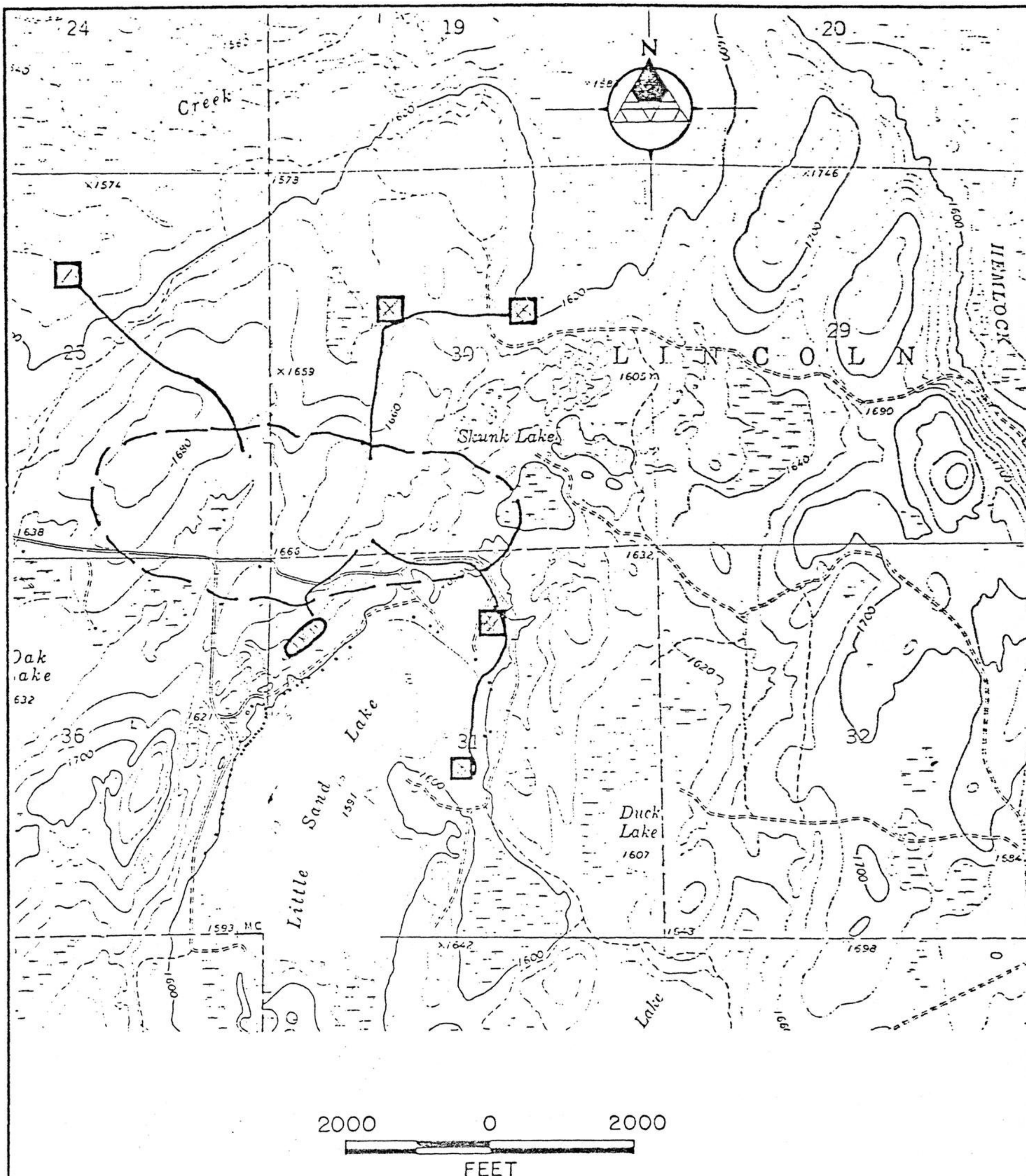
FIGURE 17

CONTOUR MAP SHOWING DRAWDOWN DISTRIBUTION IN THE LOWER AQUIFER AFTER 900 DAYS WITH 4 WELLS PUMPING AT 200 gpm



POSSIBLE OPEN PIT OUTLINE

RECHARGE PIT

CONNECTOR PIPELINES

FIGURE 18
POSSIBLE LOCATIONS
OF RECHARGE PITS

DAMES & MOORE

APPENDIX

Pumping Test Data

PUMPING TEST -1

UPPER AQUIFER

Pumping Well - Test Well -1, Pumping rate 80 gpm

Observation Well - DW-2U

Pumping Period - 1645 - 1 April 77 to 1715 - 2 April 77

Recovery Period - 1715 - 2 April 77 to 1508 - 3 April 77

Record of Water Well -3

Time From Start (min)	Depth to Water Level (feet)	Time From Start (min)	Depth to Water Level (feet)
_	12.21	285	34.70
0	12.22	315	34.70
1	33.20	380	34.90
3	34.20	453	34.70
5	34.20	585	35.30
7	34.40	626	35.00
10	34.40	700	35.10
15	34.60	740	35.40
20	34.60	910	35.00
25	34.60	965	35.00
30	34.60	1035	34.80
35	34.60	1095	35.10
40	34.60	1155	34.60
50	34.60	1215	35.00
60	34.60	1275	35.50
80	34.70	1335	35.50
105	34.70	1415	35.50
135	34.70	1455	35.30
165	34.70	1478	35.30
195	34.80	-Stop Pump-	
225	34.70	-Start Recovery-	
255	34.70	0	35.30

Time From Start (min)	Depth to Water Level (feet)	Time From Start (min)	Depth to Water Level (feet)
½	26.70	30	12.60
1	19.20	38	12.60
2	14.20	65	12.60
3	13.20	90	12.60
4	12.80	170	12.56
7	12.80	320	12.52
10	12.70	1005	12.36
15	12.70	1137	12.35
23	12.60	1300	12.35

PUMPING TEST -1

UPPER AQUIFER

Pumping Well - Test Well -1, Pumping rate 80 gpm

Observation Well - DW-2U

Pumping Period - 1645 - 1 April 77 to 1715 - 2 April 77

Recovery Period - 1715 - 2 April 77 to 1508 - 3 April 77

Record for DW-2U

Time From Start (min)	Depth to Water Level (feet)	Time From Start (min)	Depth to Water Level (feet)
-	11.68	135	12.72
0	11.68	165	12.74
½	11.91	195	12.74
1	12.09	225	12.74
2	12.16	255	12.73
3	12.24	285	12.69
4	12.28	360	12.73
5	12.35	405	12.71
6½	12.39	540	12.73
8	12.41	600	12.72
10	12.44	670	12.73
12	12.53	705	12.74
15	12.53	795	12.77
20	12.58	930	12.80
25	12.58	965	12.82
30	12.60	1035	12.84
35	12.63	1095	12.91
40	12.61	1155	12.78
50	12.66	1215	12.81
60	12.67	1275	12.82
75	12.67	1335	12.84
90	12.69	1405	12.87
115	12.71	1440	12.86

Time From Start (min)	Depth to Water Level (feet)	Time From Start (min)	Depth to Water Level (feet)
1478	12.86	23	11.92
-Stop Pump-		25	11.91
-Start Recovery-		30	11.90
0	12.86	38	11.87
½	12.85	56	11.84
1	12.75	75	11.84
2	12.54	80	11.86
3	12.38	90	11.86
4	12.29	120	11.85
5	12.21	180	11.83
7	12.12	325	11.81
8	12.09	1010	11.76
10	12.06	1142	11.76
12	12.02	1305	11.75
15	11.98		

PUMPING TEST -2

LOWER AQUIFER

Pumping Well - Water Well -2, Pumping rate - 48 gpm

Observation Well - DW-1L

Pumping Period - 1500 - 11 April 77 to 1510 - 13 April 77

Recovery Period - 1510 - 13 April 77 to 1530 - 14 April 77

Record for Water Well-2

Time From Start (min)	Depth to Water Level (feet)	Time From Start (min)	Depth to Water Level (feet)
--	66.28	1508	75.28
0	66.31	1688	75.77
45 sec	70.61	1998	75.49
2	71.96	2500	75.87
3	72.31	2730	75.83
4	72.60	2800	75.50
10	73.64	2880	75.97
12½	73.76	-Stop Pump-	
24	74.02	-Start Recovery-	
38	74.12	0	75.97
55	74.59	½	71.62
61	74.69	1	70.40
73	74.83	2	69.82
90	74.84	2½	69.52
116	74.94	3	69.28
162	75.05	4	69.07
193	75.13	5	68.91
233	75.16	6	68.75
334	75.76	7	68.54
510	75.84	8	68.32
784	75.82	9	68.30
1073	75.80	10	68.27
1368	75.80	11	68.15

Time From Start (min)	Depth to Water Level (feet)
12	68.07
14	67.93
16	67.83
18	67.75
20	67.67
22	67.59
24	67.54
26	67.50
28	67.45
30	67.41
32	67.36
34	67.33
36	67.29
38	67.26
40	67.23
42	67.19
44	67.16
48	67.13
52	67.09
60	67.01
70	66.95
80	66.90
90	66.85
101	66.79
111	66.76
125	66.73
135	66.70
152	66.67

Time From Start (min)	Depth to Water Level (feet)
160	66.66
170	66.65
180	66.63
191	66.62
201	66.61
220	66.60
250	66.58
280	66.56
313	66.56
340	66.55
378	66.55
425	66.53
470	66.52
595	66.47
715	66.43
795	66.43
1045	66.44
1166	66.43
1432	66.34
1462	66.33

PUMPING TEST -2
LOWER AQUIFER

Pumping Well - Water Well -2, Pumping rate - 48 gpm

Observation Well - DW-1L

Pumping Period - 1500 - 11 April 77 to 1510 - 13 April 77

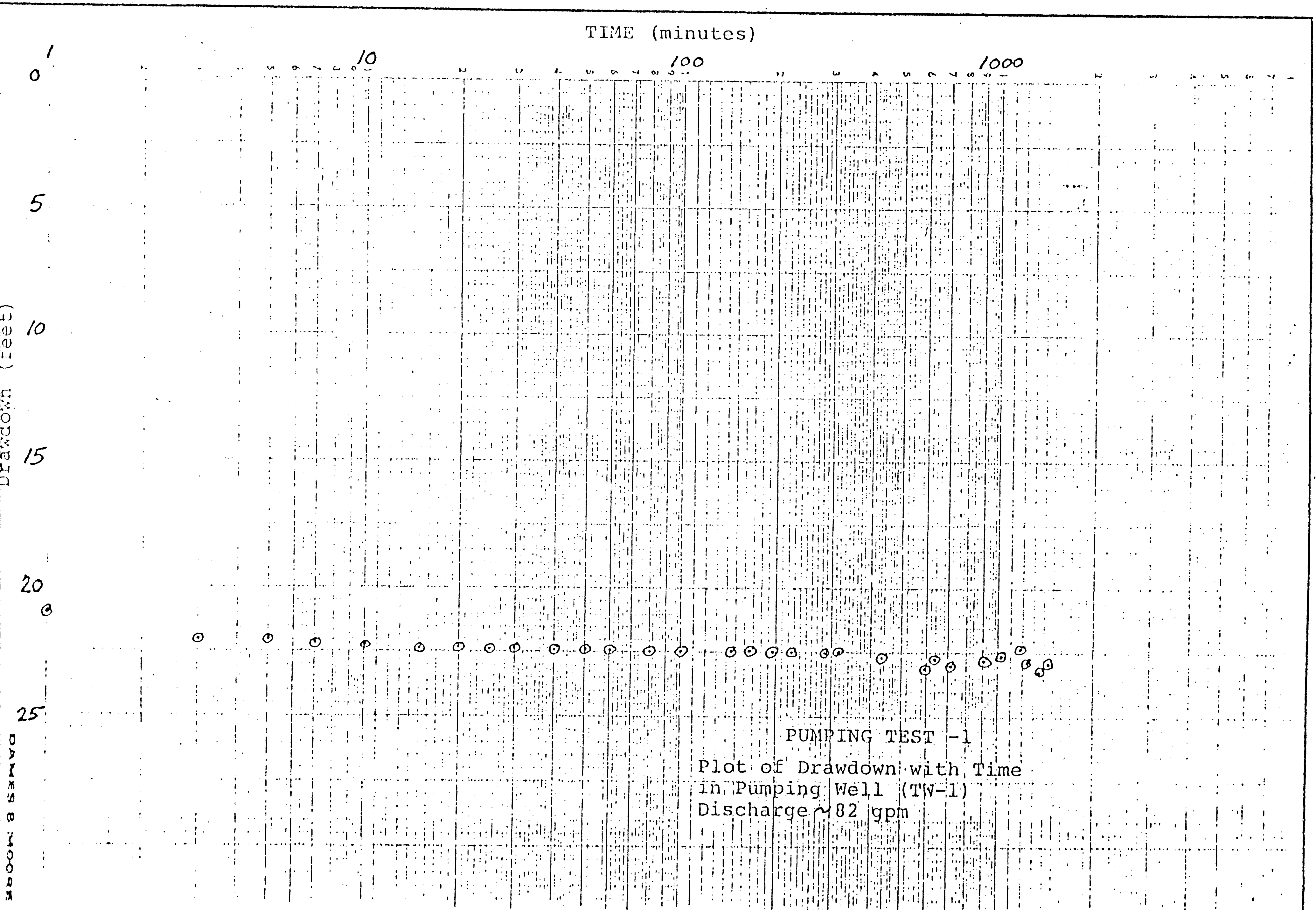
Recovery Period - 1510 - 13 April 77 to 1530 - 14 April 77

Record for DW-1L

Time From Start (min)	Depth to Water Level (feet)	Time From Start (min)	Depth to Water Level (feet)
		775	73.20
--	69.85	1058	73.20
0	69.85	1210	73.18
6	70.40	1372	73.14
7	70.55	1390	73.08
8	70.70	1490	73.13
15	71.60	1695	73.26
21	71.96	1985	73.18
30	72.23	2495	73.32
35½	72.24	2670	73.29
42½	72.34	2770	73.28
49	72.41	2885	73.28
53	72.48	-Stop Pump-	
64	72.55	-Start Recovery-	
77	72.65	0	73.28
99	72.76	1	73.25
107	72.80	1½	73.20
118	72.81	2	73.10
150	72.89	2½	73.00
180	72.96	3	72.90
235	73.03	5	72.58
337	73.11	6	72.38
500	73.20	7	72.26

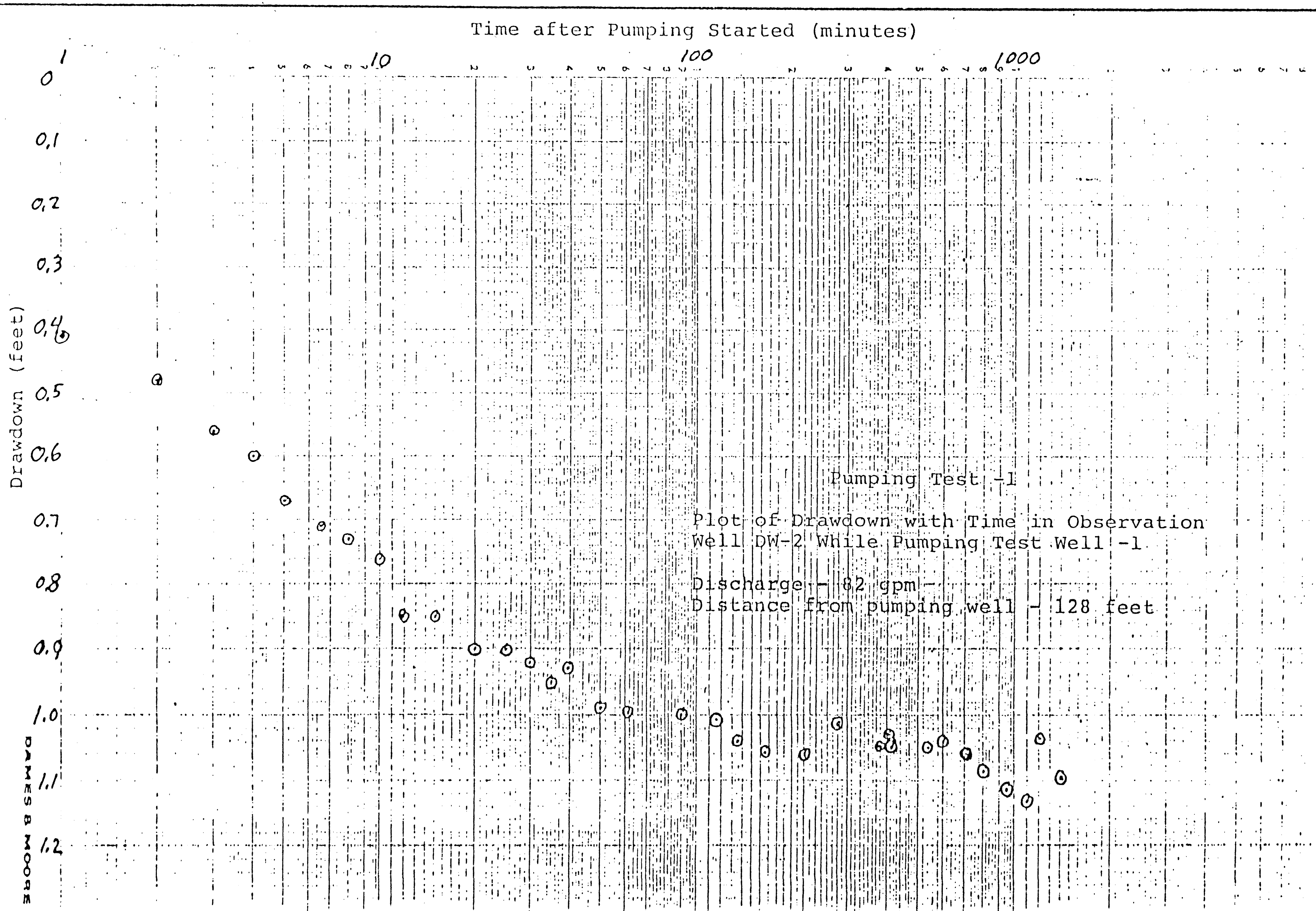
Time From Start (min)	Depth to Water Level (feet)
8	72.14
9	72.03
10	71.96
12	71.78
14	71.63
16	71.50
18	71.44
21	71.28
23	71.21
25	71.14
28	71.11
30	71.04
32	70.99
35	70.90
37	70.87
40	70.81
44	70.78
49	70.71
50	70.69
57	70.62
60	70.59
65	70.55
70	70.52
76	70.48
80	70.48
90	70.41
111	70.33

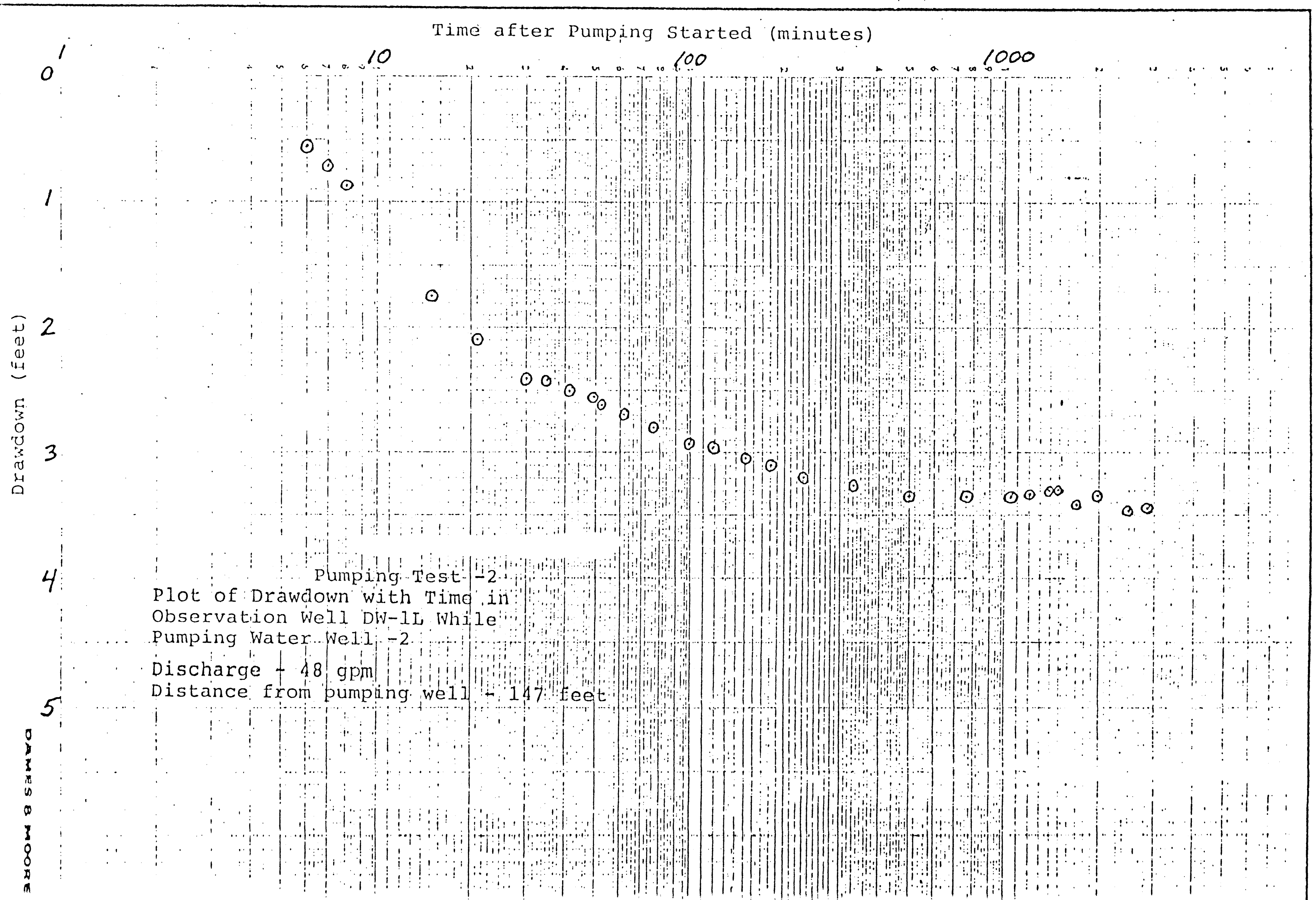
Time From Start (min)	Depth to Water Level (feet)
132	70.27
151	70.26
184	70.22
225	70.17
252	70.15
283	70.14
315	70.11
342	70.10
380	70.10
430	70.10
475	70.10
598	70.08
717	70.04
798	70.05
1047	70.12
1290	70.08
1435	70.04
1460	69.98



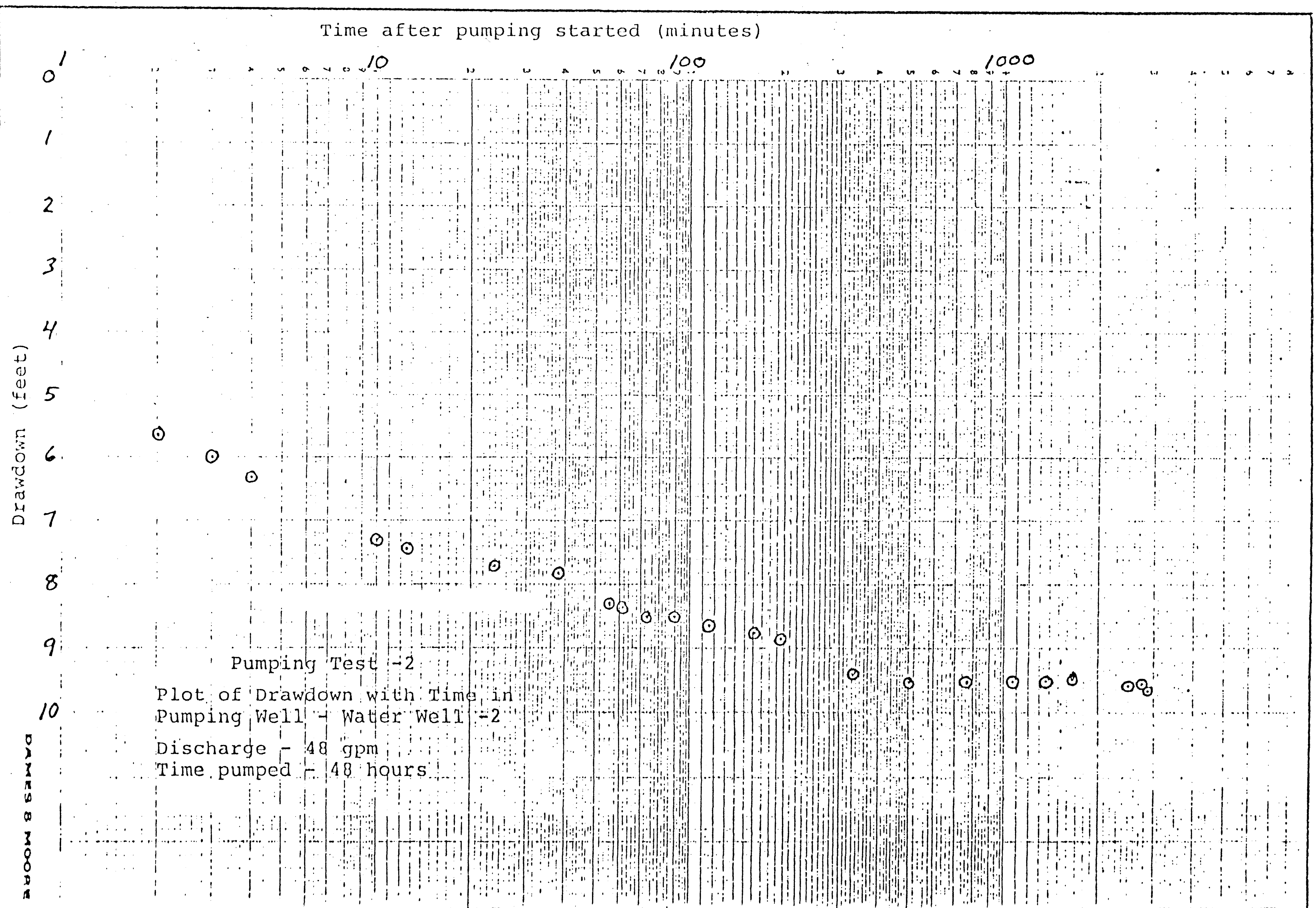
PUMPING TEST -1

Plot of Drawdown with Time
in Pumping Well (TW-1)
Discharge ~82 gpm


Time after Pumping Started (minutes)
1
10
100
1000
Drawdown (feet)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
Pumping Test -1
Plot of Drawdown with Time in Observation Well DW-2 While Pumping Test Well -1
Discharge - 82 gpm -
Distance from pumping well - 128 feet
DAMES & MOORE



Time after Pumping Started (minutes)
10
100
1000
Drawdown (feet)
0
1
2
3
4
5
Pumping Test -2
Plot of Drawdown with Time in
Observation Well DW-1L While
Pumping Water Well -2
Discharge - 48 gpm
Distance from pumping well - 147 feet
DAMES & MOORE


Time after pumping started (minutes)
1
10
100
1000
0
1
2
3
4
5
6
7
8
9
10
Drawdown (feet)
Pumping Test -2
Plot of Drawdown with Time in
Pumping Well - Water Well -2
Discharge - 48 gpm
Time pumped - 48 hours
DAMES & MOORE