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The bulk hydraulic conductivity and storativity of Wisconsin age unoxidized till in Central Iowa

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by

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INTRODUCTION AND OBJECTIVES

Much of Iowa is covered with multiple layers of glacial material from the four most recent glacial periods of the Pleistocene epoch (Anderson, 1983). Glaciers have deposited poorly sorted tills interspersed with sorted and stratified glacial outwash materials. Some sorting occurred where meltwaters deposited sediment, resulting in sand and gravel lenses. The mass of ice that overlaid the till would determine the amount of compression of the deposits. The hydraulic properties of glacial tills are different as a result of the depositional environment and the mineralology of the parent material. Since deposition, weathering processes and vegetation have altered the uppermost layer of the till producing the oxidized layer. Below this oxidized till lies a very dense unoxidized till.

Structural features, such as fractures, and depositional features such as sand inclusions in tills alter the hydraulic properties, enhancing the capability of groundwater flow which may increase the potential for contaminant migration. The variability of flow regime in glacial tills has implications for environmental assessment, waste disposal, wetland management, and agricultural practices. Landfills releasing leachate, underground leaking storage tanks releasing hydrocarbons, chemical spills, and agricultural fertilizers and pesticides are common problems that affect groundwater quality. Many of these problems occur in the dense unoxidized till where little field research has been done to show bulk hydraulic conductivity and storativity.

The selection of suitable sites for waste disposal facilities is based upon hydrogeological studies. Hydraulic conductivity is the parameter describing a material's ability to transmit water and is a function of the porous medium and the fluid flowing through it. Hydraulic conductivity values based on laboratory tests may severely underestimate the contaminant migration rates (Goodall and Quigley, 1977). More detailed information on the spatial variability of hydraulic conductivity will be useful in determining groundwater flow patterns affecting groundwater monitoring, contaminant migration, predicting rates and direction of movement, and determining the most suitable remedial action. With the multitude of new regulations affecting groundwater remediation, field measurements of hydraulic conductivity will become increasingly important in order to more accurately determine rates of contaminant migration.

The objectives of this research are to obtain reliable in-situ data for the determination of hydrological characteristics of the Wisconsin aged unoxidized glacial till and to determine if a constant-head pumping test is a suitable method for finding the bulk hydraulic conductivity and storativity of the till. This will be accomplished by comparing results from the pumping tests and bail tests with previous studies on the site and with studies from similar North American till research. The constant-head pumping test was selected due to the extremely low pumping rates possible in unoxidized till. Bail tests were performed for comparison of values with the pumping test results.

LITERATURE REVIEW

Very little work has been done with in-situ hydraulic testing of unoxidized glacial tills to determine hydraulic conductivity and storativity. Tills are generally believed to have low hydraulic conductivities which retard the vertical movement of water, although fractures produce secondary permeabilities which are much higher than the till matrix (Hendry, 1988). Grisak and Cherry (1975) believe the most likely cause of fracturing of till is crustal extension that has occurred during the post-glacial uplift. Most hydological research on glacial tills has consisted of laboratory permeameter tests which tend to produce hydraulic conductivities 10 to 1000 times lower than that of field measurements (Herzog et al., 1989). Keller, Van Der Kamp, and Cherry (1985) found bulk permeability exceeded matrix permeability by two orders of magnitude in unoxidized glacial till in Saskatchewan, Canada. This large difference may be attributed to the lack of fractures or discontinuous sand lenses encountered in the small laboratory samples. Keller, Van Der Kamp, and Cherry (1988) found that the presence of secondary conductive features is responsible for the higher values of hydraulic conductivity for slug tests when compared to laboratory permeameter tests in unoxidized till. This study also revealed that, as the well screen length increases, the slug test hydraulic conductivity exhibits less variability and the values tend to be closer to the true bulk hydraulic conductivity. Grisak and Cherry (1975) found evidence that fractures that are open and interconnected control the bulk hydraulic conductivity of the glacial till in Manitoba, Canada. Ruland, Cherry, and Feenstra (1991) suggest that vertical bore holes, because of the increase in fracture spacing with depth, may miss intersecting enough fractures to accurately determine the number of vertical fractures in deeper clay tills. D'Astous et al. (1988) found that larger diameter wells and angled drilling both have a higher probability of intersecting vertical fractures, allowing measurement of of a bulk hydraulic conductivity that is in the range expected for fracture flow in glacial till.

Piezometers may not accurately reflect true contaminant concentration in glacial till if the bore hole has not intersected the higher permeability fractures which are the primary paths of flow. D'Astous et al. (1988) observed a smear zone in the bore hole caused by the spiralling action of the auger resulting in occluded fractures at the bore-hole periphery. This will substantially reduce the hydraulic pathways which makes well development a critical step before field testing to determine hydraulic parameters. It was noted that during a period of drought, clay units with fractures drained more rapidly than blocks of clay matrix. Fluctuations in water levels due to the growing season evapotranspiration is much more pronounced in fractured clays than in unfractured clays. This is a result of the low permeability deposits inhibiting water movement even with a considerable hydraulic gradient (Ruland, Cherry, and Feenstra,1991).

Cravens and Ruedisili (1987) determined that oxidized till is recharged through root holes and discharges primarily through evapotranspiration rather than to the underlying unoxidized till. Fracture frequency and the degree of interconnectedness decreases with depth which causes hydraulic conductivity values to decrease with depth below the ground surface

(Ruland, Cherry, and Feenstra, 1991). The reduction of hydraulic conductivity with depth is also attributed to overburden pressure (Prudic, 1982). Grisak and Cherry (1975) found evidence that suggests interconnected fractures in till, rather than intergranular pore network, control the bulk hydraulic conductivity. Fractures in tills have been found at depths of 15 meters (50 ft) in southeastern Manitoba (Grisak and Cherry, 1975).

Hendry (1982) observed sand layers and streaks in both the weathered and unweathered till zones of the Interior Plains Region of southern Alberta, Canada. Ground-water velocity in the downward direction is approximately 2-6 meters/1000 years while the lateral movement is approximately 9 meters/1000 years. The low lateral value is attributed to the relatively flat surficial topography. Field tests in South Dakota tills revealed no significant migration of water between the oxidized and unoxidized tills (Cravens and Ruedisili, 1987). In some regions of North America where deposits of glacial till are thin, fractures may penetrate the entire unit allowing groundwater flow through the fractures to the underlying aquifers (Ruland, Cherry, and Feenstra, 1991). Hydraulic conductivities determined from field tests in unoxidized tills in Canada range from 1.8×10^{-6} - 6.3×10^{-9} cm/s. Studies of Illinois and Wisconsin unoxidized tills range from 3.2×10^{-5} - 8.4×10^{-8} cm/s. Results from selected studies can be found in Table 1.

Glacial tills of the St. Clair Basin of Ontario, Canada are comprised of 40-60% clay, 30-40% silt, and 5-10% sand (Desaulniers, Cherry, and Fritz, 1981). Hendry (1988) found oxidized and unoxidized tills had similar textures and matrix hydraulic conductivities in studies in Ontario, Canada. Bradbury (1991) found that the larger scale measurements gave higher values of hydraulic conductivity. Significant depositional environment changes over a short distance can severely alter the hydraulic conductivity, therefore field measurements to determine permeability should be on the same scale as the field problem (Bradbury, 1991). Particle size distribution produced widely varying results when used to estimate hydraulic conductivity. However, investigators often base sampling and testing methods on economic factors which may not produce the most accurate estimates of hydrological parameters (Bradbury and Muldoon, 1990).

Field Site Literature

Table 2 summarizes the hydraulic conductivity tests of the till at the study site, and identifies the reported value, the type of test used to determine hydraulic conductivity, and the investigator. Lutenegger (1989) reports finding no indication of fractures at depths below 4 meters (13 ft) in the massive gray till which is unoxidized and unleached. In field tests by Jones, Raaii, and Tsai (1990) in the oxidized till, the average hydraulic conductivity was 2.0×10^{-4} cm/s and the specific yield was 0.04. Lutenegger (1990) found that hydraulic conductivity ranging in values from 2×10^{-8} cm/s to 5.3×10^{-8} cm/s in samples taken at 6 meters (20 ft), decreased as confining pressure increased.

With higher levels of confining stress, micro-cracks may seal and porosity may be reduced due to consolidation. In triaxial permeability tests,

Table 1. Reported hydraulic conductivity values of glacial till

Hydraulic conductivity (cm/s)	Depth (m)	Test	Location	Investigator
$0.8\text{–}4.0 \times 10^{-8}$	9.5	slug	Ontario, Canada	Ruland, Cherry, & Feenstra
$1.8\text{–}2.2 \times 10^{-6}$	11.2	slug	Saskatchewan, Canada	Keller, Van Der Kamp, & Cherry
$3.9\text{–}8.4 \times 10^{-8}$	n.g.	slug	Wilsonville, Illinois	Herzog, et al.
8.5×10^{-7}	n.g.	recovery	Vandalia till	Herzog, et al.
6.3×10^{-7} – 3.2×10^{-5}	n.g.	slug	Eastern Wisconsin	Bradbury & Muldoon
1.6×10^{-8}	11.6	slug	Sarnia, Ontario Canada	Goodall & Quigley
$2\text{–}7 \times 10^{-6}$	n.g.	slug	Sarnia, Ontario Canada	D'Astous
6.13×10^{-9}	n.g.	slug	Manitoba, Canada	Grisak & Cherry
6×10^{-8}	n.g.	slug	New York	Prudic
4.27×10^{-7}	n.g.	slug	South Dakota	Cravens & Ruedisili
3.9×10^{-7} (V)	8.8	lab	Eastern Iowa	Lutenegger
8.0×10^{-9} (V)	n.g.	lab	Eastern Iowa	Lutenegger
9.8×10^{-9} (H)	n.g.	lab	Eastern Iowa	Lutenegger

Handy and Wang (1990) found that lateral stresses are many times higher than the present overburden pressures, some of which were high enough to effectively close vertical fractures.

Lutenegger (1989) believes that below 9 meters (30 ft), hydraulic conductivity is nearly unchanged with an average value of 1.7×10^{-8} cm/s. Lutenegger (1989) also found very little difference in measured values of vertical and horizontal values. Tsai (1991) determined that the average hydraulic conductivity of the oxidized till at field 5 to be 5×10^{-4} cm/s. At a depth from 9-18 meters (30-60 ft) the mean hydraulic conductivity from recovery tests was found to be 1.8×10^{-6} cm/s (Everts, et al., 1990). Estimates of hydraulic conductivity based on DMT dissipation tests show values of approximately 1×10^{-6} cm/s while flexible-wall hydraulic conductivity tests showed 1×10^{-7} cm/s. Shelby tube cores taken at 11.5 meters (38 ft) depth were lab tested for saturated hydraulic conductivity by Lutenegger (1989) revealing average values of 4.44×10^{-7} cm/s to 2.94×10^{-6} cm/s. Through recovery tests, Kanwar et al. (1989) found the geometric mean hydraulic conductivity to be 4.1×10^{-6} cm/s with a range from 4.8×10^{-7} cm/s to 1.5×10^{-4} cm/s. Using two-foot screens, Kanwar et al. (1989) obtained hydraulic conductivity values of 5.3×10^{-7} cm/s at 10.5 meters (35 ft) depth and 9.9×10^{-7} cm/s at 12 meters (40 ft) depth.

Shorter screens tend to produce lower hydraulic conductivity values since they are more likely to miss fractures and sand inclusions. From 9.5-13.7 meters (31 to 45 ft) below the ground surface, the geometric mean hydraulic conductivity of the unoxidized till is 1.2×10^{-6} cm/s (Kanwar, 1989). Lutenegger (1989) found very little anisotrophy in the hydraulic conductivity values as the horizontal flow generally gave the same results as the vertical flow. In Triaxial permeability tests, Handy and Wang (1990) found the hydraulic conductivity of the unweathered till ranged from 1 $\times 10^{-8}$ cm/s to 1 $\times 10^{-7}$ cm/s.

In triaxial compression tests conducted at this site by Lutenegger (1990), at depths of 9 meters (30 ft) in the unoxidized till, the % water = 13.9%, and the porosity = 26.6%. Lutenegger (1989) reported an average porosity of 27% for the unoxidized till and 30-40% for the overlying oxidized till. Tsai (1991) determined the bulk densities of the oxidized till and the unoxidized till to be 1.84 g/cm^3 and 1.91 g/cm^3 respectively for this site. The till is classified as a sandy loam close to loam with average percentages of particle size: clay = 10.7%, silt = 31.8%, sand = 54.1%, and gravel = 3.5% (Tsai, 1991). Lutenegger (1989) reported the bulk weight measurements of the oxidized till was 1.85 g/cc and 1.95 g/cc for the unoxidized till, and water content ranged from 13-15% for the unoxidized till and 15-23% for the oxidized till. Recharge estimates for the unoxidized till at a depth of 6-12 meters (20-40 ft) averaged 2 cm over a 14 month period (Everts et al. 1990).

Table 2. Reported hydraulic conductivity values at the test site.

Hydraulic conductivity (cm/s)	Depth (m)	Test	Investigator
$2 - 5 \times 10^{-8}$	6	confining stress	Lutenegger
1.7×10^{-8}	9	flexible wall	Lutenegger
5×10^{-4} (oxid)	2	pump	Tsai
1×10^{-7}	n.g.	flexible wall	Everts et al.
1×10^{-6}	9-18	DMT dissipation	Everts et al.
4.44×10^{-7} - 2.94×10^{-6}	11.5	laboratory	Lutenegger
4.1×10^{-6}	n.g.	recovery	Kanwar et al.
$5.3 - 9.9 \times 10^{-7}$	11	recovery	Kanwar et al.
1.2×10^{-6}	11.5	recovery	Kanwar et al.
1×10^{-8} - 1×10^{-7}	n.g.	triaxial permeability	Handy & Wang

(n.g. = not given)

MATERIALS AND METHODS

Field Site

The field test site is located at the Iowa State University Agronomy/Agricultural Engineering Research Center 8 miles west of Ames and one-half mile south of US Highway 30 (SE1/4 of NW1/4 of Section 8, T.83N, R.25W; Boone County, Iowa). Figure 1 shows the location of test field 5 referenced to U. S. Hwy. 30 and other site features. The site has a surface cover of short grass and has approximately a 3% grade sloping to the west-southwest (WSW). The uppermost till unit was deposited as a basal till by the Cary Lobe of the Wisconsin glacier approximately 13000 to 14000 years before present. There is a shallow layer of soil 0.6 meters (2 ft) covering the oxidized till which is underlain by a thicker bed of unoxidized till.

The oxidized till is a yellow brown to yellow tan layer approximately 3 meters (10 ft) thick overlying the unoxidized till which is a poorly sorted dense gray till approximately 19 meters (63 ft) thick with a variety of particle sizes. Monitoring of wells in the unconfined oxidized till performed by Tsai (1991) indicates the water table gradient closely resembles the surface drainage which is to the WSW. Piezometers screened at 6 to 7.5 meters (20-25 ft) show ground water head contours that also indicate flow toward the WSW (Everts, et al.,1990). The potentiometric head of the wells in the unoxidized till is very close to the water table level, although there is a slight downward head gradient. A set of shallow tile drainage lines are located on either side of the site but it was not anticipated that they would interfere with the pumping tests.

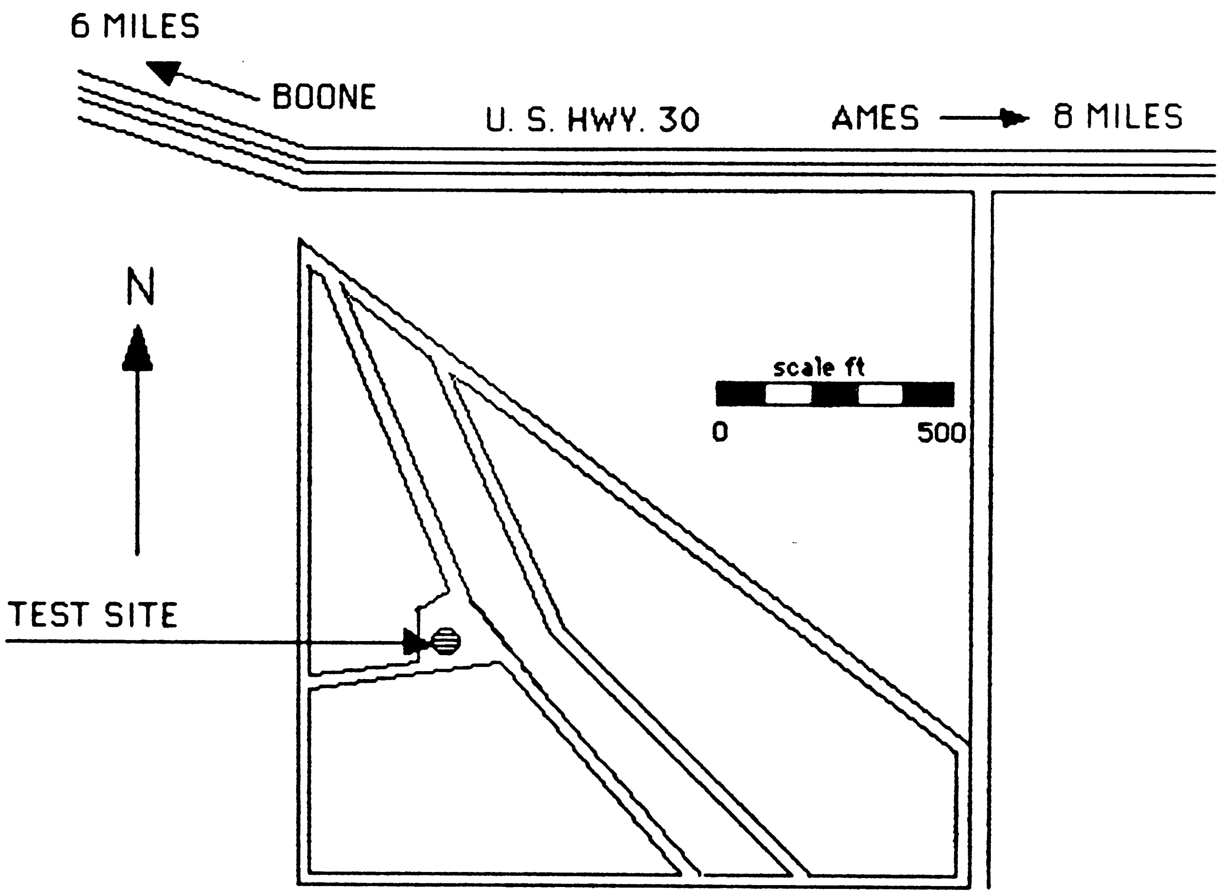

Figure 1. Location of Well Field 5

Well Construction and Development

Bore holes for wells IW-1 and MW-1 were machine augered using 21.6 cm (8.5 inch) diameter hollow-stem augers without the addition of any drilling fluids. A plan view of the monitoring well layout is shown in figure 2. Great resistance was encountered while drilling beyond 7.5 meters (25 ft) into the dense unoxidized till. Drilling from 9 to 12 meters (30 to 40 ft) depth required approximately the same amount of time as drilling the first 9 meters (30 ft). Two bore holes were abandoned, one at 3 meters (10 ft) depth and another at 6 meters (20 ft) depth as a result of a bore hole collapse. The abandoned holes were filled with bentonite and capped off with augered dense till to prevent interference with the active wells.

Wells IW-1 and MW-1 were constructed of 5.08 cm (2 inch) diameter schedule 40 PVC pipe commercially slotted at 25.4 mm (0.01 inch) and threaded. Figure 3 shows a vertical cross-section of the site depicting the construction details for wells IW-1 & MW-1 and till contacts. The 3 meter (10 ft) sections of risers and screen were threaded together using rubber O-rings to prevent leakage. The well screen and casing and gravel pack were set within the hollow stem auger in the bore hole to prevent a collapse of the hole. Pea gravel was placed from the bottom of the 21.6 cm (8.5 inch) bore hole to 0.15 meters (0.5 ft) above the top of the screen. Powdered bentonite was placed atop the gravel pack in the annular space for 6 meters (20 ft) to prevent water from infiltrating from the overlying unoxidized till. The remainder of the bore hole was filled with dense till which had been

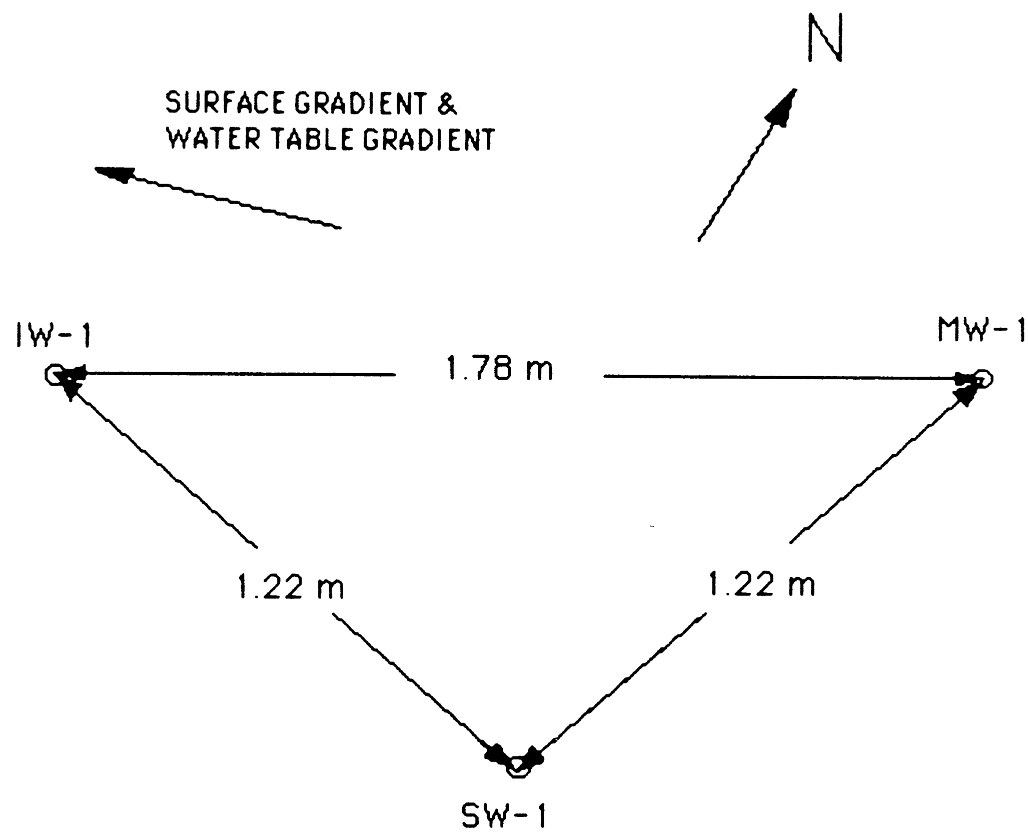

Figure 2. Plan view of the well layout. IW-1 & MW-1 are screened from 9.02 to 12.07 meters, SW-1 is screened from 1.16 to 4.21 meters. (scale: 1 inch = 0.39 meters)

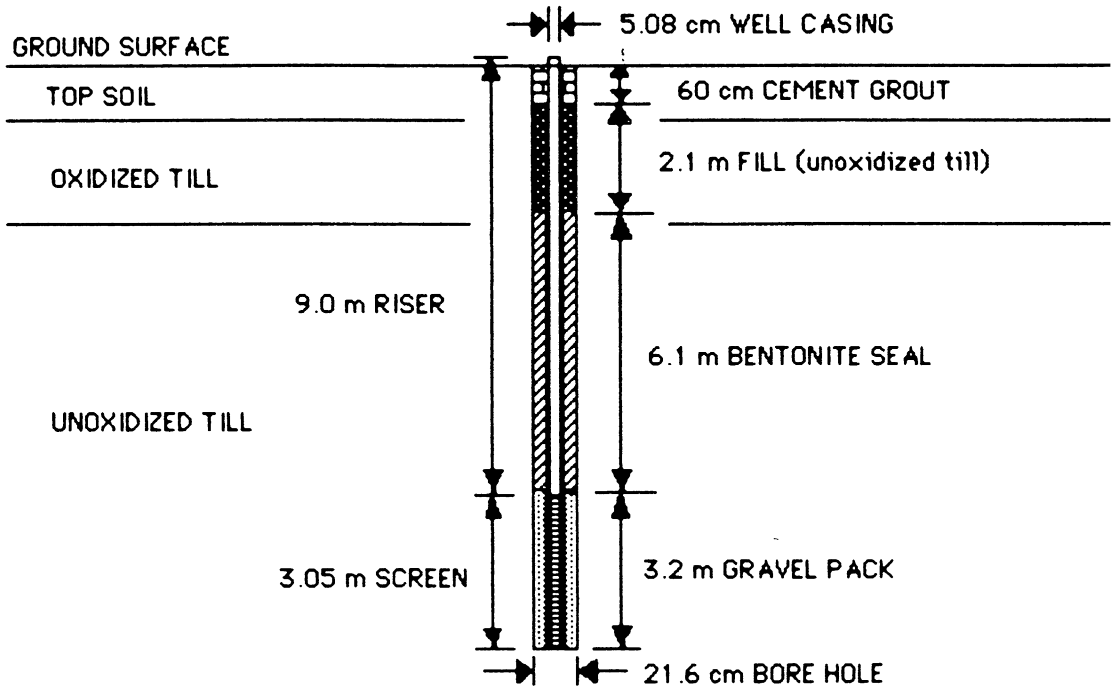

Figure 3. Vertical cross-section of wells IW-1 and MW-1 (not to scale)

augered up from drilling and was finished off with a 10 cm (4 inch) diameter PVC casing encased in 0.6 meters (2 ft) of concrete which was mounded up to prevent surface water from pooling near the wellhead. MW-1 is 1.78 meters (5.84 ft) east (upgradient) from IW-1. Both wells were 12.07 meters (39.6 ft) deep and are screened for 3.05 meters (10 ft) from 9.02 to 12.07 meters (29.6-39.6 ft) below ground surface, partially penetrating the unoxidized till.

Prior to pump test #2 a 8.9 cm (3.5 inch) bore hole was hand augered and a 4.2 meter (13.8 ft) shallow well (SW-1) was installed equidistant 1.22 meters (4 feet) south from MW-1 and IW-1, thus forming a triangle a (see Figure 2). A 2.54 cm (1 inch) diameter commercially screened and threaded PVC was used with the screen placement from 1.2-4.2 meters (3.8-13.8 ft) below the surface. Monitoring well SW-1 was set primarily through the unconfined aquifer in the oxidized till and slightly into the unoxidized till. A filter pack of pea gravel was placed from 9.5 to 4.2 meters (3.1-13.8 ft) below the surface. Powdered bentonite and cement were used to seal off the remainder of the well to prevent the infiltration of surface water. Figure 4 shows the construction details of monitoring well SW-1.

All three wells were developed by two methods: 1) pumping manually with a foot valve and 5/8 inch I.D. High Density Polyethylene tubing until empty, and 2) alternately pumping the well dry with a peristaltic pump then adding water and air pressure. This purge and surge action was undertaken to remove fine particles from the gravel pack and to alleviate the problem of sidewall smearing by the augers which may have sealed off fractures and sand lenses. Generally the water level in the shallow well SW-1 was 1.4

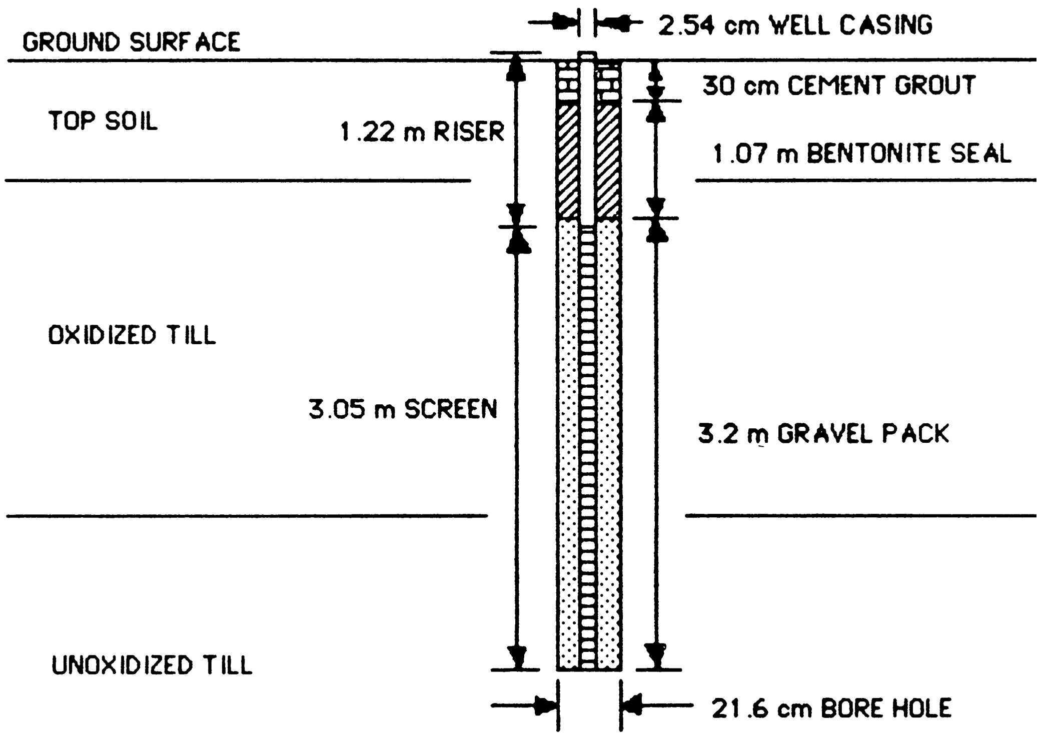

Figure 4. Vertical cross-section of well SW-1
(not to scale)

meters (4.5 ft) and the potentiometric head in deep wells IW-1 & MW-1 was 1.59 meters (5.21 ft) below the ground surface.

Pump Test Procedures

Two constant-head pumping tests were performed on well IW-1 while observing the hydraulic head response in monitoring well MW-1. Well IW-1 was pumped down to a constant level while recording the pumping rate and observing the hydraulic head response in well MW-1 versus time. The drawdown in the pumping well remained constant while the discharge varied against time. A gasoline powered generator provided the electrical power to operate the pumps, lights, and heater used for the tests. Two peristaltic tubing pumps with #17 rubber tubing were used to keep a constant drawdown in well IW-1 in the event of one pump failing. The two tubes were taped together and inserted into IW-1 and then were secured with a lab clip to the top of the well riser. Flow rate was determined by capturing the pumped water in 2-liter graduated cylinders for periods of time, beginning with 2 minute intervals and gradually increasing to 15 and 20 minute intervals.

Pump Test 1 Performance

Pump test # 1 was run November 21, 1990 before the ground was frozen with an initial water level before pumping of 1.75 meters (5.74 ft) below ground surface. There was no recorded precipitation during the test. Flow rates were taken spontaneously at approximately 7 hour intervals after closer monitoring during the early portions of the test when rates were rapidly changing. The constant head pumping test was performed on well IW-1 for a period of 98 hours (4.08 days), reducing the water level from 1.75 meters (5.74 ft) to 5.72 meters (18.76 ft) below ground level for a drawdown of 3.97 meters (13.02 ft). The hydraulic head response in monitoring well MW-1 was observed during the pumping test. A one-hour electrical failure during pump test #1 did not significantly affect the drawdown response in the monitoring well. The average pumping rate was 26.59 ml/min. Figure 5 shows the flowrate versus time for IW-1 in pump test 1. The monitoring well MW-1 drawdown and recovery curve is shown in Figure 6.

Pump Test 2 Performance

Pump test #2 pumping began at 8:21 AM on January 10,1990 and ran for a period of 123.8 hours (5.16 days). A heated 4 X 6 foot metal shed was placed over the three wells to facilitate pumping and monitoring through adverse winter conditions. The pump test was started while there was a heavy snowcover over the site 5 field. Daily temperatures ranged from a daytime high of 22 degrees F to overnight lows down to 5 degrees F throughout the testing period. The initial water level in IW-1 was 1.47 meters (4.83 ft) from the ground surface. Pumping at a rate of 850 to 900 ml/min., while drawing primarily from storage, the level in the pumping well

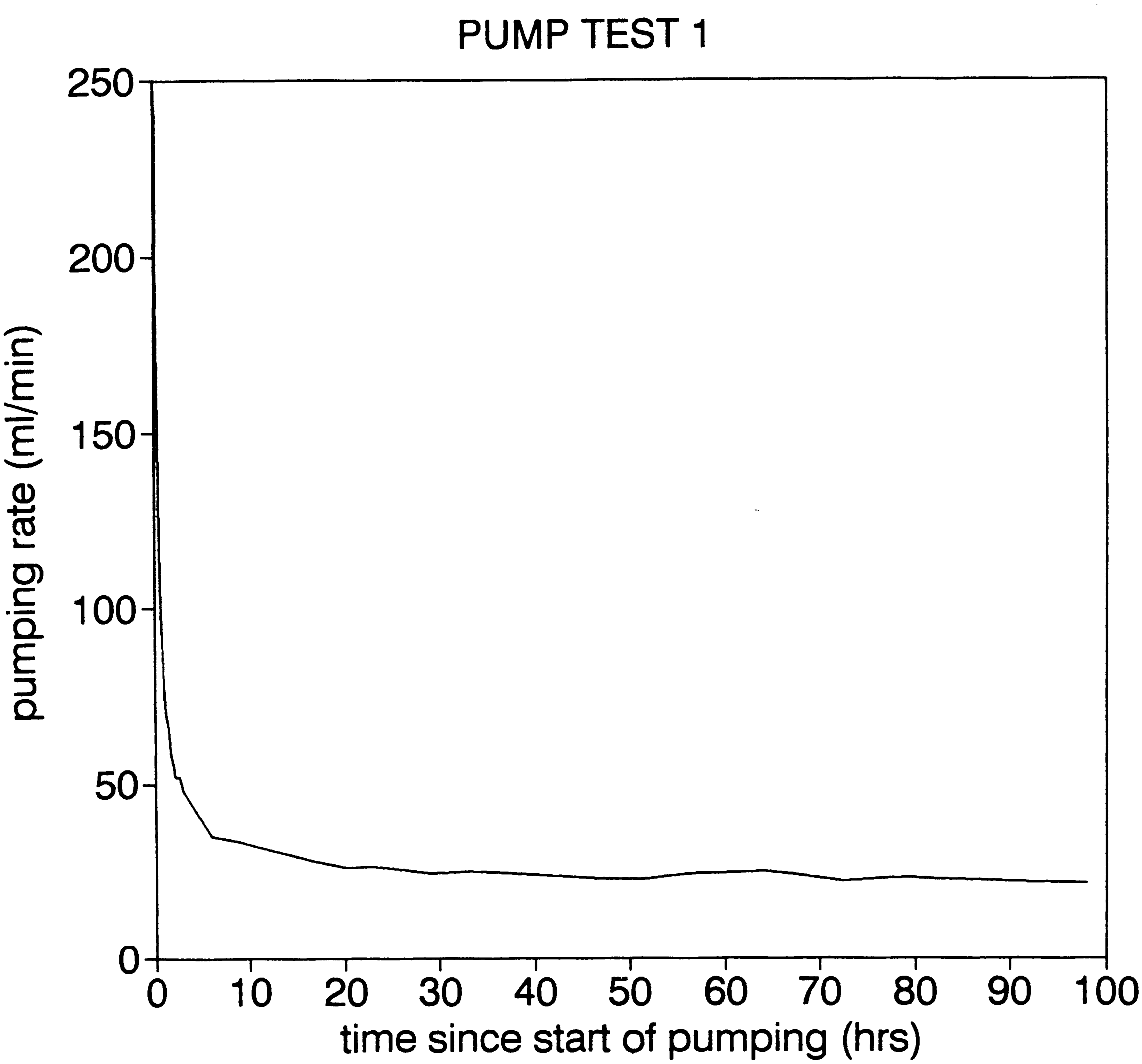

Figure 5. Pump test 1 pumping rate versus time

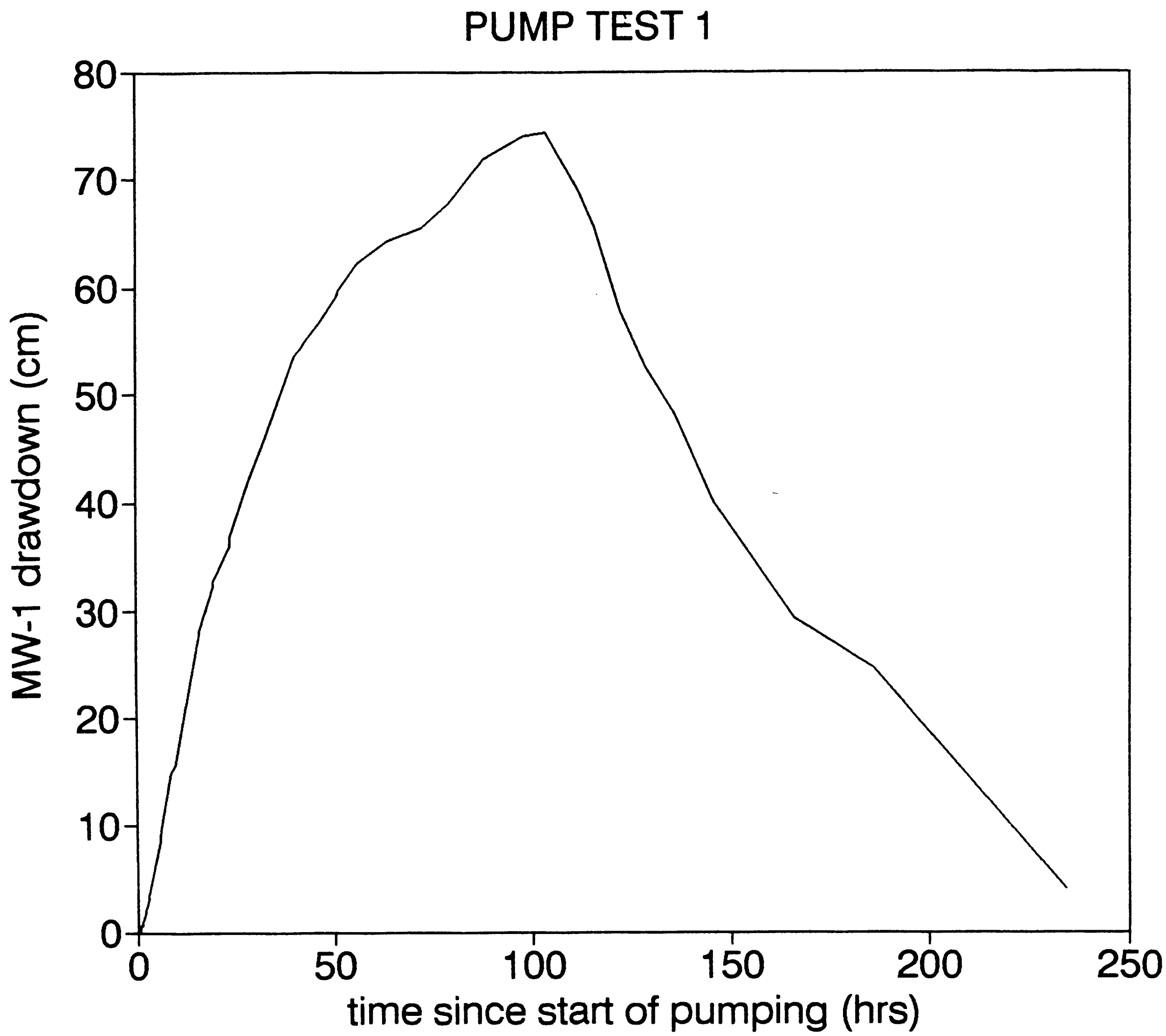

Figure 6. Pump test 1 MW-1 drawdown response and recovery

IW-1 was lowered from 1.47 meters (4.83 ft) to 5.87 meters (19.25 ft), for a drawdown of 4.41 meters (14.47 ft), in approximately 16 minutes. Figure 7 shows the flowrate versus time for IW-1 in pump test 2.

The flow rate quickly dropped off to 140 ml/min. at 30 minutes and 93 ml/ min. at 1 hour into the test. Later in pump test 2 when flow rates were much lower, pumped water was captured in a 5 gallon plastic bottle for periods up to 8 hours. This pumped water was then carefully measured in a 2 liter graduated cylinder and the total pumping time was noted to obtain an average flow rate over the period. The average pumping rate over the test was 26.30 ml/min.

Thirty five minutes into pump test *2, monitoring well MW-1, which is located 1.78 meters (5.84 ft) from pumping well IW-1 showed a head drop of .305 cm (0.01 ft). After 5 hours the drop in head was 8.84 cm (0.29 feet), and there was a 30.5 cm (1.00 ft) drop after 13.5 hours. By the end of the pump test (123 hours) MW-1 had a head reduction of 1.04 meters (3.41 ft). Figure 8 shows the drawdown response and recovery of well MW-1 for pump test 2.

SW-1 was monitored for any changes in head which would indicate that some recharge of the unoxidized till was occurring from the overlying unconfined oxidized till. There was no significant drop in the water level of SW-1 during the pump test. The slight variations in water levels were attributed to natural fluctuations in the water table. It is doubtful that a pumping test with extremely low flow rates could induce a head change in the overlying unconfined aquifer which has a hydraulic conductivity two orders of magnitude higher.

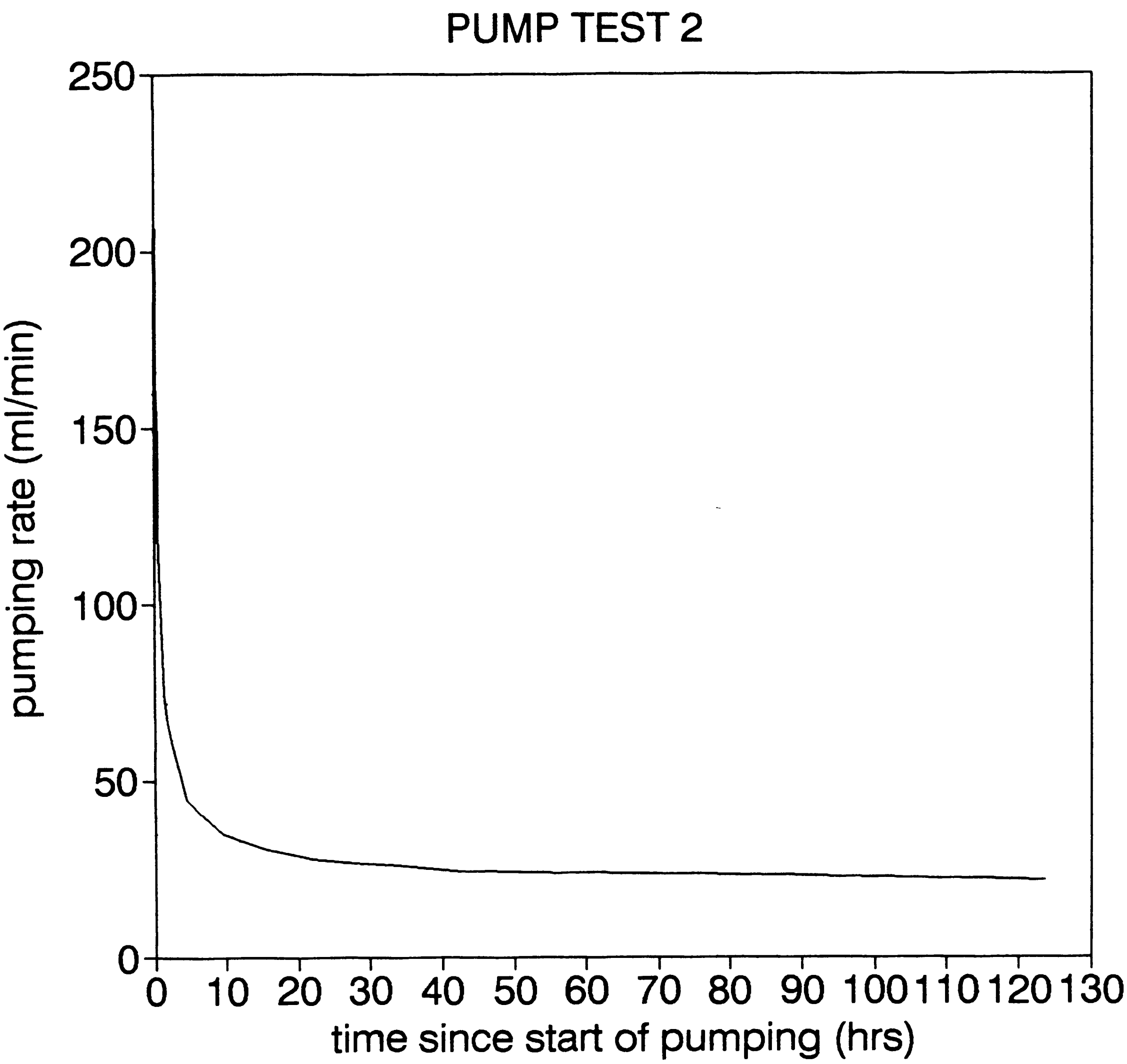

Figure 7. Pump test 2 pumping rate versus time

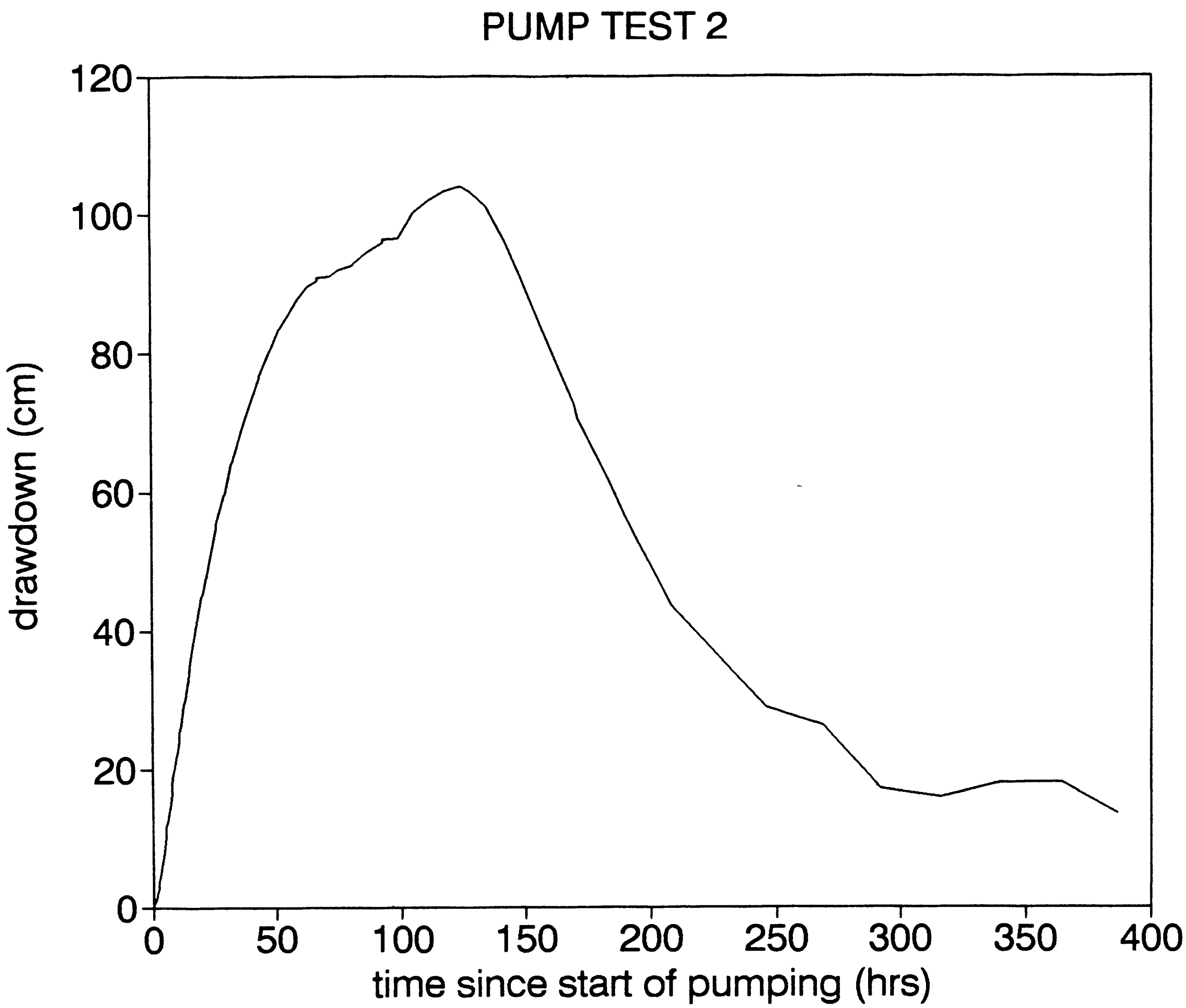

Figure 8. Pump test 2 MW-1 drawdown response and recovery

At 14 hours and 15 minutes into the test a small amount of pumped water was lost due to a split that formed in the tubing on the discharge side of the pump. This also occurred at 25 hours and 36 minutes into the test. An application of duct tape effectively sealed the splits and two older Masterflex pumps were installed to replace the quick release models in order to prevent further splitting. While a small amount of pumped water was lost for measuring, IW-1 remained pumped down to 19.3 feet during the entire test. The flow rates of pump test #2 were adjusted for the short period where pumped water was leaking from splits in the tubing. In the early portion of the pump tests the volume of water stored in the well casing was subtracted from the pumped volume to determine the true flow rate from the aquifer.

Pump Test Analysis Methods

Theis solution

The confined aquifer flow equation was solved by C. V. Theis in 1935 with existing solutions used to solve heat flow equations (Fetter, 1988). The governing equation for radial flow in a homogeneous and isotropic confined aquifer is :

$$\frac{\partial^2 s}{\partial r^2} + \frac{1}{r}\frac{\partial s}{\partial r} = \frac{S_c}{T}\frac{\partial s}{\partial t} \qquad (1)$$

with the boundary conditions:

$$s(r,t) = 0 \text{ for all } r,\ t = 0 \tag{2}$$

$$s(r,t) = 0 \text{ for } t > 0,\ r \rightarrow \infty \tag{3}$$

$$\lim_{r \rightarrow r_w} \left(2\pi r T \frac{\partial s}{\partial r}\right) = Q \tag{4}$$

The solution Theis developed is:

$$s = \frac{Q}{4\pi T}\ W(u) \tag{5}$$

where the argument u is:

$$u = \frac{r^2\ S_c}{4Tt} \tag{6}$$

$$W(u) = \int_u^{\infty} \frac{e^{-u}}{u}\, du \tag{7}$$

where t = time since pumping started, s = the drawdown, r = the radial distance from the pumping well to the observation well, T = the coefficient of transmissibility of the aquifer, and S_c = the coefficient of storage of the aquifer.

The Theis solution requires the following assumptions:

1. the aquifer is homogeneous and isotropic and has infinite areal extent
2. the aquifer is confined by impermeable layers top and bottom
3. the pumping well fully penetrates the aquifer and pumps at a constant rate
4. no water is stored in the wells
5. the pumping well is considered to have an infinitesimal diameter

Super position

Super-position was used to account for the changes in pumping rate during the constant-head pumping test. Using discrete changes in the pumping rate, the theoretical drawdown (Bear, 1979) is estimated from super-position of the Theis solution by:

$$s_t(r, t, T, S_c) = \frac{1}{4\pi T}\sum_{i=1}^{n}(Q_i - Q_{i-1})W\left(\frac{r^2 S_c}{4T(t_i - t_{i-1})}\right) \tag{8}$$

$$t_{n-1} \leq t \leq t_n$$

where W( ) is the Theis well function, Q_i is the pumping rate from time t_{i-1} to t_i, with $Q_0 = 0$.

Pumping flowrate measurements were divided into discrete time intervals where the flowrate was treated as a constant during each time interval using the average flowrate over the time intervals. The time intervals were selected so that the actual changes in the pumping rate over the time interval were small. Time intervals in the early portion of the test were shorter, due to the rapidly changing pumping rate, and gradually become longer later in the test.

A least squares fitting method was used to determine transmissivity and storage coefficient with the super-position Theis solution. By assigning a set of T and S_c to the super-position Theis solution, a theoretical drawdown curve can be produced. The best fit is obtained by using the least squares method to determine the target T and S_c whose values minimize the sum of squared differences between the observed and predicted drawdown generated by the super-position Theis solution. The least squares objective is,

$$\text{minimize } F(T, S_c) = \sum_{i=1}^{n}(s_o(r_i, t_i) - s_t(r_i, t_i, T, S_c))^2 \qquad (9)$$

where s_o is the observed drawdown, s_t is the theoretical drawdown predicted by the super-position Theis solution, and n is the number of data points. A FORTRAN program using a generic least squares routine was used to solve the least squares problem. A subroutine in the program computes the difference between the measured values of the functions and the predicted values of the functions for particular values of the parameters; the values of $f_i(T, S_c)$. The subroutine solves the least squares minimization using a modified Levenberg-Marquardt algorithm, calling the problem specific subroutine to compute function values and first derivatives.

Partially penetrating wells

To examine whether the horizontal flow assumption of the Theis solution was reasonable, the transmissivity and storativity were also estimated with the use of the type curve for nonleaky isotropic confined aquifers with partially penetrating wells and constant discharge (Walton, 1970). The governing equation for partially penetrating wells (Walton,1970) includes both vertical and horizontal flow:

$$\frac{\partial^2 s}{\partial r^2} = \frac{1}{r}\frac{\partial Sc}{\partial r} = \frac{\partial^2 s}{\partial z^2} = \frac{Sc}{T}\frac{\partial s}{\partial t} \qquad (10)$$

where z is the vertical elevation above the arbitrary datum.

The time-drawdown data from each pump test was plotted on logarithmic paper and superimposed over the appropriate type given. For this purpose the unoxidized confined aquifer was estimated to be 14.2 meters (46.5 ft) thick with the wells screened through its center. Average pumping rate for each test was used in place of constant discharge (Q). A rather close fit was found for both pump test curves and the match points were obtained giving 1/u and W(u, r/m, y) values. Transmissivity and storativity were estimated by applying these values to the following equations:

$$s = \frac{Q}{T} W(u, \frac{r}{m}, y) \tag{11}$$

$$u = \frac{r^2 S_c}{Tt} \tag{12}$$

where s = drawdown (cm), r = distance from pumping well to monitoring well (cm), Q = discharge (ml/m), t = time since pumping started (min), T = coefficient of transmissibilty (cm^2/s), S_c = coefficient of storage fraction, m = aquifer thickness (cm), md = distance from top of aquifer to top of screen (cm), and $y = \frac{m-m_d}{m}$.

Pumping Test Results

It was anticipated that drawing down the water level in well IW-1 for a period of time would create some response in a nearby well screened at the same depth. Ruland, Cherry, and Feenstra (1991) determined that strong hydraulic heads can develop without a significant amount of water being drawn from unfractured clay due to the capillary effects which holds water in the small pore spaces. The data gathered from pump tests 1 & 2 were analyzed with a Theis solution computer program provided by Jones (1990) ignoring vertical flow, to estimate the bulk storativity and transmissivity of the aquitard. The transmissivities obtained from the Theis solution were converted to hydraulic conductivity (K=T/b) assuming horizontal flow using an aquifer thickness of 3.2 meters (10.5 ft), which is the length of the gravel pack.

Different portions of the drawdown data were analyzed to determine if one portion of the data substantially changed the results. These included the full test, from 6 hours into the test to the end, and the first one-half of the test. Data from the full test and from 6 hours into the test to the end were also evaluated using average flow. Results are listed in Table 3. Using all the time data, Figures 9 and 10 show the measured response and the Theis solution least squares fit to the drawdown in MW-1 for pumping tests 1 and 2, respectively.

Eliminating the first 6 hours of the test data did not substantially change the hydraulic conductivity or storativity values. Evaluating the entire data set with a changing pumping rate for pump test #1 showed a transmissivity of 4.14×10^{-2} cm^2/s, a hydraulic conductivity of 2.16×10^{-6} cm/s and a storativity value of 1.23×10^{-5} cm^{-1}. The entire data set for pump test #2 gave a transmissivity of 3.14×10^{-2} cm^2/min, a hydraulic conductivity of 1.64×10^{-6} cm/sec, and a storativity of 1.01×10^{-5} cm^{-1}.

Analyzing the pump test data with the partially penetrating type curve method produced an estimate of transmissivity of 1.80×10^{-3} cm^2/s for pump test 1 and 1.46×10^{-3} cm^2/s for pump test 2. This yields hydraulic conductivity values of 1.27×10^{-6} cm/s and 1.03×10^{-6} cm/s respectively as shown in Table 3. Coefficients of storage estimates were 4.89×10^{-3} for test 1 and 3.55×10^{-3} for test 2 giving storativities of 3.45×10^{-6} cm^{-1} and 2.51×10^{-6} cm^{-1} respectively. These hydraulic conductivities are only slightly lower than those estimated by the Theis solution using average flow

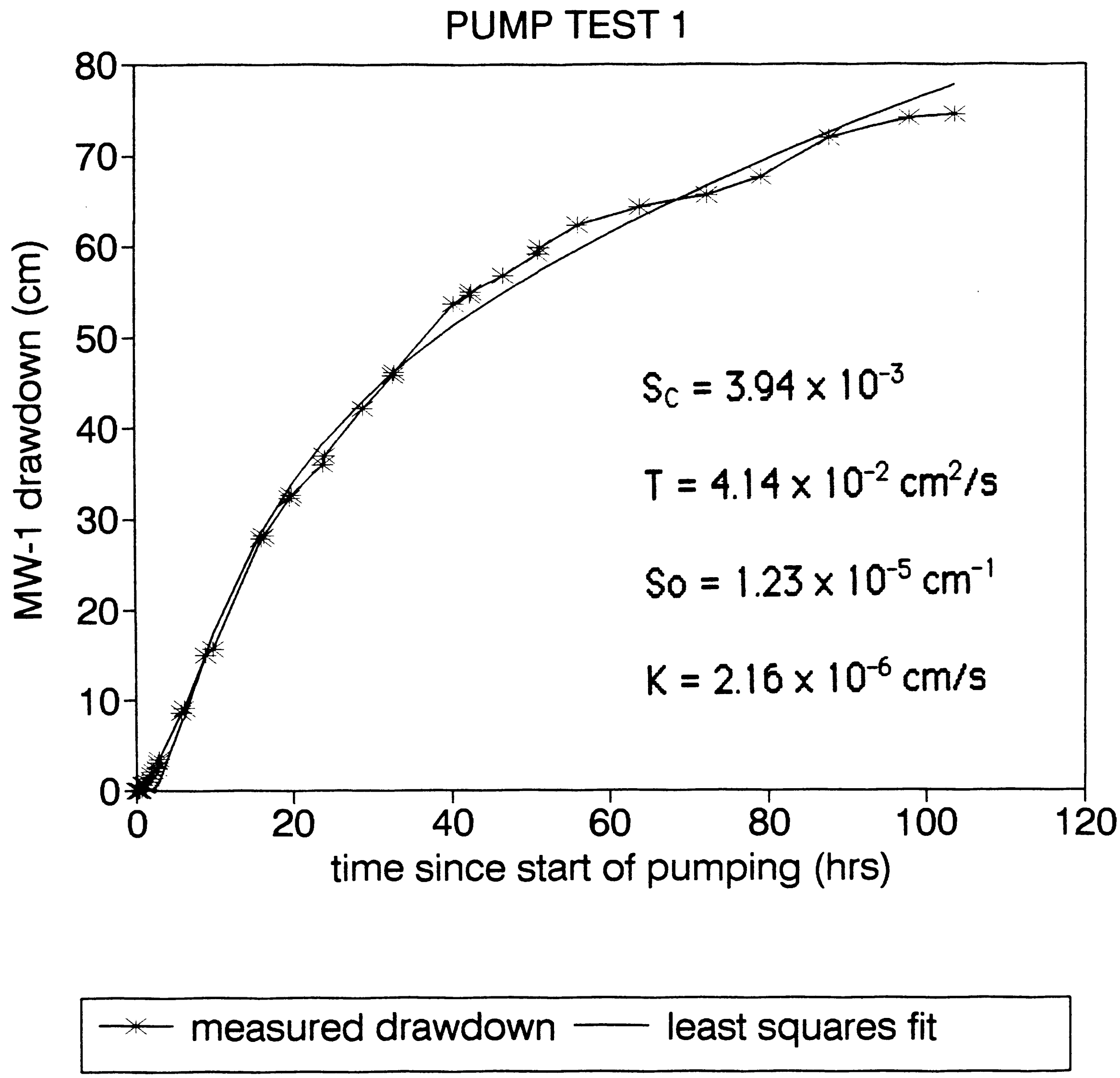

Figure 9. Pump test 1 drawdown in MW-1 vs. time and Theis solution least squares fit

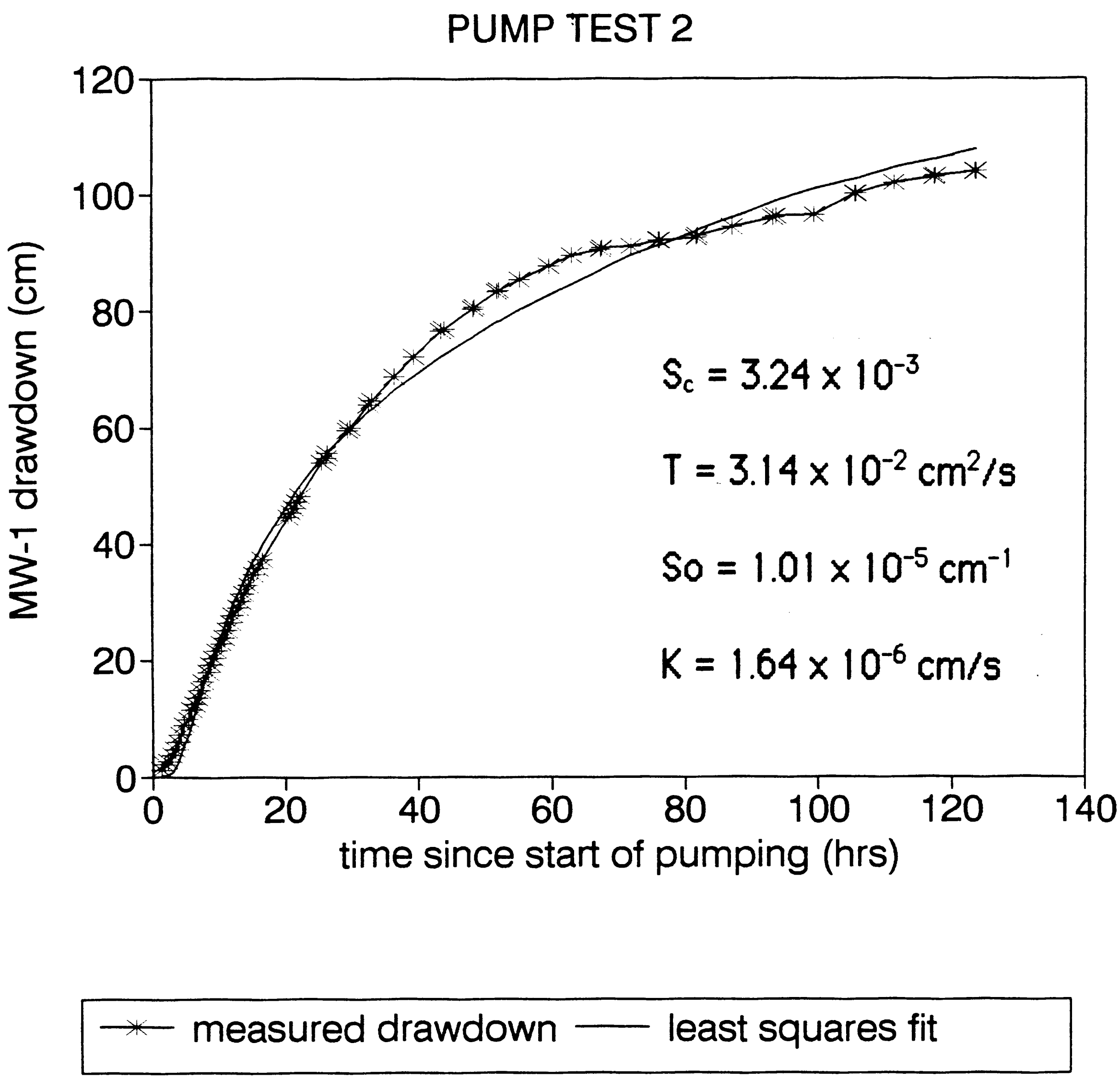

Figure 10. Pump test 2 drawdown in MW-1 vs. time and Theis solution least squares fit

rate. The partially penetrating type curve estimates which are shown in Table 3 are very supportive of the Theis solution results.

The bulk hydraulic conductivity values obtained from these pump tests were generally 1 to 2 orders of magnitude higher than those reported in unoxidized tills in Canada and South Dakota as reported by Goodall and Quigley (1977) and Cravens and Ruedisili (1987) and shown in Table 1.

Table 3. Bulk hydraulic conductivity and storativity results from pump tests.

PORTION OF	PUMP TEST 1		PUMP TEST 2	
DATA ANALYZED	K (cm/s)	STORATIVITY (cm^{-1})	K (cm/s)	STORATIVITY (cm^{-1})
Full test	2.16E-6	1.23E-5	1.64E-6	1.01E-5
6 hours to end	2.14E-6	1.24E-5	1.62E-6	1.01E-5
First half	2.28E-6	1.26E-5	1.46E-6	9.96E-6
Average flow full test	3.67E-6	7.68E-6	2.72E-6	5.64E-6
Average flow 6 hours to end	3.66E-6	7.68E-6	2.73E-6	5.64E-6
Partially penetrating well type curve	1.27E-6	3.45E-6	1.03E-6	2.51E-6

Bail Tests

Performance

Three bail tests were performed on wells IW-1 and MW-1 to estimate hydraulic conductivity for comparison with values obtained from pumping tests. Hydraulic conductivities were determined using the method of Hvorslev (1951) which involves rapidly lowering the hydraulic head and monitoring the rate of recovery in the wells. After the water level has been rapidly lowered, an equal amount of water will eventually flow into the well until the water level in the well reestablishes equilibrium with the potentiometric head of the aquifer.

The first bail tests were performed on October 21, 1990 before the wells were developed. With a peristaltic pump, the water level in IW-1 was lowered from 1.61 to 6.09 meters (5.27- 20 ft) below ground surface and MW-1 was lowered from 1.72 to 6.49 meters (5.63 -21.3 ft) below ground surface.

The second set of bail tests were performed on November 4, 1990 after the wells had been developed. Water levels were lowered manually with a nylon tube and ball valve. IW-1 was lowered from 1.69 to 8.90 meters (5.53-29.2 ft) and MW-1 was lowered from 1.64 to 9.24 meters (5.38-30.3 ft). For bail test numbers 1 & 2, recovery was monitored manually with an electric water depth indicator and was recorded against time at specific intervals.

The third bail tests on MW-1 and IW-1 were performed on February 10 and 18, 1991 respectively. The water was pumped down to 6.54 meters

(21.45 ft) manually with a nylon tube and ball valve. Recovery was monitored with a pressure transducer and recorded on a data logger. Water levels were periodically read manually with an electric water depth indicator to calibrate the millivolt readings of the data logger.

Analysis methods

Hvorslev (1951) developed a method of measuring hydrostatic pressure and determining permeability of observation wells using a time lag theory (Cedergren, 1989). To analyze a bail or slug test where water is either flowing into or out of a well, the flow q is expressed as:

$$q = Fkh = Fk(z-y) \tag{13}$$

where F = a factor that is determined by the size and shape of the well intake, h = the active head, z = the distance from the reference level to the water level in the well in the transient state. It is assumed that friction losses in the well can be neglected and the volume of flow during time dt is:

$$q\,dt = A\,dy \tag{14}$$

where A = the cross-sectional area of the well. Substituting the expression for q in (13) into (14) results in:

$$\frac{dy}{z-y} = \frac{Fk}{A}\,dt \tag{15}$$

$V = Ah$ is the volume of flow necessary to equalize the pressure difference. When the original rate of flow $q = Fkh$ is maintained, a basic time lag T is required for equalization of the pressure difference. Therefore,

the the basic differential equation for the hydrostatic time lag is:

$$T = \frac{V}{q} = \frac{Ah}{Fkh} = \frac{A}{Fk} \tag{16}$$

$$\text{and} \quad \frac{dy}{z-y} = \frac{dt}{T} \tag{17}$$

After lowering the head in a well and recording the recovering head at different time intervals, a plot is made of time on an arithmetic scale and head ratio (h/h_0) on log scale. When the head ratio = 0.37, this is the basic time lag. When the equalization ratio ($1 - h/h_0$) is 0.90, it corresponds to a time lag of 2.3 times the basic time lag. To corresponds to $h = 0.37h_0$ by:

$$\log_e\left(\frac{h_0}{h}\right) = \log_e\left(\frac{h_0}{0.37h_0}\right) = \log_e 2.7 = 1.0 \tag{18}$$

If a steady rate of flow q and a constant head h_0 are maintained:

$$k = \frac{q}{Fhc} \tag{19}$$

and with variable head conditions:

$$k = \frac{A}{F(t_2-t_1)} \log_e \frac{h_1}{h_2} \tag{20}$$

Once the basic time lag T is known, the coefficient of permeability can be determined by:

$$k = \frac{A}{FT} \tag{21}$$

A plot was made of time on the arithmetic scale and the head ratio since initial drawdown (h/h_0) using the early portion of the data (first 35 minutes). It was applied to the Hvorslev (1951) formula to determine the hydraulic conductivity:

$$K = \frac{r^2 \ln(L/R)}{2\ L\ T_0} \tag{22}$$

Where K = hydraulic conductivity, r = radius of well casing, L = well screen length, R = radius of the highest permeable zone (gravel pack), and T_0 = time required for the water to recover to 37% of the initial static level.

The assumptions of the Hvorslev method are: the aquifer is homogeneous, isotropic, infinite in vertical and areal extent, and that the medium and water are incompressible (Fetter, 1988).

Results

While a bail test provides information only for a small area surrounding the well, it does provide some values for comparison with pump test results. Figures 9-14 show recovery vs. time with least squares fit to determine T_0 for all three bail tests on wells MW-1 and IW-1.

Results from the three bail tests proved to be very consistent as the mean hydraulic conductivity for the three tests on IW-1 was 7.29 x 10^{-6} cm/sec. and 1.47 x 10^{-6} cm/sec. for MW-1 (see Table 4). Pump test permeability values fell between the individual well bail test values. Hydraulic conductivity values estimated with recovery tests by Everts et al. (1990) at comparable depths to MW-1 and IW-1 at the same study site are shown in Table 5. These values are 2 to 3 orders of magnitude lower than those estimated from the pump tests and bail tests in this study. There are two possible differences that may have caused this large disparity. First, wells MW-1 and IW-1 were augered dry, while the Everts et al. (1990) wells were installed with the use of drilling fluids. Fractures may have been sealed by the fine particles circulated in the drilling fluids. In addition, the screen lengths in the Everts et al. (1990) study were only 0.61 meters (2 ft) compared to the 3.05 meters (10 ft) screen length of MW-1 and IW-1 used in the present study. The longer screens would have a higher probability of intersecting hydraulically significant fractures.

The more rapid recovery of IW-1 compared to MW-1 in all the bail tests is probably due to a higher number of fractures or sand lenses in the vicinity of the screen of IW-1. These values are within a reasonable range for hydraulic conductivities for unoxidized till and concur with the pump test results for this site. Laboratory derived hydraulic conductivity values performed by Lutenegger (1990) and Handy and Wang (1990) tend to be lower, by one to two orders of magnitude, than these in-situ field test values.

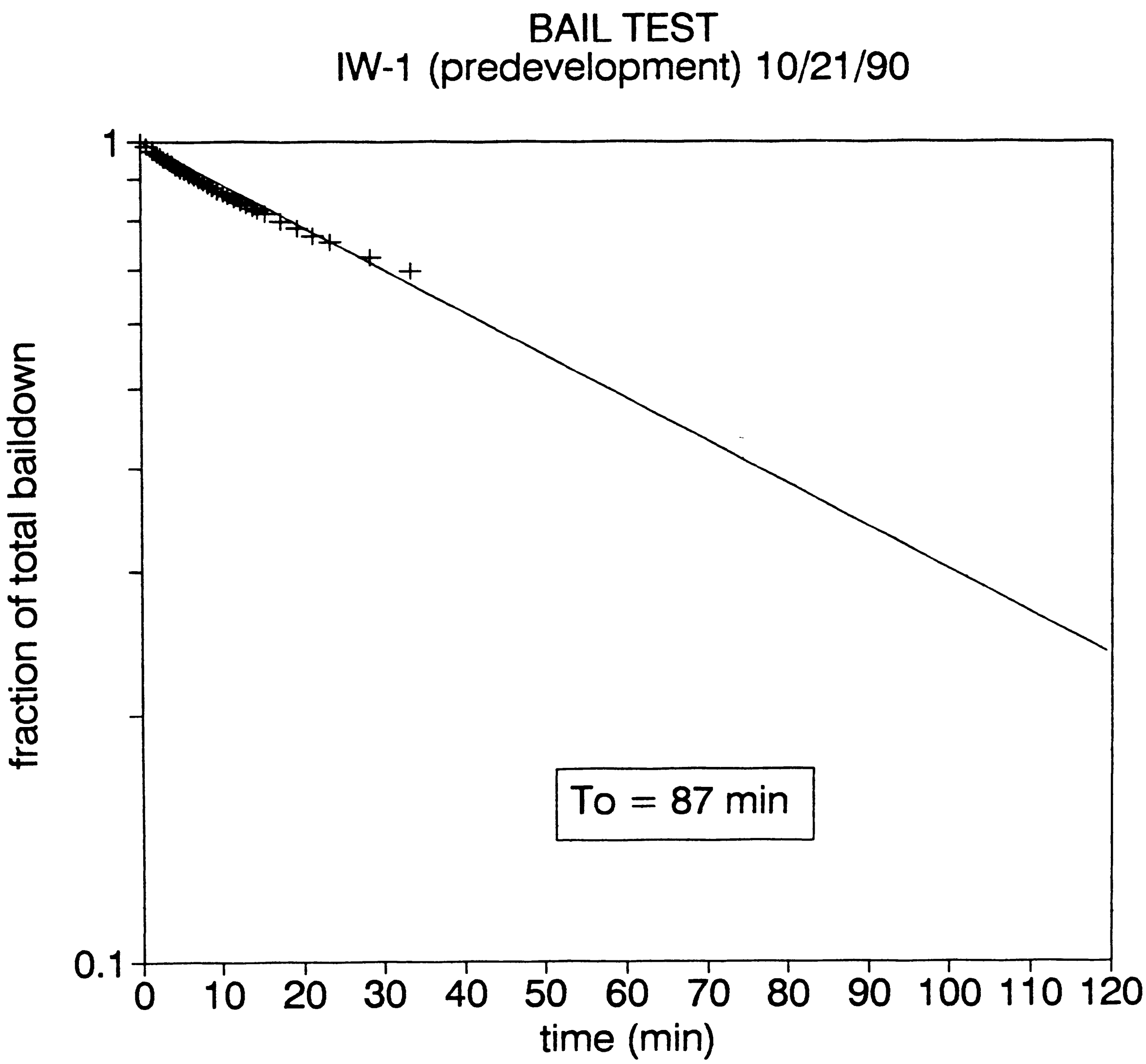

Figure 11. IW-1 predevelopment bail test recovery vs. time and least squares fit

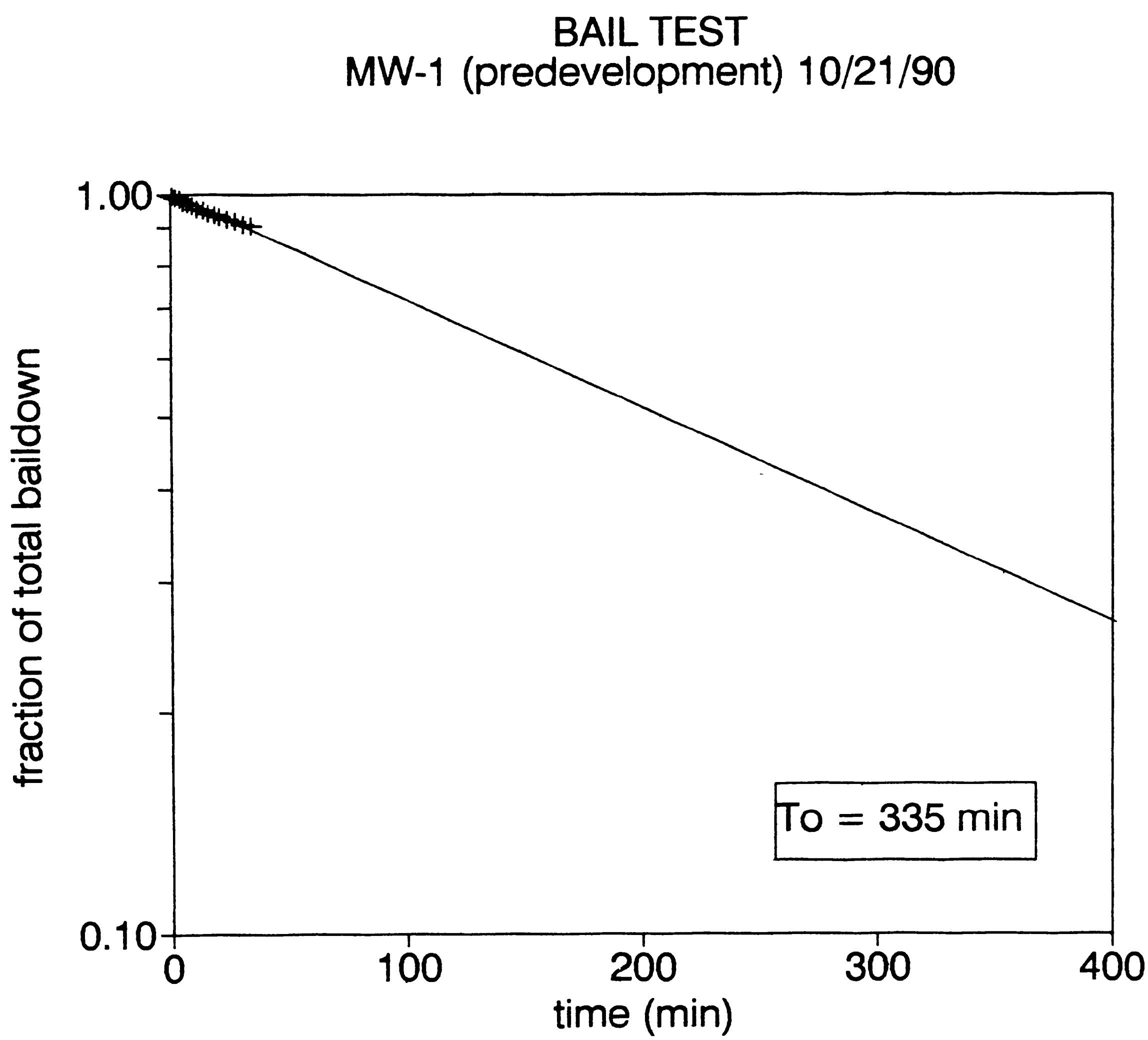

Figure 12. MW-1 predevelopment bail test recovery vs. time and least squares fit

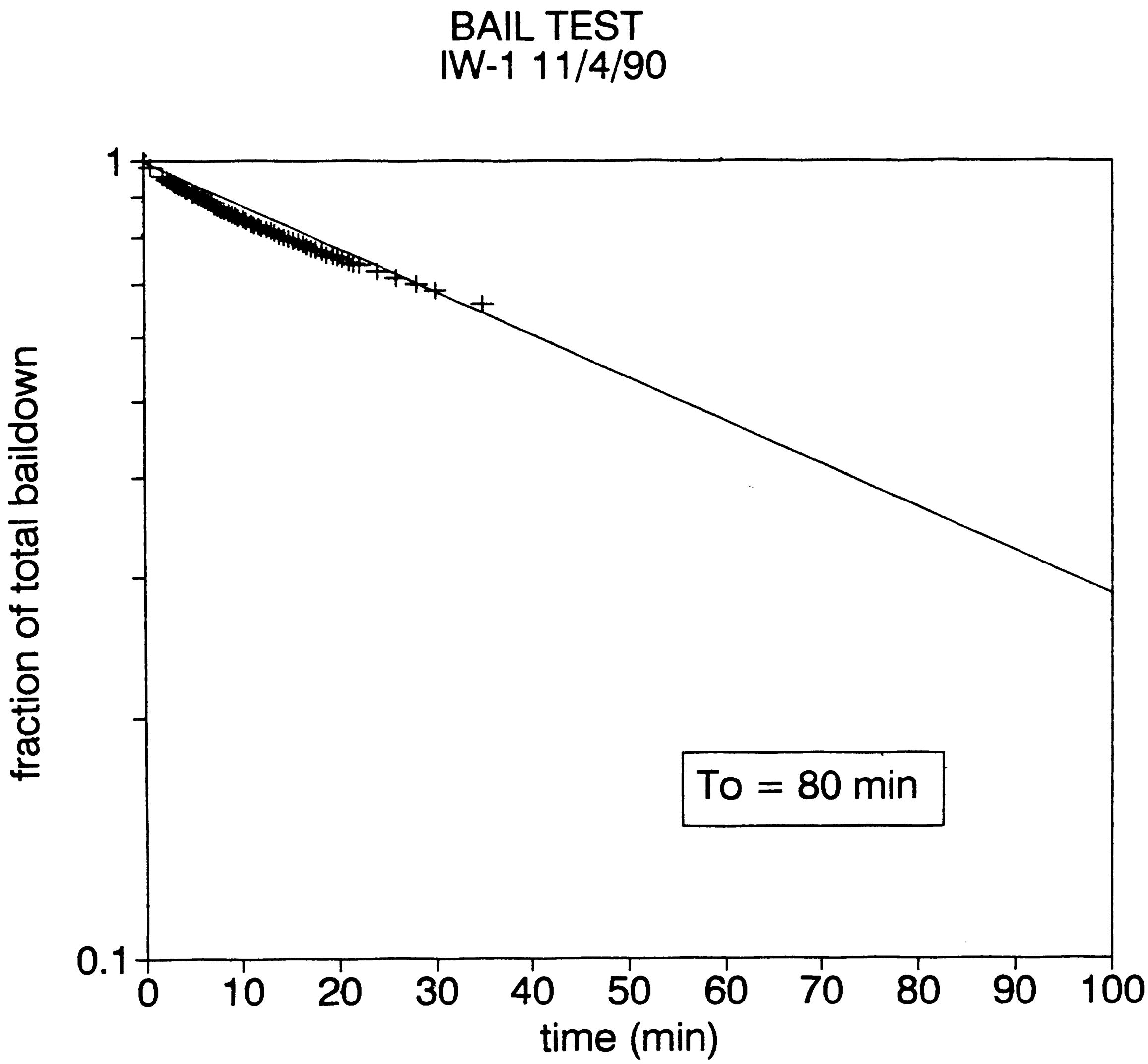

Figure 13. IW-1 bail test recovery vs. time and least squares fit

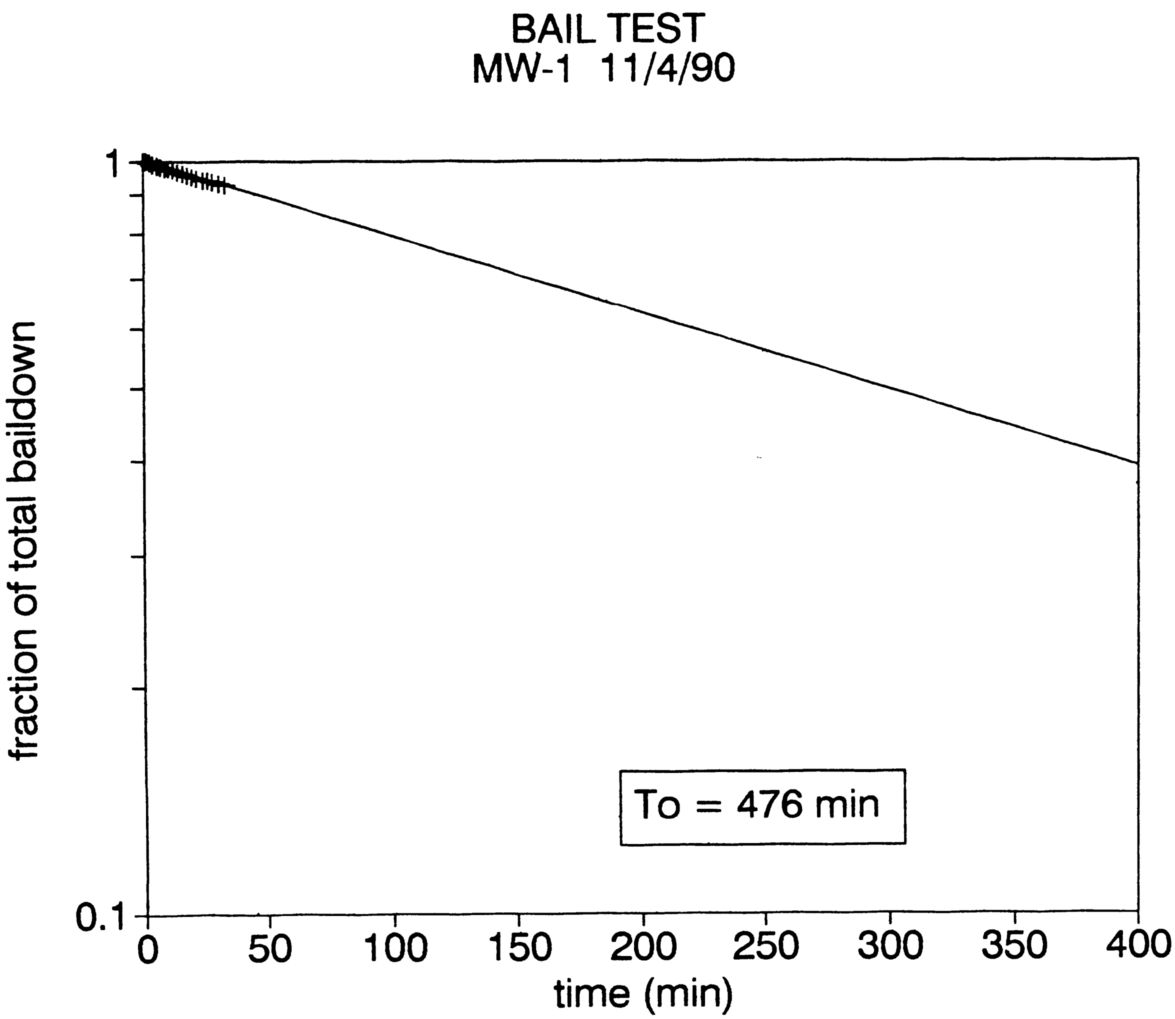

Figure 14. MW-1 bail test recovery vs. time and least squares fit

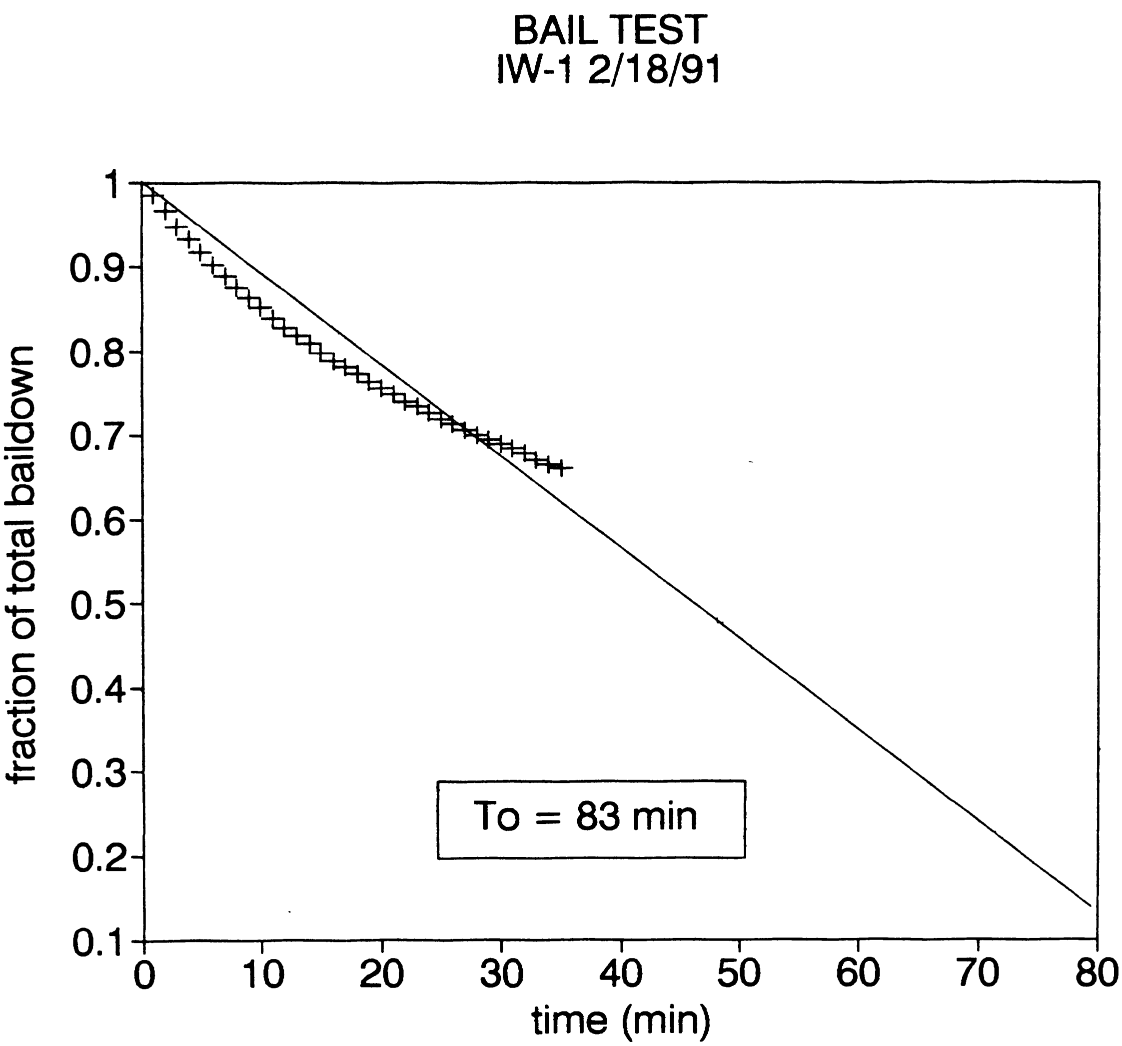

Figure 15. IW-1 bail test recovery vs. time and least squares fit

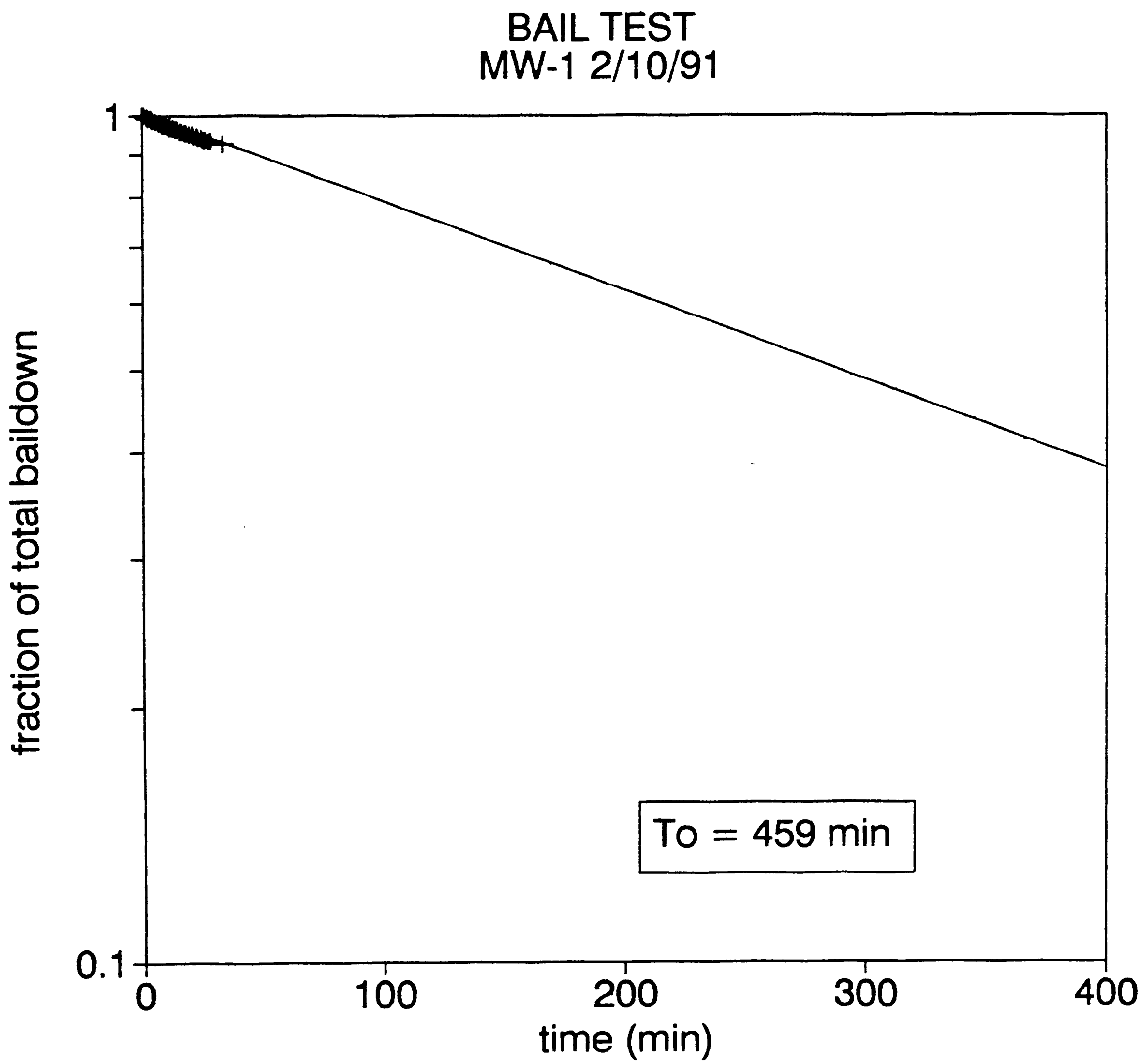

Figure 16. MW-1 bail test recovery vs. time and least squares fit

Table 4. Results of bail tests

Date	Well	static level (m)	bail-down level (m)	hydraulic conductivity cm/s
10-21-90	IW-1	1.61	`6.10	6.96×10^{-6}
11-4-90	IW-1	1.69	8.90	7.58×10^{-6}
2-18-91	IW-1	1.50	6.46	7.34×10^{-6}
10-21-90	MW-1	1.72	6.49	1.82×10^{-6}
11-4-90	MW-1	1.64	9.24	1.28×10^{-6}
2-10-90	MW-1	1.55	6.54	1.32×10^{-6}

*IW-1 & MW-1 are 12.07 meters (39.6 ft) deep, screened from 9.02 to 12.07 meters (29.6-39.6 ft) in the unoxidized till.

Table 5. Hydraulic conductivities of earlier slug tests performed at the study site.

Hydraulic conductivity (cm/s)	Depth (m)	Test	Investigator
1.55×10^{-8}	8.8	recovery	Everts et al.
3.88×10^{-9}	10.0	recovery	Everts et al.
4.55×10^{-9}	11.8	recovery	Everts et al.
3.12×10^{-8}	12.0	recovery	Everts et al.

SUMMARY AND DISCUSSION

The pumping tests were deemed successful as the level of response in the nearby monitoring well was much greater than anticipated considering the extremely low pumping rate. These constant-head pumping tests performed in the Wisconsin-aged unoxidized glacial till of Central Iowa produced higher hydraulic conductivity values than laboratory test methods on similar material. Pump test *1 estimated a transmissivity of 4.14×10^{-2} cm^2/s, a hydraulic conductivity of 2.16×10^{-6} cm/s, and a storativity value of $1.23 \times 10^{-5}\ cm^{-1}$ (Table 3). Grisak and Cherry (1975) reported a storativity of unoxidized till in Manitoba of $9.91 \times 10^{-5}\ cm^{-1}$ which is very close to the pump test values found in this study. The results from pump test *2 gave a transmissivity of $3.14 \times 10^{-2}\ cm^2/min$, a hydraulic conductivity of 1.64×10^{-6} cm/sec, and a storativity of $1.01 \times 10^{-5}\ cm^{-1}$. A summary of the pump test and bail test results are listed in Table 6. These permeability values are 1 to 2 orders of magnitude higher than most slug tests done on similar tills in Canada (Grisak and Cherry, 1975), New York (Prudic, 1982), and Illinois (Herzog,1989) as shown in Table 1.

The low hydraulic conductivity values of Canadian tills compared to those of Central Iowa may be due in part to climatic factors in Canada such as the shorter growing season and lower annual rainfall. Precipitation and vegetation play an important role in the weathering process, therefore an area receiving more rain may exhibit a more rapid weathering of its soils.

Table 6. Summary of pump test and bail test results in wells IW-1 and M-1 with mid-screen 10.55 m depth

Date	Well	Hydraulic conductivity (cm/s)	Storativity (cm^{-1})	Test
10-21-90	IW-1	6.96×10^{-6}	n.a.	bail
11-4-90	IW-1	7.58×10^{-6}	n.a.	bail
2-18-91	IW-1	7.34×10^{-6}	n.a.	bail
10-21-90	MW-1	1.82×10^{-6}	n.a.	bail
11-4-90	MW-1	1.28×10^{-6}	n.a.	bail
2-10-90	MW-1	1.32×10^{-6}	n.a.	bail
11-21-90	IW-1 & MW-1	2.16×10^{-6}	1.23×10^{-5}	pump
1-10-90	IW-1 & MW-1	1.64×10^{-6}	1.01×10^{-5}	pump

(n.a. = not applicable)

The disparity between laboratory values (Lutenegger, 1990) and the field values from this pump test is most likely due to the absence of sand lenses and fractures in the smaller laboratory sample units. One possible cause for the lower hydraulic conductivity values from the recovery tests performed by Kanwar et al. (1989) at this site is due to the use of short screens (61 cm), which were only 20% of the length of pump test wells IW-1 and MW-1.

Although samples noted during drilling revealed a heterogeneous aquifer material, the Theis method of aquifer analysis appeared to give reasonable approximations of hydraulic conductivities and storativity. The use of suitable portions of the drawdown data minimized differences from the assumptions. Pumping in very low permeability materials such as this unoxidized till, results in a small radius of influence. Differences between hydraulic conductivity values and storativity values of this site and others reviewed are related to: the depositional environment of the continental glacier, the mass of the ice atop the sediments, and the mineralogy of the sediments. Glacial tills of Canada and the northern U.S.A. tend to have higher levels of clay and silt than the tills of central Iowa which would explain the higher bulk hydraulic conductivity at this site.

The hydraulic conductivity values obtained from the pumping tests and bail tests are higher than other findings for unoxidized till at this depth. This difference may possibly be due to the high sand and low clay content and the possibility of the presence of fractures at field site 5. Since no fractures have been reported in the unoxidized till at the depth of these test wells, the most likely factors causing higher values of permeability are: small sand inclusions, the low clay content of the till, and the possibility that micro-fractures have gone undetected at the site.

Bail tests were in close agreement with the pumping tests, producing similar values of hydraulic conductivity. The use of in-situ testing for low permeability glacial tills through pumping tests and bail tests gives much more realistic values of the bulk hydraulic characteristics of the glacial till than laboratory tests.

RECOMMENDATION FOR FURTHER RESEARCH

The use of shelby tube samples for laboratory permeameter tests to supplement the pumping test and slug test data could give some indication whether there are secondary features affecting the bulk hydraulic conductivity. A larger nest of wells screened at various depths in the unoxidized till would allow for more comprehensive testing of the aquitard's characteristics. Introducing a tracer to a central well in a long-term test could reveal important flow patterns through the dense till. Tritium tests, which are effective for dating water less than 30 years old (Bradbury, 1991), may be applicable in the shallower portions of the unoxidized till to determine the recharge and flow characteristics. The unconfined aquifer in the oxidized till responds rapidly to a substantial rain event. Running a constant head pumping test in the unoxidized till over a period of time that includes a rain event may be of value by showing some communication between the two tills units.

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