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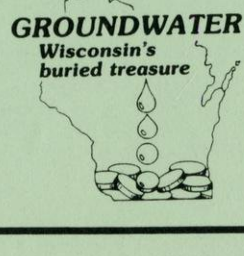

Wisconsin Department of Natural Resources

THE EFFECT OF CONSTRUCTION, INSTALLATION, AND DEVELOPMENT TECHNIQUES ON THE PERFORMANCE OF MONITORING WELLS IN FINE-GRAINED GLACIAL TILLS

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Final Report

Prepared for the Wisconsin Department of Natural Resources

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EXECUTIVE SUMMARY

The results of this study are summarized as: 1) the effect of piezometer construction, installation, and development techniques on water sample turbidity, and 2) the effect of piezometer installation and development techniques on calculated hydraulic conductivity.

Turbidity Results

Major conclusions concerning the effect of monitoring well construction, installation, and development practices on the turbidity of water samples obtained from monitoring wells installed in fine-grained glacial till can be summarized as follows:

1. The turbidity of water samples obtained from wells that were installed after water had begun filling the bottom of the borehole was 50 to 200 times greater than in samples from wells that were installed in essentially dry boreholes.

2. Monitoring wells that were surged produced water samples with 3 to 100 times greater turbidity than wells that were only bailed.

3. For the given sand pack material, there are no inherent differences in water turbidity obtained from monitoring wells finished with factory slot, factory slot with Mirafi™ wrap or continuous slot screens.

4. The turbidity of water samples obtained from surged wells did not show a significant decrease with the second sampling, but the turbidity of samples obtained from wells that were bailed-only decreased by a factor of 3.

5. Commonly available well screens and sand packs are not capable of filtering out clay-sized particles in fine-grained glacial tills. The optimal well design will require a silt-sized sand pack and a very fine-meshed screen (< 0.05 mm).

Hydraulic Conductivity Results

Major conclusions concerning the effect of monitoring well construction, and development practices on hydraulic conductivity calculated from bail-slug test on monitoring wells installed installed in fine-grained glacial till can be summarized as follows:

1. The hydraulic conductivity of the screen and sand pack material used in this study have been determined to be approximately four orders of magnitude greater than the hydraulic conductivities determined from bail-slug tests in the field. There fore, the screen and sand pack materials used in this study have no effect on the bail-slug test calculated hydraulic conductivity.

2. Near-surface damage of the augered holes was observed during piezometer installation. Turbidity results suggest that well development by surging is effective in removing formation material on the borehole wall and pulling the suspended material into the well annulus. Development by surging, therefore, should be effective in reducing any auger-induced skin effects. However, the hydraulic conductivity results of this study indicate that development by surging has no significant effect of reducing skin effects and increasing the hydraulic conductivity calculated from field tests over development by only bailing.

3. Bail test recovery data plotted as relative head versus time should yield a straight line on a semi-log plot. Deviations in the early portion of the total water level recovery versus time curve may suggest effects of sand pack dewatering or unsaturated recovery within the sand pack itself. Evaluation of a non-linear plot by any of the currently used bail tests solutions will produce hydraulic conductivities that are not representative of formation material.

4. Formation hydraulic conductivities as calculated by the slug-bail test solution of Bouwer and Rice (1976) are consistently 0.75 times those calculated by the method of Hvorslev (1951). In contrast, formation hydraulic conductivities as calculated by the method of Cooper, et al. (1967) - Papadopulos, et al. (1973) are approximately three times those calculated by the method of Hvorslev (1951).

5. The range of hydraulic conductivities as determined from bail-slug tests indicates a bimodal distribution of values for the gray silty-clay till of the Oak Creek Formation. Evidence for lenses of sand and/or gravel was not observed in turnings from the drilling process or in Shelby tube soil samples collected in the field. However, a fracture was observed in hand sample. This suggests that fracture flow may occur in some of the wells, resulting in higher measured hydraulic conductivities.

6. Hydraulic conductivity determined from time-consolidation data has been shown to give results which are within an order of magnitude of triaxial permeabilities. However, due to unavoidable operator error, instrument error, and error intrinsic in the graphical interpretation of time consolidation tests, hydraulic conductivity as determined in the triaxial cell produces conductivities which are more truly representative of the intergranular hydraulic conductivity of the formation.

7. The triaxial cell hydraulic conductivity results of this study agree within a factor of two of the lower formation hydraulic conductivities as calculated by the methods of Hvorslev (1951) and Bouwer and Rice (1976). However, intergranular triaxial cell hydraulic conductivities may not be representative of the bulk hydraulic conductivity of the formation as shown by the bimodality of slug-bail test results in this study. If large (order of magnitude) discrepancies occur between laboratory triaxial cell and slug-bail test hydraulic conductivities of a fine-grained formation material ($K < 10^{-6}$ cm/sec), primary structures such as sand and gravel stringers and secondary structures such as fractures should be thoroughly evaluated when considering the material for waste containment.

RECOMMENDATIONS

The overall recommendations regarding monitoring well construction, installation, and development techniques necessary to obtain representative formation hydraulic conductivities from bail tests and sediment free water samples based on the results of this study are as follows:

1. A non-surged factory slot piezometer set in an essentially dry borehole and packed with TDS2150 sand is the optimal monitoring well design which will result in bail test hydraulic conductivities which are representative of the formation conductivity and produce essentially turbidity-free water.

2. Surging has little influence on calculated hydraulic conductivity measured from bail tests, but increases water sample turbidity. The effect is an overall increase in the cost of a sampling program. Therefore, surging is not recommended as a development technique in fine-grained materials.

3. Turbidity results from this study have shown no differences between the various screen-filter combinations used and their ablility to keep suspended material out of the well annulus. Therefore, factory slot screen alone is recommended because it is the cheapest method of obtaining representative water samples and formation hydraulic conductivities.

4. A monitoring well which is set in an essentially "dry" borehole is best for reducing water sample turbidity. Setting the piezometer in a borehole partially filled with water allows water carrying suspended material into the well annulus prior to setting the sand pack. The result is sediment build-up in the bottom the piezometer. This sediment was observed to be brought into suspension especially when the bailer was allowed to strike the bottom. The use of a bailer to remove such sediment is not effective. Some surging of the water present in the monitoring well may be necessary to suspend the bottom sediment so that

it may be removed from the well annulus. The use of a sampling pump to remove bottom sediment may also be an effective means of removing sediment present in the bottom of piezometers.

5. Hydraulic conductivity as calculated by the methods of Hvorslev (1951) and Bouwer and Rice (1976) have been shown to be within a factor of two of the intergranular triaxial cell permeabilities. Both methods are recommended for the determination of aquitard hydraulic conductivity. If a monitoring well is installed in a homogeneous medium of infinite vertical extent, the analytical solution of Hvorslev (1951) should be used. If the monitoring well is partially penetrating, the analytical solution of Bouwer and Rice should be used. The method of Cooper, et al. (1967) - Papadopulos, et al. (1973) is not recommended for evaluating bail test hydraulic conductivities because the assumptions and boundary conditions for the aquifer solution are not representative of a well installed in an aquitard. This method produces hydraulic conductivities which are a factor of 3 times greater than those calculated by the methods of Hvorslev (1951) and Bouwer and Rice (1976).

6. Downward leakage due to a near-surface friction fitting connecting two lengths of PVC was observed in this study. To avoid this problem and any biases that may result from such leakage, it is recommended that joints connecting lengths of PVC standpipe be threaded and grouted. Bentonite pellets as a sand pack sealant in fine-grained materials should not be used to avoid problems that may develop because of bridging and wetting-up of the pellets.

THE EFFECT OF CONSTRUCTION, INSTALLATION, AND DEVELOPMENT TECHNIQUES ON THE PERFORMANCE OF MONITORING WELLS IN FINE-GRAINED GLACIAL TILLS

ABSTRACT

Twenty monitoring wells were installed in the fine-grained glacial of the Oak Creek Formation in southeastern Wisconsin to evaluate the effects of piezometer construction, installation, and development on the calculated formation hydraulic conductivity and well-water turbidity. The types of well screens used in construction of the piezometer were factory slot, factory slot with a filter wrap, continuous slot, and porous stone tips. Some of the wells were installed after the borehole began to fill with water while others were installed in essentially dry boreholes. About half of the wells were developed by surging while others were developed by only bailing.

Installation of monitoring wells in essentially dry boreholes produced water samples of very low turbidity compared to those wells which were installed in wet boreholes. Water samples of surged wells were much more turbid than the water samples from wells which were bailed-only. The type of screen and sand pack materials used had no effect on prohibiting suspended materials from entering the well annulus.

Hydraulic conductivity of surged wells increased less than a factor of two with repeated surging. The hydraulic conductivity of bailed-only wells also increased by a factor of two with repeated bailing. The type of screen and sand pack materials used had no effect on bail-slug test calculated hydraulic conductivity.

Formation hydraulic conductivities as calculated by the slug-bail test solution of Bouwer and Rice (1976) are consistently 0.75 times those calculated by the method of Hvorslev (1951). In contrast, formation hydraulic conductivities as calculated by the method of Cooper, et al. (1967) - Papadopulous, et al. (1973) are approximately three times those calculated by the method of Hvorslev (1951). Intergranular hydraulic conductivities of Shelby tube samples as determined in the triaxial cell are within a factor of two of the lower formation hydraulic conductivities as calculated by the method of Hvorslev (1951) and Bouwer and Rice (1976).

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INTRODUCTION

The protection of groundwater resources from contamination by hazardous and non-hazardous pollutants is a national concern. Better protection of groundwater quality and the public health is afforded through the installation of monitoring wells in the vicinity of waste disposal sites. These wells are used to measure the groundwater quality, the potentiometric surface within the aquifer and/or aquitard, and the hydraulic parameters of the material in which the well was completed. This information can then be used to predict the rate and direction at which contaminants are leaving a waste disposal site.

Many monitoring wells in Wisconsin are, or will be, completed within fine-grained glacial tills. Standard monitoring well construction and development procedures used in coarse-grained materials may not be adequate for fine-grained formations. The use of improper construction and development techniques within these tills can result in poor estimates of water quality and the hydraulic parameters of the formation. The suspension of large amounts of clay-sized particles in samples obtained from such wells may require extra filtering and bias analysis and interpretation of water chemistry. In addition, extra filtering requires time and adds to the overall cost of a sampling program.

There are many factors that may influence the computed hydraulic conductivity from slug tests performed on wells in fine-grained formations. These include the general well construction, the formation of a low hydraulic conductivity "skin" due to action of the auger, the amount of development the well has undergone, heterogeneities in the formation, and interpretation of the slug test data itself. The proper evaluation of these parameters will control the quality and reliability of information obtained from monitoring wells installed in fine-grained materials.

This study has three main objectives: 1) to evaluate currently used analytical techniques and assess the extent of piezometer development in relation to calculated field hydraulic conductivity obtained from slug tests on piezometers in fine-grained materials, 2) to examine the effect of piezometer construction, installation, and development techniques on well water turbidity of these piezometers and 3) to recommend an optimum installation procedure for obtaining a representative aquifer sample. Recommendations will be based on formation hydraulic conductivity and water sample turbidity. The ideal monitoring well should be constructed and developed in such a manner that hydraulic parameters obtained from slug tests on these wells represent the true values of the formation and water samples obtained from the wells are sediment free.

PREVIOUS STUDIES

Although field testing of hydraulic conductivity values is well-documented in the literature, the majority of solutions derived are specifically designed for measuring hydraulic conductivity of aquifer (coarse-grained) materials. The recent interest in contaminant hydrogeology and disposal of wastes in fine-grained materials now puts a major emphasis on the accurate determination of aquitard hydraulic conductivity. In the midwest, fine-grained glacial tills are the most readily available low permeability materials for waste disposal.

Field and laboratory hydraulic conductivity testing of fine-grained material include work done in fractured and non-fractured glacial till. Field or in-situ hydraulic conductivity is determined by slug tests. Laboratory hydraulic conductivity is determined by triaxial cell and consolidation tests.

Rehm, et al. (1980) reported a wide range of hydraulic conductivities (10^{-4} to 10^{-9} cm/s) for Quaternary pebble-loam till in the Northern Great Plains (North Dakota, Montana , and Wyoming). The distribution of hydraulic conductivities has several nodes and a mean value of 7×10^{-7} cm/s. Rehm, et al. (1980) observed fractures in mine high walls and in caves. The specific methods used for the determination of these values are not reported.

Desaulniers, et al., (1981) used slug tests and consolidation tests to determine the hydraulic conductivity of a clayey till and glaciolacustrine clay in southwestern Ontario. Hydraulic conductivity determined by field (1.7×10^{-8} cm/s) and laboratory ($2.7 - 2.9 \times 10^{-8}$ cm/s) tests indicate agreement within a factor of two. Intergranular conductivities of clay till and lacustrine clay typically range from 1.2×10^{-8} to 3×10^{-11} cm/s in the Interior Plains Region (Grisak, et al., 1976).

Grisak and Cherry (1975) conducted pump tests in a sandy aquifer overlain by a fractured lacustrine clay and clay-loam till at the Whiteshell Nuclear Research Establishment (WNRE) in southeastern Manitoba. The bulk hydraulic conductivity of the clay-loam till as determined by finite element mathematical modelling was 1.8×10^{-7} cm/s. Specific storage as determined by the method of Neuman and Witherspoon (1969a,b) was in the range of 3.0×10^{-5} to 1.5×10^{-5} m^{-1}.

Using the equation for flow in fractured media by Snow (1969) and fracture spacing values from test pits at WRNE, Grisak, et. al., (1976) calculated a hydraulic conductivity of 2.5×10^{-7} cm/s for the fractured clay-loam unit. This is very close to the digital model simulation for the same till unit at WNRE.

At a nuclear waste landfill in western New York, hydraulic conductivity of unfractured, fine-grained till was determined from slug tests, permeameter tests and mercury porisometer tests (Prudic, 1982). These different methods were found to agree within a factor of 25. Slug tests on augered-hole piezometers in the unfractured till had a mean hydraulic conductivity of 2 x 10^{-8} cm/s. However, slug tests of piezometers finished in the overlying fractured till ranged from 6 x 10^{-6} to 2 x 10^{-8} cm/s. This suggests that all piezometers in the fractured till did not intersect an equal number of fractures or the fractures may have been smeared during the drilling operation.

Hendry (1982) conducted a similar study of glacial till in Alberta. Intergranular conductivity of the weathered till is in the range of 3.4 x 10^{-8} to 9.8 x 10^{-8} cm/s as determined by falling head test in a consolidometer. Constant head tests of the fractured, weathered till yield conductivities in the range of 5.1 x 10^{-5} to 2.0 x 10^{-9} cm/s. Slug tests of 41 piezometers produced a bimodal distribution of hydraulic conductivity values ranging from 1.0 x 10^{-5} to 2.5 x 10^{-7} cm/s. The bimodal distribution was attributed to the presence of two different fracture patterns within the weathered till (Hendry, 1982).

In Saskatchewan, study of an unweathered glacial till has shown the field-derived permeabilities to be greater than laboratory time-consolidation permeabilities by two orders of magnitude (Keller, et al., 1986).

Tritium data for the unweathered unit indicate much higher recharge rates than calculated by laboratory-derived hydraulic conductivities. Therefore, fracture networks in the unweathered till have been used to explain differences between the bulk and intergranular permeabilities.

Quantitative studies of till in Wisconsin have been on the red clay till in the northwestern part of the state. Studies of the Miller Creek till have shown the upper unit of the till to be highly jointed (Bradbury, et al., 1985). Field hydraulic conductivities as determined by the method of Hvorslev (1.7×10^{-7} to 5.2×10^{-9} cm/s) indicate approximately one and a half orders of magnitude difference in the field hydraulic conductivities. However, no variation is indicated with depth. Laboratory triaxial cell determinations of hydraulic conductivity range from 7.0×10^{-7} to 1.2×10^{-8} cm/s. This is in good agreement with the field-derived values.

Studies of zone of saturation landfills in Wisconsin have shown field permeabilities to exceed laboratory derived values by two orders of magnitude (Gordon and Huebner, 1983). The presence of fractures and heterogeneities with the till units have been used to explain these differences.

However, drilling by the auger borehole method may cause smearing of fine grained materials on the well bore face and can conceivably cause the filling of secondary fractures in the unconsolidated material. The result is the formation of a low permeability "skin" on the bore face of the well which produces lower hydraulic conductivities than are truly representative of the fine-grained material (Faust and Mercer, 1984). This implies that full well development should be an important facet to reduce any effects of auger-induced bore hole smearing.

Previous studies of glacial till have presented and compared values of field and laboratory hydraulic conductivity. However, these studies have not fully addressed the effect of piezometer installation, construction, and development on potential bias in computed hydraulic conductivity or well water turbidity. Therefore, the scope of this study is to evaluate several methods of piezometer construction, installation and development in fine-grained glacial till and their relation to computed hydraulic conductivity and well turbidity.

METHODOLOGY

The objectives of this study have been accomplished with field work, laboratory work, and analytical work. Field work consisted of installing twenty piezometers of different design at two field sites in fine-grained glacial till. The wells were drilled using the auger boring technique. Numerous slug tests have been performed on these wells and sample turbidities analyzed. Evaluations have been made of piezometer construction, installation, and development techniques based on water samples obtained from these wells.

Piezometer construction refers to the type of materials used (i.e. screen type, filter wrap, and sand pack). Piezometer installation in the present study refers to the emplacement of sand pack material in "dry" versus "wet" bore holes and evaluation of the effects on well water turbidity. Finally, two types of development were studied: 1) surging and 2) bailing. The effect of this well development on the calculated hydraulic conductivity of consecutive slug tests performed on the wells and turbidity of well water samples is evaluated.

Laboratory tests were performed on the screens and sand packing materials used in piezometer construction to determine their permeabilities and porosities. Laboratory tests were also performed on samples collected in the field. Engineering soil tests were used to evaluate grain-size distribution and soil consistency of formation materials. In addition, time-consolidation tests and constant head tests in the triaxial cell were performed to determine the intergranular hydraulic conductivity of formation materials.

A critical evaluation of the currently used analytical solutions for determining hydraulic conductivity from slug test data in fine-grained materials was completed. The methods of Hvorslev (1951), Cooper, et al., (1967), Papadopulos, et al., (1973) and Bouwer and Rice (1976) were used to evaluate the slug test data in this study. Comparisons are made of computed hydraulic conductivity of the various analytical solutions results. Finally, comparisons are made between field and laboratory measurements of hydraulic conductivity and conclusions drawn as to the applicability of the results.

FIELD EQUIPMENT AND LABORATORY TESTS

Prior to installation, the screens and sand pack materials were tested in the laboratory to determine their permeabilities and porosities. Specifications provided by the screen manufacturing companies are not complete enough to derive screen hydraulic conductivities. For the factory and continuous slot screens, the companies have calculated tables for expected capacities of the screens (in gal/min/foot of screen) knowing the open area per foot of screen and assuming an entrance velocity of 0.1 ft/sec. Such specifications on the polypropylene filter wrap and porous tip piezometers are not available. Therefore, the screens and sand pack materials were tested in the laboratory to determine their permeabilties and porosities prior to installation.

Screens and Filter Wrap

Four types of monitoring well screens currently used in ground water monitoring investigations were evaluated in this study: 1) standard PVC factory slot, 2) factory slot with a Mirafi^{R} 140N polypropylene filter wrap, 3) PVC continuous slot, and 4) 50 micron porous tip piezometers.

The factory slot piezometers manufactured by Timco^{TM} Manufacturing, Inc., are the 2 inch diameter with 0.006 inch slot size. The 0.006 inch slot is the smallest manufactured slot size available. The polypropylene filter wrap is also manufactured by Timco™ Manufacturing. The wrap extends the entire length of a factory slot screen and is designed to allow water to flow into the screen while preventing fine particles from entering. The continuous slot screens, manufactured by Johnson Well Co., also have a 0.006 inch slot. The slot continuously spirals for the length of the screen and is supported on the inside by sixteen 1/8 inch wide rods. The screen is designed for high capacity flow while allowing minimal amounts of fine grained particles to enter the screen. The 50 micron porous tip piezometers, manufactured by Timco™, have a 1 1/2 inch outside diameter with a 3/4 inch standpipe.

As previously stated, the screens were tested in the laboratory to determine their hydraulic conductivities. The screens were tested using a constant head test (Fig. 1). A length of screen was connected to a corresponding diameter standpipe and placed inside a 12 inch diameter PVC reservoir. A garden hose was used along with an on-off valve to adjust the water flow to produce the head difference Δh. The water was allowed to freely flow over the top of the reservoir and the head difference Δh was taken as the distance between the top of the standpipe and the height of

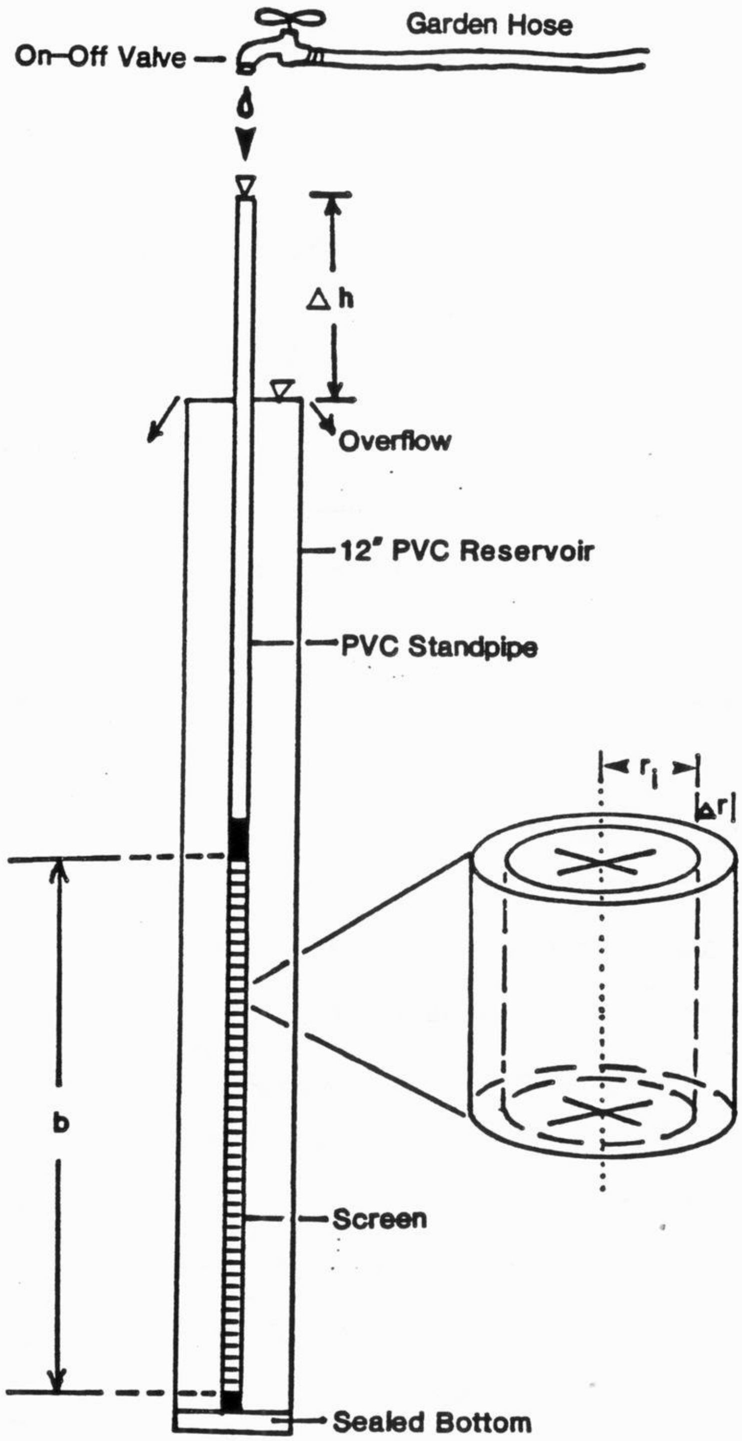

Figure 1. Cross-section of constant head apparatus to determine screen hydraulic conductivities.

overflow in the reservoir (Fig. 1) After the head difference was stabilized, the rate of flow was measured by transferring the garden hose to a calibrated 20 gallon barrel and measuring the amount of time it took to fill the barrel. Three trials were performed on each screen (Appendix I) and the flow rates (Q) averaged. Water temperatures taken during the tests (11.5 ± 0.1°C) are used in the calculations of the Reynolds' number.

From Darcy's equation for radial flow:

$$Q = -KIA, \tag{1}$$

where: Q = flow rate (cm^3/sec)
K = hydraulic conductivity (cm/sec)
I = gradient across the screen(cm/cm)
A = inside area of screen (cm^2)

From this the equation can be further expanded to represent our constant head set up as:

$$Q = -K(\Delta h/\Delta r)(2\pi r_i b) \tag{2}$$

where: Δh = head diference between the top of the standpipe and the reservoir overflow (cm)
Δr = sreeen thickness (cm)
r_i = standpipe radius (cm)
b = screen length (cm)

Equation 2 can be rearranged to form the desired relationship to estimate screen hydraulic conductivity (K):

$$K = -Q/[(\Delta h/\Delta r)(2\pi r_i b)] \tag{3}$$

where: Q = flow rate (cm^3/min)
Δh = head difference (cm)
Δr = screen thickness (cm)
r_i = inside radius (cm)
b = screen length (cm)

The results of the constant head tests show that the factory slot screen with the filter wrap is slightly more permeable than the factory slot alone (Table 1). This is probably due to small variations in the hydraulic conductivity of the factory slot piezometers used or it may be due to experimental error. The hydraulic conductivity of the filter wrap alone must be very high and consequently it has no effect of the hydraulic conductivity of the screen in our test. If the flow of the water had been toward the piezometer rather than out of the piezometer, the filter wrap may have been compressed and a lower hydraulic conductivity may have been obtained.

Secondly, the conductivity of the continuous slot screen is smaller than that of the factory slot screen. This was not expected. Investigation of the screen construction, revealed that there are sixteen 1/8 inch support rods extending the length of the screened interval. The relatively large number of rods within the two inch screen reduces the effective open area and hence the hydraulic conductivity of the screen.

A Reynolds number was calculated for each screen type (Table 2) to determine whether or not the screen testing procedure was conducted under conditions of laminar flow. The Reynold's number (R) is defined (Bear, 1979) as:

$$R = v\rho d/\mu \tag{4}$$

Table 1. Calculated screen hydraulic conductivities and constant head parameters.

Screen Type	Q	Q std. dev. (cm /s)	Δh	r_i	Δr	b	K
Factory Slot	330 cm^3/s	1.0	54 cm	2.4 cm	0.6 cm	143 cm	1.7E-3 cm/s
Factory Slot w/ filter wrap	340 cm^3/s	1.0	54 cm	2.4 cm	0.6 cm	143 cm	1.8E-3 cm/s
Continuous Slot	350 cm^3/s	8.1	54 cm	2.7 cm	0.25 cm	61 cm	1.5E-3 cm/s
Porous Piezometer	259 cm^3/s	6.0	67 cm	1.3 cm	0.13 cm	61 cm	5.6E-3 cm/s

* Standard deviation of flow rate (Q) for three trials.

LEGEND:

Q = flow rate (cm^3/s)
Δh = head difference (cm)
r_i = inside screen radius (cm)
Δr = screen thickness (cm)
b = screen length (cm)
K = hydraulic conductivity (cm/s)

Table 2. Reynolds numbers and parameters for the various screens.

Screen Type	ρ*	v	q	n	d	μ*	R
Factory Slot	0.99964 gm/ml	7.1 cm/s	.15 cm/s	.021	.0152 cm	1.253E-2 gm/cm sec	8.7
Factory Slot w/ filter wrap	0.99964 gm/ml	7.6 cm/s	.16 cm/s	.021	.0152 cm	1.253E-2 gm/cm sec	9.3
Continuous Slot	0.99964 gm/ml	8.5 cm/s	.33 cm/s	.039	.0152 cm	1.253E-2 gm/cm sec	10.
Porous Piezometer	0.99964 gm/ml	2.1 cm/s	.54 cm/s	.26	.005 cm	1.253E-2 gm/cm sec	0.83

* @ 11.5°C, Handbook of Physics and Chemistry

LEGEND:

ρ = density of water (gm/ml)
v = average linear velocity (cm/s)
q = Darcian velocity (cm/s)
n = porosity (cm^2/cm^2)
d = mean slot diameter (cm)
μ = dynamic viscosity of water (gm/cm sec)
R = Reynold's number

where ρ = density of water (gm/cm^3)
v = average linear velocity of water entering the slots (cm/sec)
d = mean diameter of slot (cm)
μ = dynamic viscosity of water (gm/cm sec)

A Reynolds number of 10 or less indicates laminar flow for a porous medium (Bear 1979). The average linear velocity (v) is calculated from the constant head derived Darcian velocity (q):

$$v = q/n \qquad (5)$$

where v = average linear velocity (cm/s)
q = Darcian velocity (cm/s)
n = porosity (cm^2/cm^2)

Porosities of the factory and continuous slot screens were calculated by dividing the open area of screen (per linear foot) by the total area (per linear foot) on the inside of the screens. Porosity of the porous piezometer was determined by a volume of water displacement taking into consideration the inner diameter, outer diameter, and length of wetted interval.

The Reynolds numbers derived (Table 2) from the screen testing procedure are within the range of laminar flow ($R \leq 10$). Therefore, it is argued that the hydraulic conductivities as calculated by Darcy's equation for radial flow are valid.

Sand Pack

Several considerations should be taken into account in choosing an appropriate sand pack material for the standard piezometer screens (continuous and factory slot). First, the sand should not enter the screen as the well is pumped. Ideally, ninety percent of the chosen sand pack material should be retained on the 0.006 inch (0.15 mm) slot size as recommended by Driscoll (1986). Secondly, the sand pack material should prevent suspended silt and clay size particles from entering the screen.

The sand used as packing material for the 0.006 inch factory and continuous slot screens is a dried and sieved beach sand (TDS2150) obtained from Lake Shore Sand Co. (Division of Construction Aggregate Corp of Michigan, Milwaukee, WI). The particle size distribution curve of the TDS2150 sand (Fig. 2) meets the requirement for keeping the sand from entering the screened portion of the well because the majority of it is greater than 0.15 mm in diameter.

TDS2150 is not a "pure" silica sand as noted in the chemical analysis of a typical sample (Table 3). It is not clears at this time how a sand of this composition will affect the chemical composition of a water sample passing through it to the screen. The

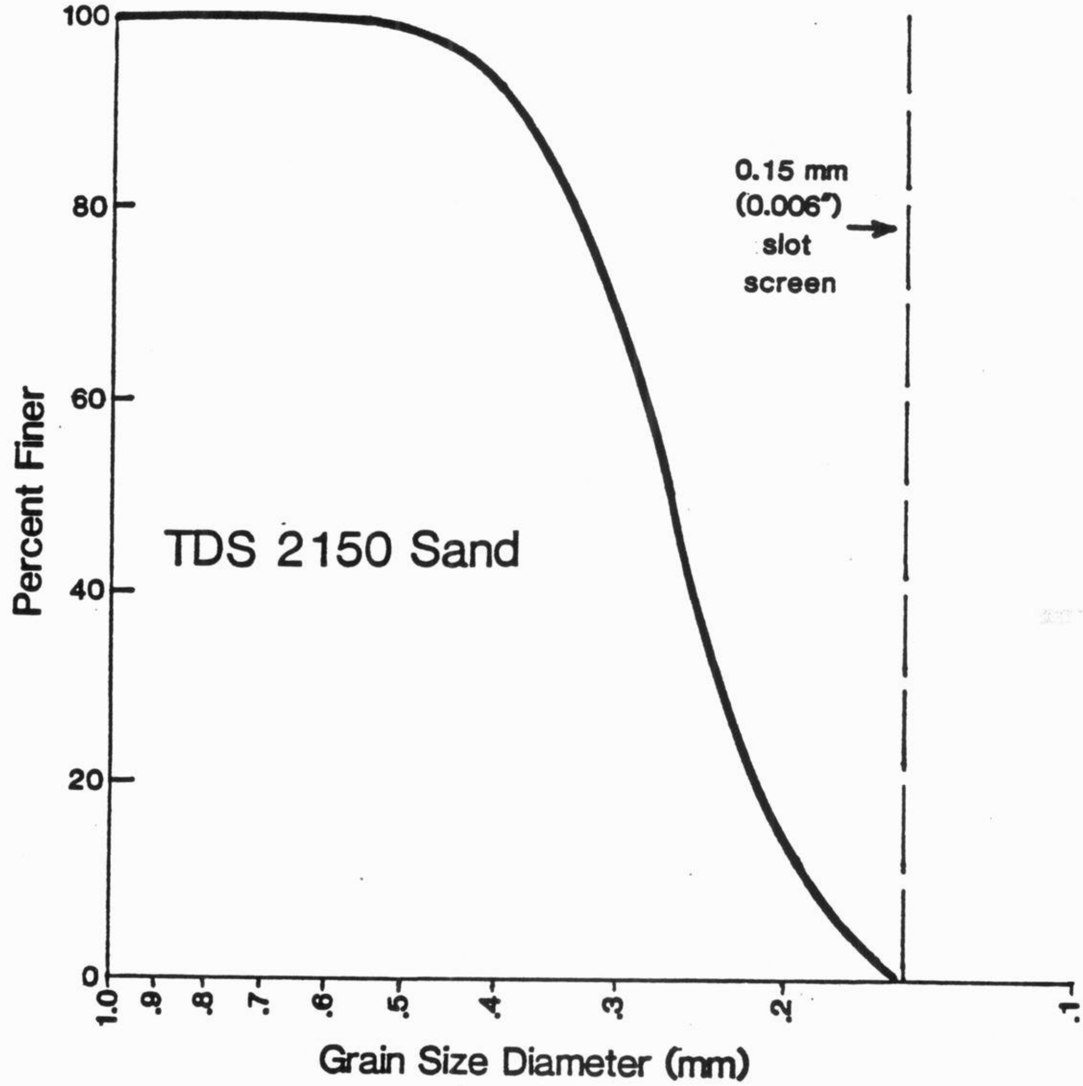

Figure 2. Grain-size distribution curve of the TDS2150 sand.

Table 3. Chemical analysis of TDS 2150 sand

Chemical	Percent by Weight
Silica	93.99
Aluminum Oxide	3.39
Iron Oxide	0.27
Calcium Oxide	0.14
Magnesium Oxide	0.10
Sodium Oxide	0.59
Potassium Oxide	1.18
Others	0.10
Loss on Ignition	0.24

Lake Shore Sand, Division of Construction Aggregates Corp. of Michigan, 515 W. Canal St., Milwaukee, WI 53203.

present study, however, is only concerned with hydraulic parameters and turbidity. Effects of the sand pack material on water sample chemistry are not evaluated.

The hydraulic conductivity of a typical sample of TDS2150 sand has been determined to be 2.3×10^{-2} cm/s using a constant head permeameter. The sand permeability was also calculated using the grain-size based approximations of Hazen (see Freeze and Cherry, 1979) and Masch and Denny (1966). The hydraulic conductivities derived from these two techniques are 3.6×10^{-2} cm/s and 2.1×10^{-2} cm/s, respectively. These values are in very good agreement with the laboratory-derived values.

A 74 micron (200 mesh) silica flour was used for the packing medium for the 50 micron porous tip piezometers. The silica flour has a mean diameter which is in the range of a fine silt. The hydraulic conductivity of the silica flour has been determined to be 4.9×10^{-3} cm/s by the method of Hazen (in Freeze and Cherry, 1979).

FIELD SITES

Two sites in Menomonee Falls, Wisconsin, have been selected and instrumented with various types of well screens. Site selection was based on auger borings from Layne Northwest, Inc., previous thesis work in the Menomonee Falls area (Neuman, 1982; Martin, 1982), and site accessability. Both field sites are owned by and located in the Village of Menomonee Falls (Fig. 3).

The general geology of the Menomonee Falls area consists of a red silty-clay till 5-20 feet thick overlying a gray silty-clay till unit ranging from 40 to 100 feet thick. Both the red silty-clay and gray silty-clay are till of the Oak Creek formation (Mickelson, pers. com., 1987). The Oak Creek Formation is classified as a strongly calcareous till which has an average compositionof 12 percent sand, 43 percent silt, and 45 percent clay (Mickelson, et al 1984). The gray silty-clay till unit unconformably overlies the Silurian Niagara Dolomite. The depth to bedrock at both field sites is between 60 and 80 feet (Neuman, 1982; Martin, 1982).

The red silty-clay till unit, which is also referred to as the red silt and clay (Martin, 1982), is heterogeneous and contains intermittent stringers of sand and gravel. The gray silty-clay till unit, which is also referred to as the blue silt and clay (Martin,

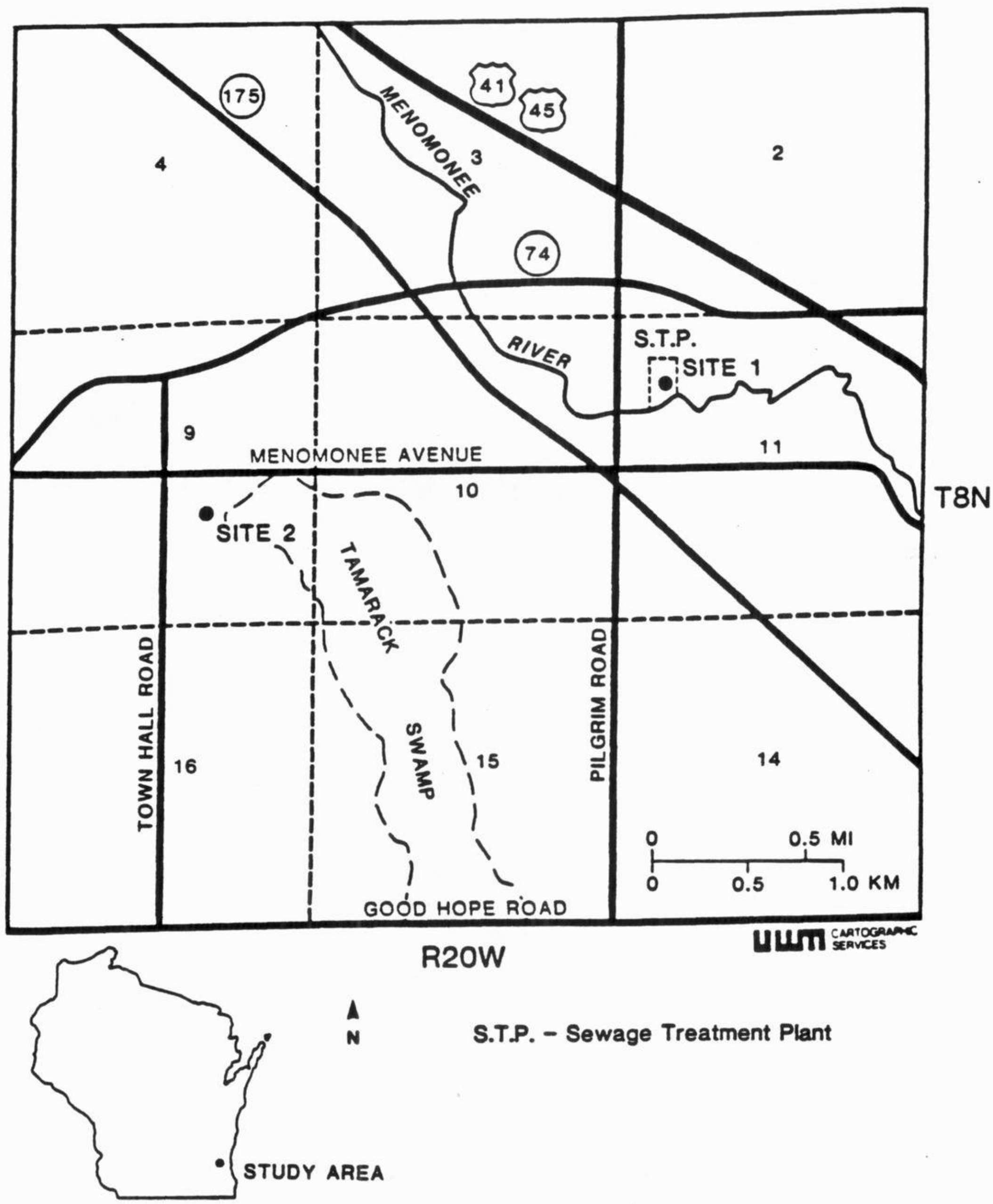

Figure 3. Location of field sites in Menomonee Falls, Wisconsin.

1982), is more extensive and thicker than the red silty-clay. The gray silty-clay does contain large lenses of sand and gravel that serve as water sources for the Village of Menomonee Falls (Neuman 1982).

Field Site 1

Field Site 1 (SE¼ ,NW¼ ,NW¼ , Sect. 11, T8N, R20E) is located along the Menomonee River within the confines of the inactive Menomonee Falls Sewage Treatment Plant (Fig. 3). The lithology consists of a red silty-clay till with many cobble and boulder-sized erratics overlying the gray silty-clay till (Fig. 4). The gray silty-clay till extends from a depth of 9 to 20 feet ending in a gravel at about 20 feet. The monitoring wells installed at this site were set and finished within the gray silty-clay unit.

Field Site 2

Field site 2 (SE¼, NW¼, SE, Sect. 9, T8N, R20 E) is located in a vacant field approximately 600 feet east of the intersection of Menomonee Avenue and Town Hall Road (Fig 3). The lithology at the site consists of a red silty-clay till overlying the gray silty-clay till unit (Fig. 4). The red silty-clay till extends to a depth of 11 feet and contains stringers of sand and gravel. The gray silty-clay till extends from 11 to 30 feet. Below 30 feet there is a sand and gravel

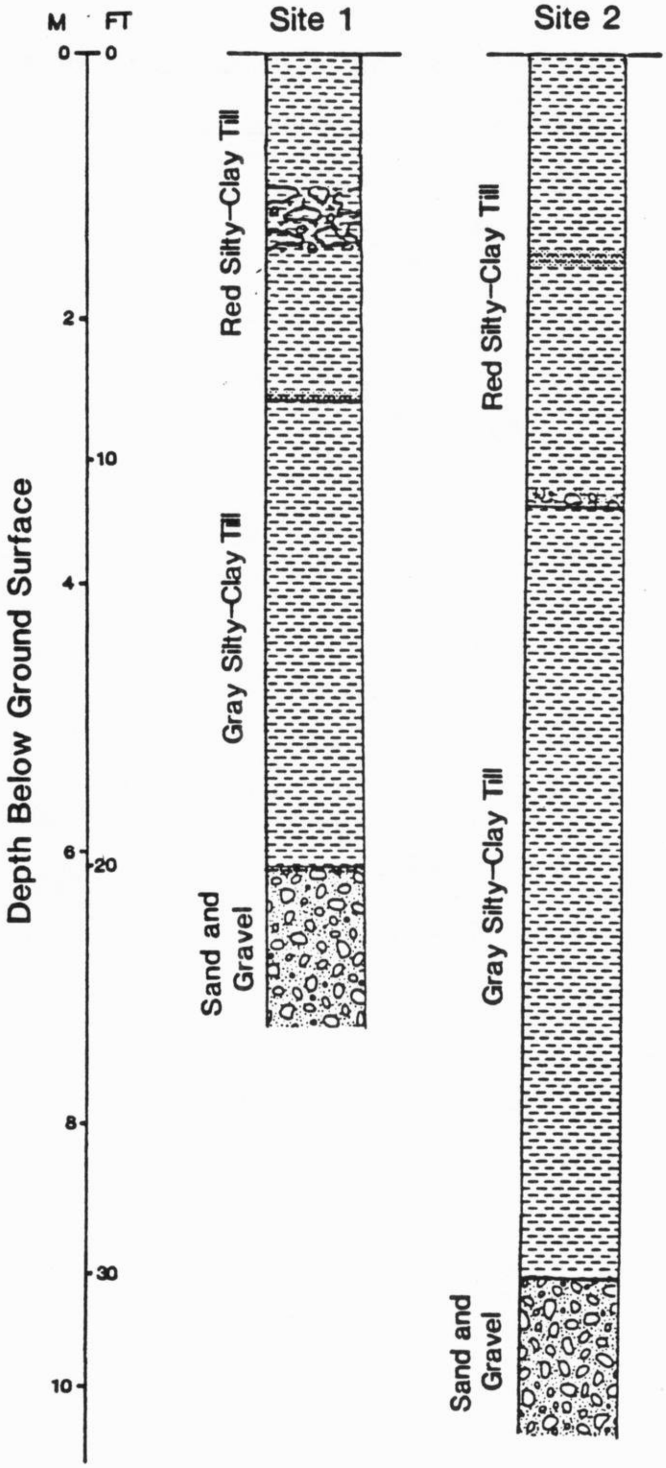

Figure 4. Logs of lithologies from exploratory boreholes at field sites 1 and 2.

aquifer. The depth to which this sand and gravel aquifer extends is not known but it is still present at 37 feet which was the deepest extent of drilling at this site. The monitoring wells were set and finished within the gray till unit.

MONITORING WELL INSTALLATION AND DEVELOPMENT

Installation Procedure

Monitoring well installation procedures used in this study followed those described by the Wisconsin Department of Natural Resources' "Guidelines for Monitoring Well Installation" (1985). The University of Wisconsin - Milwaukee's Model CME-45C drill rig equipped with 4 inch (10 cm) solid stem and 6 inch (15 cm) hollow stem auger was used to construct the piezometer boreholes. Initially, the 4 inch (10 cm) solid stem auger was to be used, but problems with near-surface caving hindered the ability to set the piezometer and sand pack properly. Therefore the 4 inch (10 cm) auger borehole method was abandoned in favor of the 6 inch (15 cm) auger-bored hole.

At each field site, an exploratory borehole was drilled to define the extent of the geologic units. A boring log of the each exploratory hole and all subsequent piezometer boreholes was made from turnings emanating from the borehole. The exploratory boreholes were back-filled and tamped with clay turnings.

For a typical borehole construction, the hole was augered to a depth of about 8 feet (2.4 m) in first gear in an attempt to create a stable annulus near the surface. This process seemed to subdue, but did not eliminate chattering of the auger when the borehole was finished in second gear.

When the depth to set the piezometer was reached, the auger was allowed to rotate until all the turnings were free of the hole. The auger was then retrieved and about 6 inches (15 cm) of fine sand was placed in the bottom of the borehole. A 2 inch (5 cm) diameter piezometer was lowered onto the sand and centered in the hole by eye, using a light source from the surface. Additional sand pack material was placed around the piezometer screen.

Initially, an attempt was made to use a 0.75 inch (2 cm) inside diameter CPVC electrical conduit with a funnel at the surface to install the sand pack. Problems soon developed with the fine sand clogging in the conduit, especially when the end became wet. Use of the electrical conduit was abandoned in favor of simply pouring the sand down the hole. The relatively shallow depths at which the piezometers were set made this feasible.

The top of the sand pack was set at about 3 feet (1 m) above the top of the screen. This sand pack was tamped down and a 1 to 2 foot (0.5 m) layer of Voclay bentonite pellets was placed on top. Clay turnings were placed on top of the bentonite and rigorously tamped with a plugged 0.75 inch (2 cm) diameter electrical pipe. The holes were filled and tamped to the surface.

During the installation of the sand pack and monitoring well, the following measurements were taken: the depth to the bottom of the auger hole, the depth to which the piezometer was set, the depth to the top of the sand pack, and the depth to the top of the bentonite seal. All measurements were taken relative to the top of the borehole. Measurements of the total piezometer length screen length, and casing length for each piezometer were made prior to installation. These measurements are recorded in Appendix II.

Field Site 1

Borehole preparation at site 1 was accomplished by drilling to within 2 to 3 feet (0.7 m) of the underlying sand and gravel aquifer. The auger was rotated until all the turnings were free of the hole. The auger was then retrieved, and the piezometer set in the manner described above.

At site 1, six piezometers of similar design were installed (Fig. 5). These wells have a horizontal spacing of 10 to 12 feet (3-4 m). The sand packs for the first piezometers installed (1-1 and 1-4) at this site were set as a sand-clay matrix which was inadvertantly produced by delays in setting the piezometer. During this time interval, the borehole would partially fill with water and clay suspended in the borehole

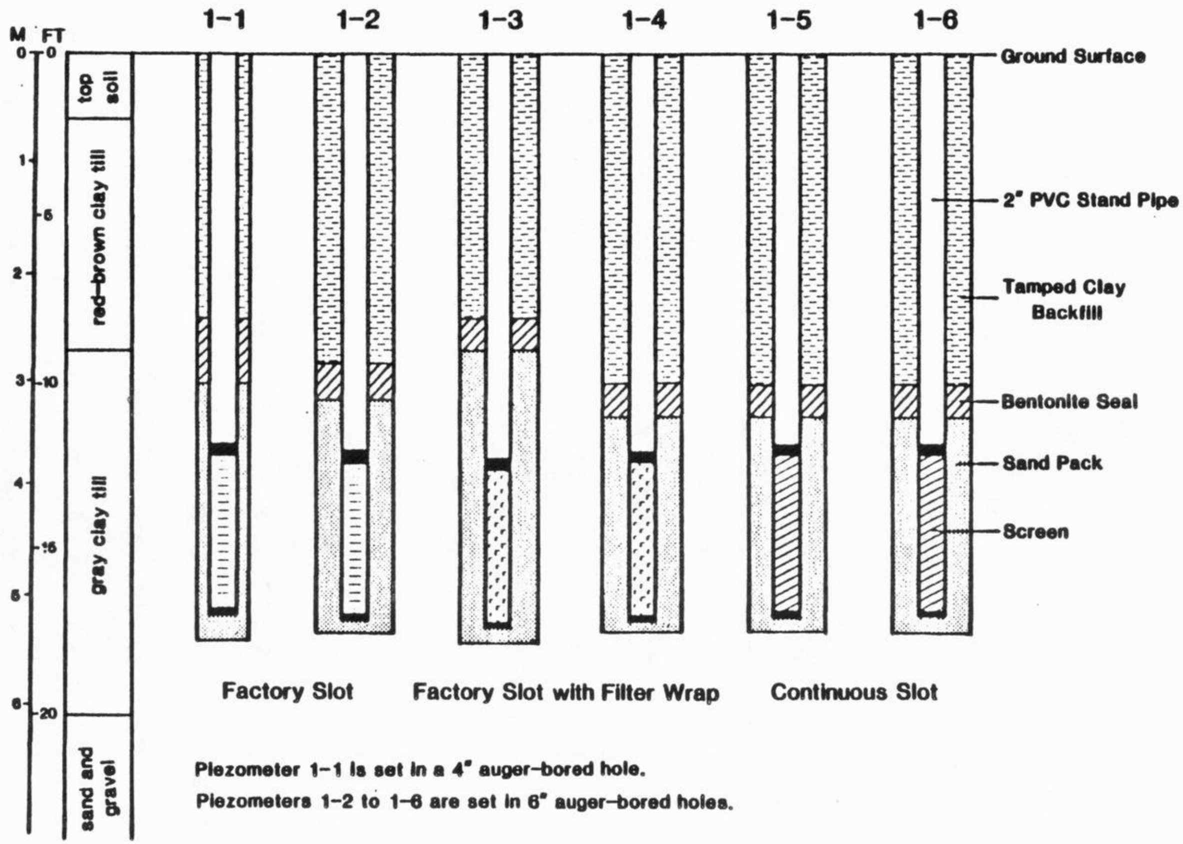

Figure 5. Cross-section of the general piezometer construction at site 1. Piezometers 1-1 to 1-6.

water became entrapped as the sand pack was set.

The last two piezometers (1-5 and 1-6) installed at this site were set in essentially dry sand. This was accomplished by setting the piezometer immediately after the auger was removed and pouring the sand from the surface. The piezometers were then finished in the manner described above.

Field Site 2

Borehole preparation at site 2 was accomplished by drilling to approximately 22 feet (7 m) and setting the piezometers within the gray clay till. At this site, two lines of six piezometers were installed with similar designs (Fig. 6). These wells also have a horizontal spacing of 10 to 12 feet (3-4 m). Again the piezometer sand packs were installed essentially dry by pouring the pack material from the surface immediately after the auger was removed from the hole.

Two porous tip piezometers (Fig. 6) have also been installed adjacent to the previous two piezometer arrays at site 2. Because the silica flour used to pack the porous tip piezometers is very fine and easily airborne, a 20 foot (6 m) length of 2 inch (5 cm) PVC pipe, supported at the surface, and a funnel were used to place the silica flour at the bottom of the hole. The 2 inch (5 cm) PVC served as a guide for setting the piezometer, and placing the rest of the pack over the

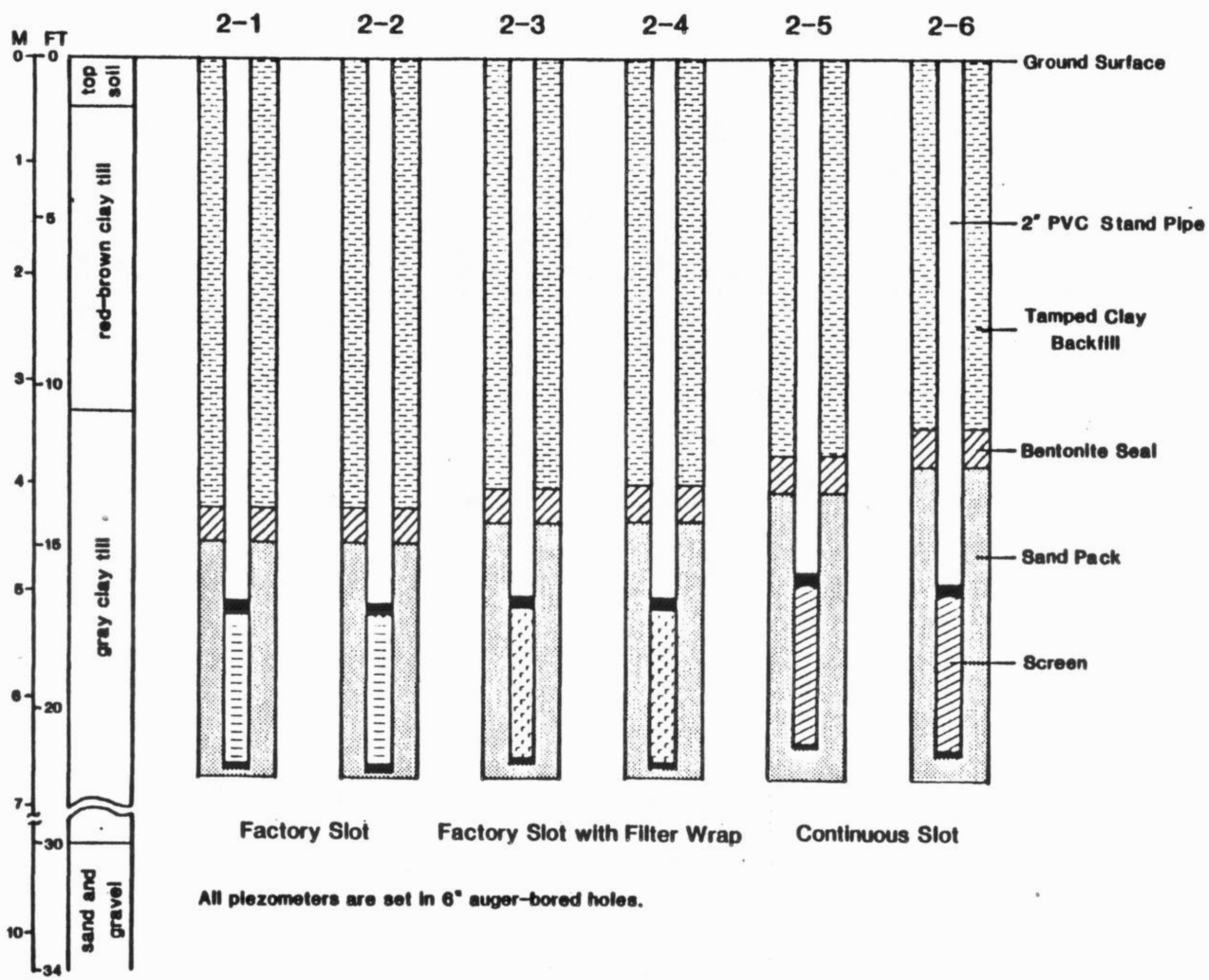

Figure 6. Cross-section of the general piezometer construction at site 2. Piezometers 2-1 to 2-6.

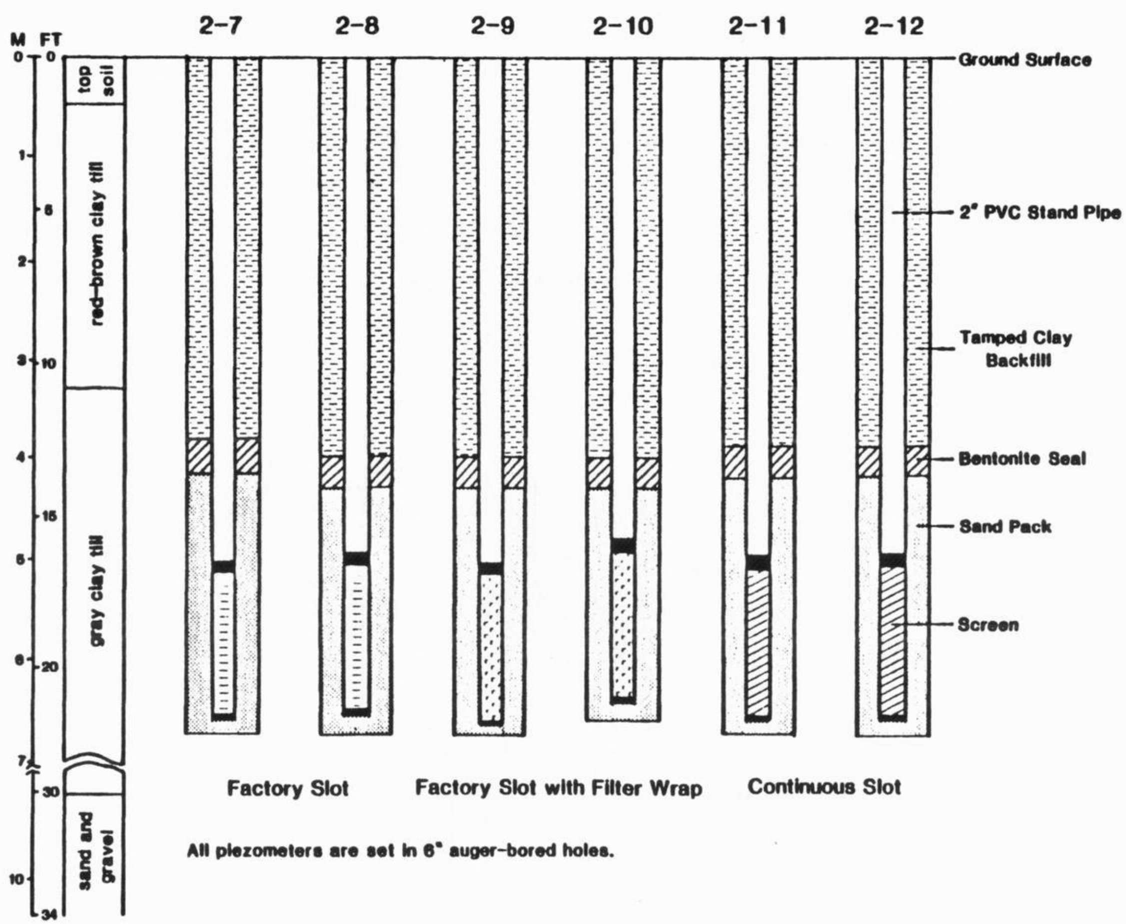

Figure 6. (Continued) Piezometers 2-7 to 2-12.

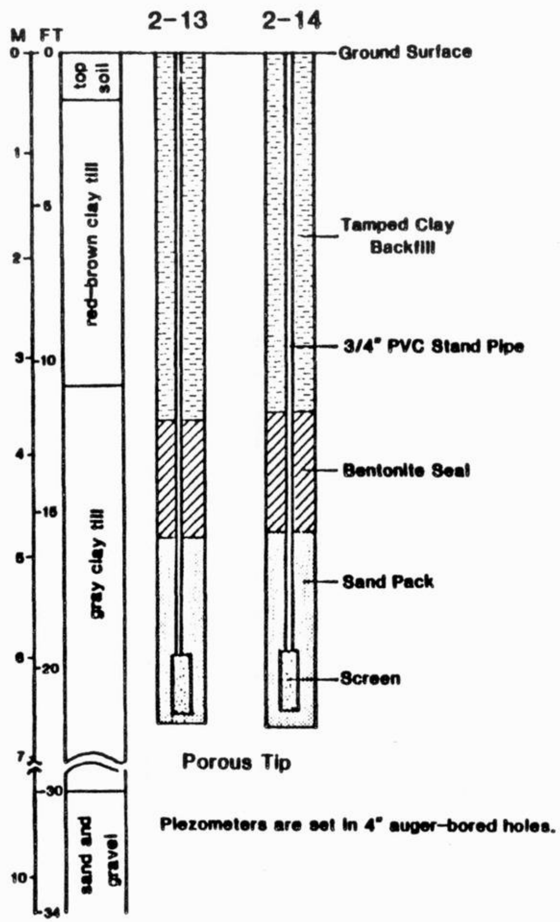

Figure 6. (Continued) Piezometers 2-13 and 2-14.

sealed 0.75 inch (2 cm) pipe. The bentonite was poured from the surface and turnings were placed on top and tamped as described in the previous section. The wells at site 2 were finished with protective metal casings and locks to prevent vandalism.

Soil Sampling Field Procedure

Soil samples were collected over the screened interval at each site using the Shelby-tube method (Driscoll, 1986). The general procedure was to auger the hole to the sampling depth and retrieve the auger. A 3 inch (7.6 cm) diameter Shelby-tube was lowered to the bottom of the hole and hydraulically pushed into the soil. After advancement of the Shelby-tube ceased, the tube and its contents were hydraulically extracted from the borehole. The sample depth and amount of recovery were recorded for each sample. This section of the hole was then augered before the installation of the sand pack and monitoring well.

Samples were collected at each site. However, samples were not collected in every bore hole constructed. At site 1, samples were taken at: 1) a depth of 9 feet in borehole 1-1, 2) a depth of 12 feet in borehole 1-3, and 3) a depth of 14 feet in borehole 1-4. At site 2, samples were taken at: 1) depths of 16, 19, and 21 feet in borehole 2-6, 2) depths of 18 and 20 feet in borehole 2-13, and 3) a depth of 19.5 feet in borehole 2-14.

The samples were field wrapped by sealing the ends of the tubes with plastic caps. The ends were heavily taped and the tubes placed in plastic bags. The samples were later extruded with a hydraulic sample extruder and enclosed in a series of freezer bags to prevent dessication of the sample prior to laboratory testing.

Monitoring Well Development

An important facet of the well construction procedure is the well development process. Common methods of well development are surging, bailing, and the use of compressed air to blow out the well. In all cases, the goal of the development process is to assure that the hydraulic conductivity obtained from a slug test is diagnostic of the formation which is being tested.

In fine-grained materials, problems that may develop in the well installation/construction procedure are numerous. In the initial construction of the borehole, the action of the auger can conceivably create a low permeability "skin" on the borehole face or smear in any fractures present. This auger-induced smearing will produce observed hydraulic conductivities that are lower than the actual formation conductivity. Another problem that may occur is the smearing of fine sediment into the slotted portion of the screen when setting the piezometer. This again, if allowed to happen, can conceivably cause the observation of lower permeability than actually diagnostic of the formation. If the sand pack material is set essentially dry or even partially saturated, there is the potential for entrapped air within the pack material. The result of this will be to decrease the observed hydraulic conductivity because the monitoring point is not truly serving as a flow-through system.

The purpose of well development should be to eliminate these problems associated with the borehole construction and well installation practices. In this study, two methods of well development have been analyzed: 1) surging and 2) bailing. In each case for site 1 and site 2, the first well in each well doublet or the odd number wells (i.e. 1-1,1-3, 2-1,2-3...) were developed by surging. In contrast, the even numbered wells were developed by bailing.

The surging process was performed with a length of 0.75 inch (2 cm) inside diameter PVC electrical pipe fitted with a rubber stopper (Fig. 7). For each sampling event, the device shown in Figure 7 was forced up and down within the screened portion of the well for 10 minutes. The rubber stopper was small enough to allow passage of the surging device through the 2 inch piezometer standpipe but large enough such that a good surging action was attained. Ideally, the

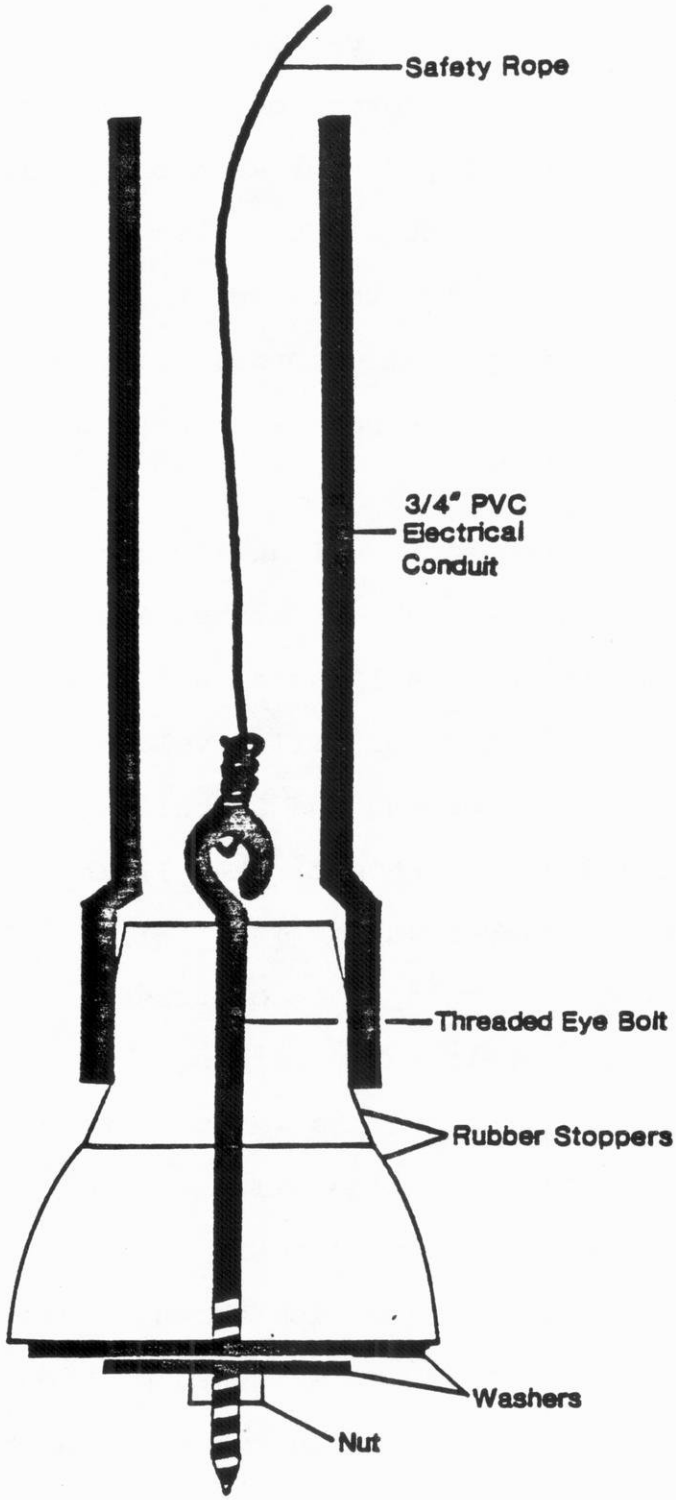

Figure 7. Schematic diagram of rubber stopper surging apparatus used for well development.

surging action should force piezometer water through the well screen and sand pack, causing the dissipation of air bubbles entrapped in the piezometer and sand pack. The surging should also reduce any adverse effects of fine-grained sediment smeared in the slotted portion of the well screen or on the well bore face.

Both the surged and non-surged wells were bailed for each sampling event. Water samples collected from the screened portion of the well were analyzed for turbidity.

TURBIDITY

The purpose of the sand pack, filter wrap, and screen are to keep the geologic formation materials from entering the well. If clay-sized particles were to enter the well and remain in suspension when the well is sampled, considerable filtering of the sample would be required. This filtering can add a considerable amount of time and expense to a sampling program. In some cases, the sample may be rejected by a regulatory agency if the turbidity is too great. It is therefore advantageous to be aware of any practices that can reduce the amount of sediment in the well. A turbidity test was performed on bailed samples from each of the monitoring wells as a measure of the suspended material entering the well.

Method of Analysis

Turbidity was measured using a Bausch and Lomb Mini 20 Nephelometer attachment on a Bausch and Lomb Mini Spectronic 20 spectrophotometer. This instrument has three operating scales of 0-1, 0-10, and 0-100 Nephelometric Turbidity Units (NTU) with a scale divisions of 0.02, 0.2, and 2 NTU, respectively. The accuracy on these three scales are ±0.1, ±0.7, and ±5 NTU, respectively (Bausch and Lomb, 1980).

The day before a field test, the instrument calibration was checked using Formazin Standards prepared in a manner described by Baush and Lomb (1980) and recommended by the U.S. Environmental Protection Agency (Baush and Lomb, 1980). The Nephelometer Reference Standards provided by the manufacturer where checked and found to be within the instrument error of their reported values. Because of the short stability time of the formazin standards (1 hour for a 0.8 NTU standard), the Manufacturer's Nephelometer Reference Standards were used to check and recalibrate the instrument in the field. Instrument calibration was checked before sampling began and before each sample was measured.

Many of these samples contained a considerable amount of sediment and required dilution to bring the turbidity into the range of the nephelometer. The samples were mixed with distilled water with a turbidity of 0.3 NTU. The sample turbidity was calculated by subtracting the background turbidity value from the instrument reading and multiplying by a dilution factor.

Turbidity Results

On 25 May, 1986, the wells at site 1 were developed and bailed. One of the last bails of water was used as a sample for the turbidity analysis. The initial turbidity of the samples at site 1 ranged from 71 to

Table 4. Turbidity results at site 1 and relation to screen type, development technique and method of installation.

WELL NO.	5/25/86 TURBIDITY (NTU)	SCREEN* TYPE	SURGING	INSTALLATION	9/23/86 TURBIDITY (NTU)
1-1	18,000	FS	YES	WET	23,000
1-2	5,000	FS	NO	WET	520
1-3	230,000	FSWMW	YES	WET	25,000
1-4	5,600	FSWMW	NO	WET	510
1-5	1,500	CS	YES	DRY	740
1-6	71	CS	NO	DRY	14

*FS = Factory Slot
FSWMW = Factory Slot with Mirafi Wrap
CS = Continuous Slot

230,000 NTU (Table 4). The highest tubidity was obtained from well 1-3. The initial bailing of wells 1-2 and 1-4 indicated a large amount of clayey sediment in the bottoms of these piezometers. The bottom sediment was easily agitated and brought into suspension especially when the bailer was allowed to touch the bottom of the well. In an attempt to decrease the clay sediment in the wells, a peristaltic sampling pump and 12 volt battery supply at the surface were used on September 6, 1986, to remove the bottom sediment from all the piezometers at site 1. (Well 1-3, which had very high initial turbidity, was heavily built up with sediment and the tygon tubing used to remove the sediment had to be unplugged several times.) After the wells were pumped "dry" , they were allowed to flow back to static equilibrium.

The wells were developed and bailed again on September 23, 1986. For this sampling, the sample turbidity ranged from 14 to 25,000 NTU (Table 4). Again the highest turbidity was in well 1-3, however, the well showed an order of magnitude decrease from the initial turbidity reading.

On September 4, 1986, the wells at site 2 were developed and bailed. The wells at this site were all set in essentially dry holes. Initial turbidity if the samples ranged from 9 to 2400 NTU (Table 5). The

Table 5. Turbidity results at site 1 and relation to screen type, development technique and method of installation.

WELL NO.	9/4/86 TURBIDITY (NTU)	SCREEN* TYPE	SURGING	9/23/86 TURBIDITY (NTU)
2-1	1900	FS	YES	1400
2-2	28	FS	NO	5
2-3	600	FSWMW	YES	1100
2-4	20	FSWMW	NO	6
2-5	1400	CS	YES	780
2-6	12	CS	NO	3
2-7	900	FS	YES	2100
2-8	13	FS	NO	4
2-9	2000	FSWMW	YES	880
2-10	14	FSWMW	NO	8
2-11	1300	CS	YES	1400
2-12	11	CS	NO	6
2-13	2400	PS	NO	520
2-14	9	PS	NO	30

*FS = FACTORY SLOT
FSWMW = FACTORY SLOT with MIRAFI WRAP
CS = CONTINUOUS SLOT
PS = POROUS STONE

highest and lowest turbidities were measured in the porous stone piezometers (2-13 and 2-14). The turbidity of the porous stone water samples had a milky white appearance indicating that the silica flour was passing through the porous stone. In contrast, the turbidity of the standard factory and continuous slot water samples was brown in appearance suggesting that the surging process was developing the borehole wall by pulling formation material through the sand pack and into the screen. The wells at site 2 were developed and bailed again on September 23, 1986. The results were similar to those obtained from the first sampling (Table 5).

The results of the turbidity measurements for sites 1 and 2 are graphically summarized in Figures 8 and 9, respectively. The four monitoring wells that were installed wet have substantially greater turbidity than wells that were intalled dry (Fig. 8). If monitoring wells that were developed in the same manner are compared, the wells that were installed wet generally have 50 to 200 times greater turbidity than wells that were installed dry (Fig. 8).

Surging of the wells increases the turbidity. Surged wells that were installed wet have 3 to 50 times greater turbidity than wells that were not surged. Wells that were installed dry and were surged

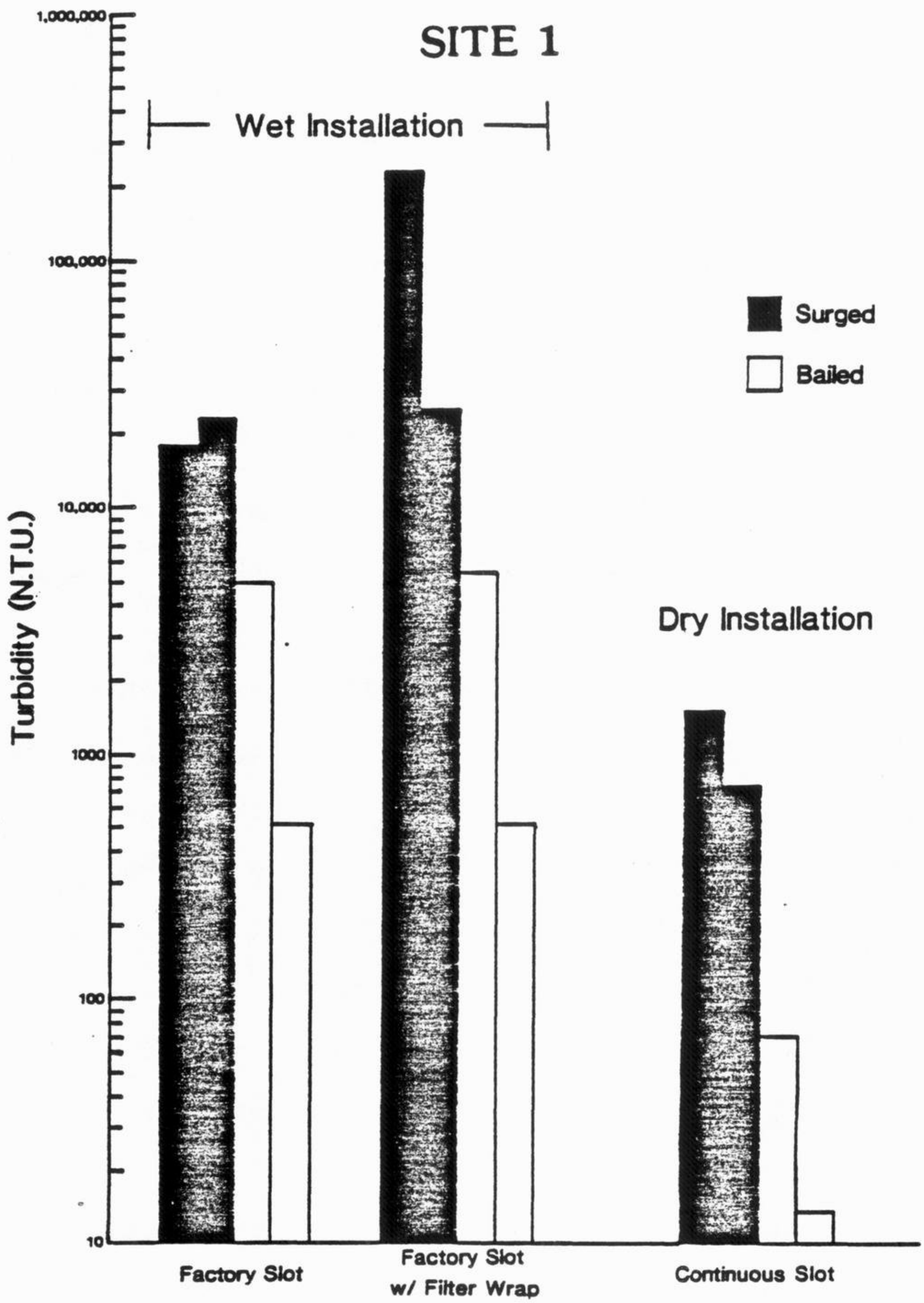

Figure 8. Turbidity results from site 1.

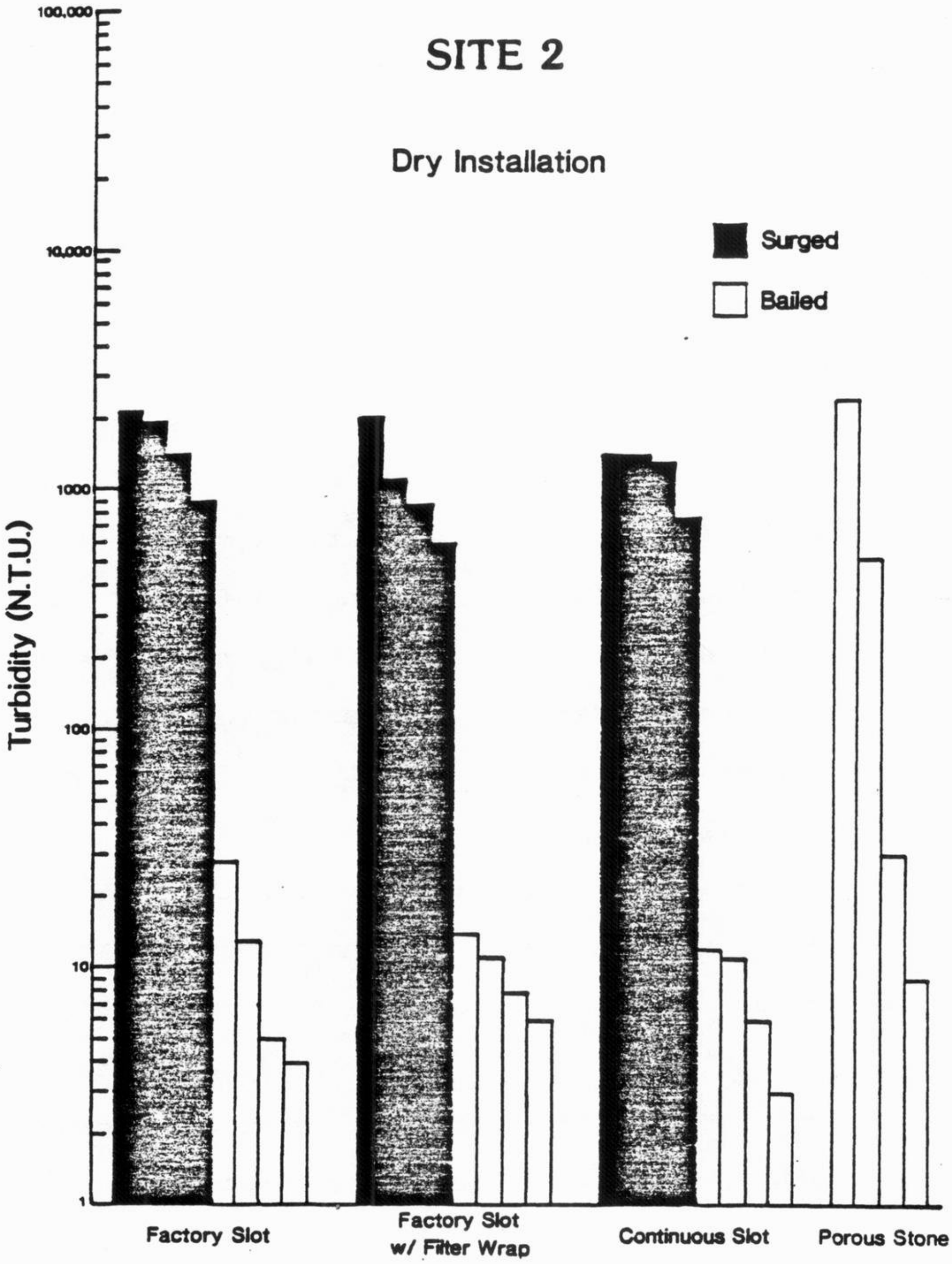

Figure 9. Turbidity results from site 2.

have about 100 times greater turbidity than wells that were installed dry and were not surged. Wells that were installed wet and were surged have about 3000 times more turbidity than wells that were installed dry and were not surged.

Well construction (i.e. sand pack - screen combinations) had little effect on the amount of turbidity in the well (Fig. 9). There are small differences in the turbidity for the different monitoring well screens for the surged wells. The factory slot wells (FS) have 1.4 to 2.5 times more sediment than continuous slot (CS) screens and factory slot screens with the Mirafi wrap (FSWMW), repectively. There are no substantial differences in the turbidity measurements between the three types of well screens for the bailed wells.

The change in the turbidity with the second bailing varied. A measure of the amount of improvement in the turbidity in the monitoring wells is the ratio of the turbidity in the samples collected in the second testing of the wells (9/23/86 for site 1 and site 2) to the turbidity obtained during the first test on the wells (5/25/86 at site 1 and 9/4/86 at site 2). If this ratio is greater than unity, the turbidity in the well increased. If the ratio is equal to unity, there is no improvement. If the ratio is less than unity, the turbidity decreased and the quality of the sample obtained from the well has improved. A comparison of the surged and the bailed wells (Fig. 10) indicates that, in general, the turbidity ratio in the surged wells was greater than the turbidity in the bailed wells. The average ratio obtained for the surged wells was 1.08, indicating no improvement in the turbidity. The average ratio for the bailed wells was 0.37 indicating about a three-fold decrease in turbidity which is a substantial improvement in the quality of the sample.

Discussion

The high turbidity values obtained for the water samples collected from the monitoring wells are not unusual for wells installed in fine-grained materials. The reason for these high turbidity values is the choice of sand pack. The function of the sand pack is to stabilize the borehole and to prevent formation materials from entering the well. Usually the sand pack is chosen to retain the formation and then a appropriate screen is chosen to retain the sand pack. However, in fine-grained glacial tills, one is constrained by the choice of screens. The smallest commercially available screen slot size is 0.006 inches (0.15 mm). Given this slot size, the choice of sand pack is based simply on the ability of this screen to

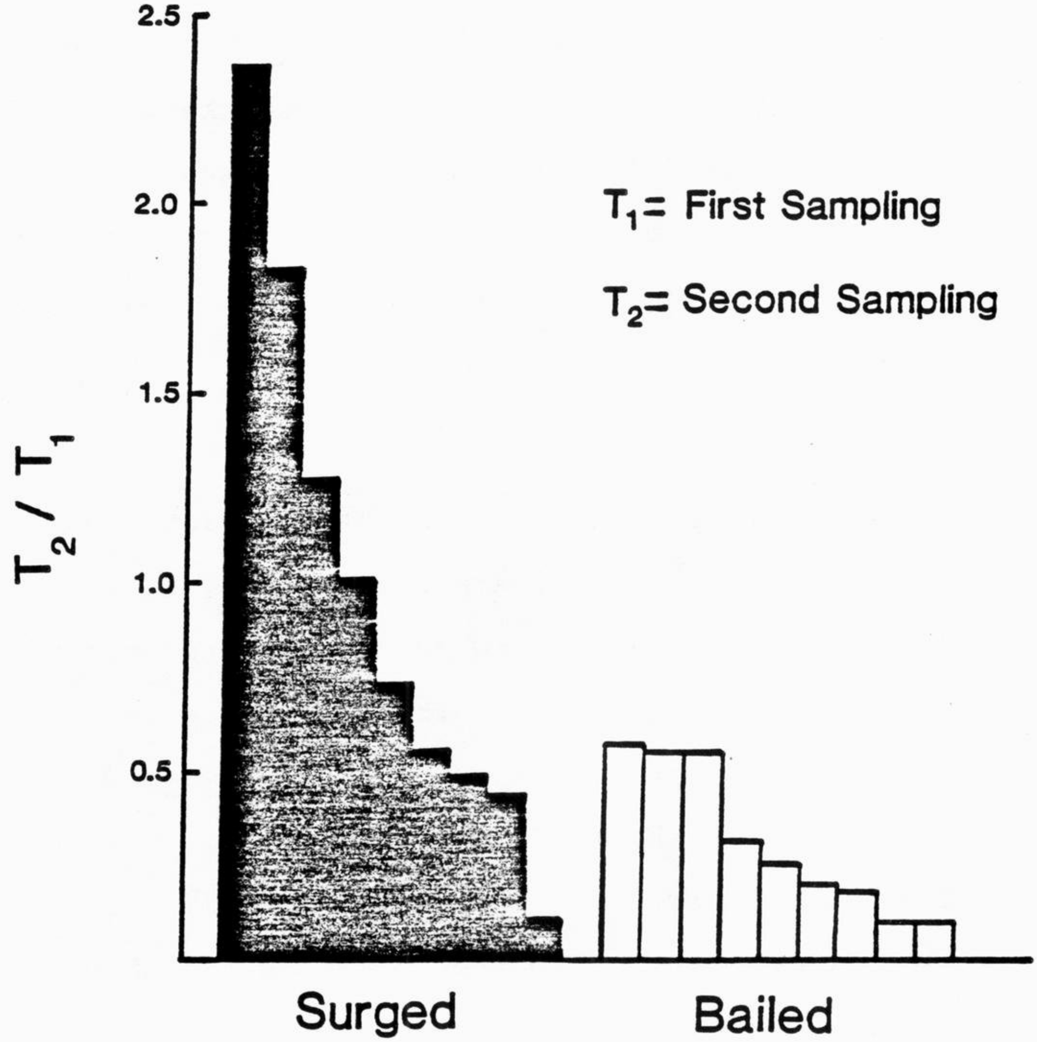

Figure 10. Ratio of turbidity from second sampling to first sampling. Data from sites 1 and 2 are combined.

retain it.

The proper size of sand pack and screen for a monitoring well can be determined from the grain-size distribution curve of the formation and applying the method outlined by Driscoll (1986). A composite sieve-hydrometer analysis grain-size distribution curve for a typical sample of the silty-clay in this study (Fig. 11) indicates a d_{70} (diameter that 70 percent of the formation is greater than) of 0.002 mm. The d_{70} of the filter pack should be 4 to 6 times this value (Driscoll, 1986) or between 0.008 and 0.012 mm. If the sand pack has a uniformity coefficient of 2.4, the sand pack should have a mean grain diameter (d_{50}) of 0.010 to 0.017 mm. Therefore, the sand pack should be a silt (0.002 mm $< d_{50} <$ 0.05 mm). With this ideal sand pack, a screen with a slot size less than or equal to the d_{90} of the sand pack or 0.0055 mm (0.0002 inch) to 0.008 mm (0.0003 inch) should be used. The smallest commercially available screen size, however, is 0.015 mm (0.006 inch). Thus, in practice, this screen size is used and the choice of the sand pack is based simply on the ability of the screen to retain it.

Another possible screen type is the porous piezometer screen. This screen is commercially available and has 0.050 mm openings. If a properly sized-filter pack would be obtained for this porous

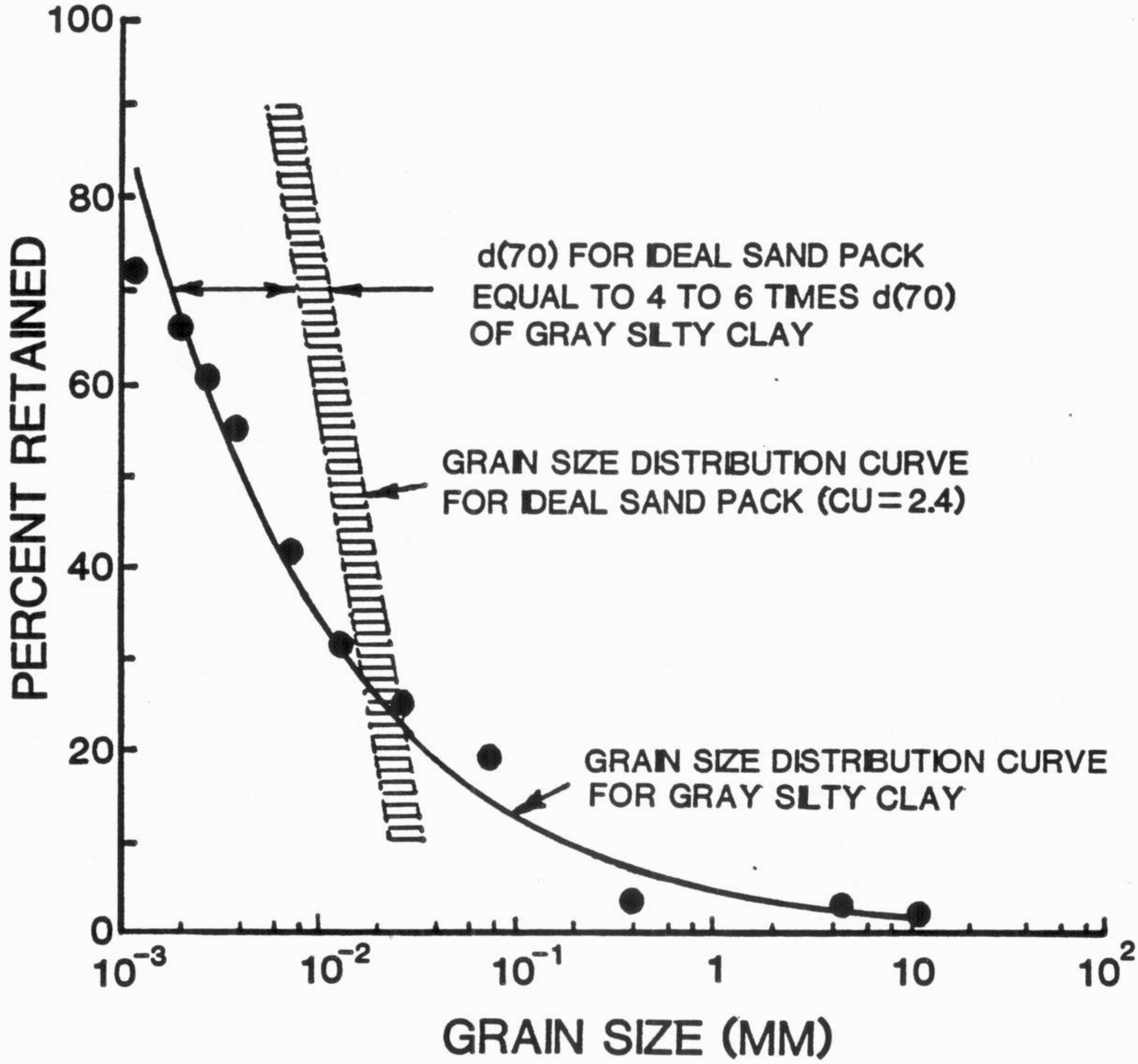

Figure 11. Graphical representation to determine the optimal sand pack size based on the size of formation material.

stone piezometer, it would require a d_{90} slightly greater than this. With a uniformity coefficient of 2.4, the d_{70} of the pack would be about 0.065 mm. This is still 6.5 times larger than a properly sized sand pack although it will retain a greater percentage of the formation than will a fine sand. The porous stone piezometer tips that were used in this study were installed in silica flour that was described as having a mean diameter of 0.074. This silica flour, however, was observed to enter the porous stone piezometers and increase the turbidity of the sample. The use of the porous stone piezometers is therefore contingent on obtaining the proper sized filter pack material.

One of the major factors that affected the amount of turbidity in the samples was the surging of the wells. The rationale for this surging is 1) to help remove fine grained particles in the well, sand pack and near the formation that may potentially enter a sample, and 2) to reduce the effects of any skin that may form during the drilling and installation of a monitoring well. The surging, however, produced samples that were much more turbid than samples from wells that were not surged. In addition, there was no decrease in the turbidity with subsequent samplings when surging was performed. In contrast, wells which were bailed-only showed a 75 percent decrease in turbidity with additional sampling.

Another problem affecting water sample turbidity is sediment present in monitoring wells following installation. This problem was most evident in those wells in which the sand pack was set in wet boreholes. Once sediment has entered the well annulus, it may settle out into the bottom of the piezometer if it is not entirely removed by the bailing process. This remaining sediment may be brought into suspension with subsequent bailing. Therefore, surging may be an effective means to bring the bottom sediment into suspension so that it may be removed.

SLUG - BAIL TESTS

A slug test is used to determine the hydraulic conductivity or transmissivity of the geologic unit being investigated. A slug test is performed by instantaneously changing the head in a well and recording the rate of recovery to static equilibrium. The instantaneous change is accomplished by removing or injecting a volume of water or by adding or removing a solid object of known volume. The latter is commonly known as a slug test. If a slug of water is removed with a bailer, the test is commonly referred to as a bail test. A third method which would coincide with the development process would be to remove the column of water instantaneously with a volume of compressed air and observe the water level recovery back to equilibrium. Injecting a slug of water would be very convenient and practical if the well is used to study the physical characteristics of the fine-grained material, but is not recommended if the well is to be used for chemical studies.

The use of slug tests to determine hydraulic parameters of aquifer material is well documented in the literature. In the present study, the methods of Hvorslev (1951), Cooper, et al. (1967) - Papadopulos, et al. (1973), and Bouwer and Rice, (1976) have been used to evaluate the slug test data. Assumptions of the slug-testing solutions will be used to critically evaluate discrepancies in the hydraulic conductivity obtained by each of the methods. Finally, comments will be made on the effect of well development on calculated hydraulic conductivity with consecutive slug tests.

Previous Studies

Probably the most common method of evaluating slug test data is that of Hvorslev (1951). His analytical solution is derived from the differential equation for hydrostatic time lag where stress adjustment time lag and other sources of error are negligible. The solution by Hvorslev (1951) assumes that:

1) Soil is present at the well intake,
2) The medium is homogeneous, isotropic and of infinite vertical extent,
3) The specific storage is equal to zero,
4) No sedimentation or leakage is present,
5) The soil, well point and standpipe are assumed air free, and,
6) Hydraulic losses in the well point, standpipe, or filter material are considered negligible.

Hvorslev (1951) presents many formulas for determining hydraulic conductivity of soils based on a variety of piezometer configurations. The typical piezometer configuration used in this study is shown in Figure 12. The basic time lag equation to determine formation hydraulic conductivity for this

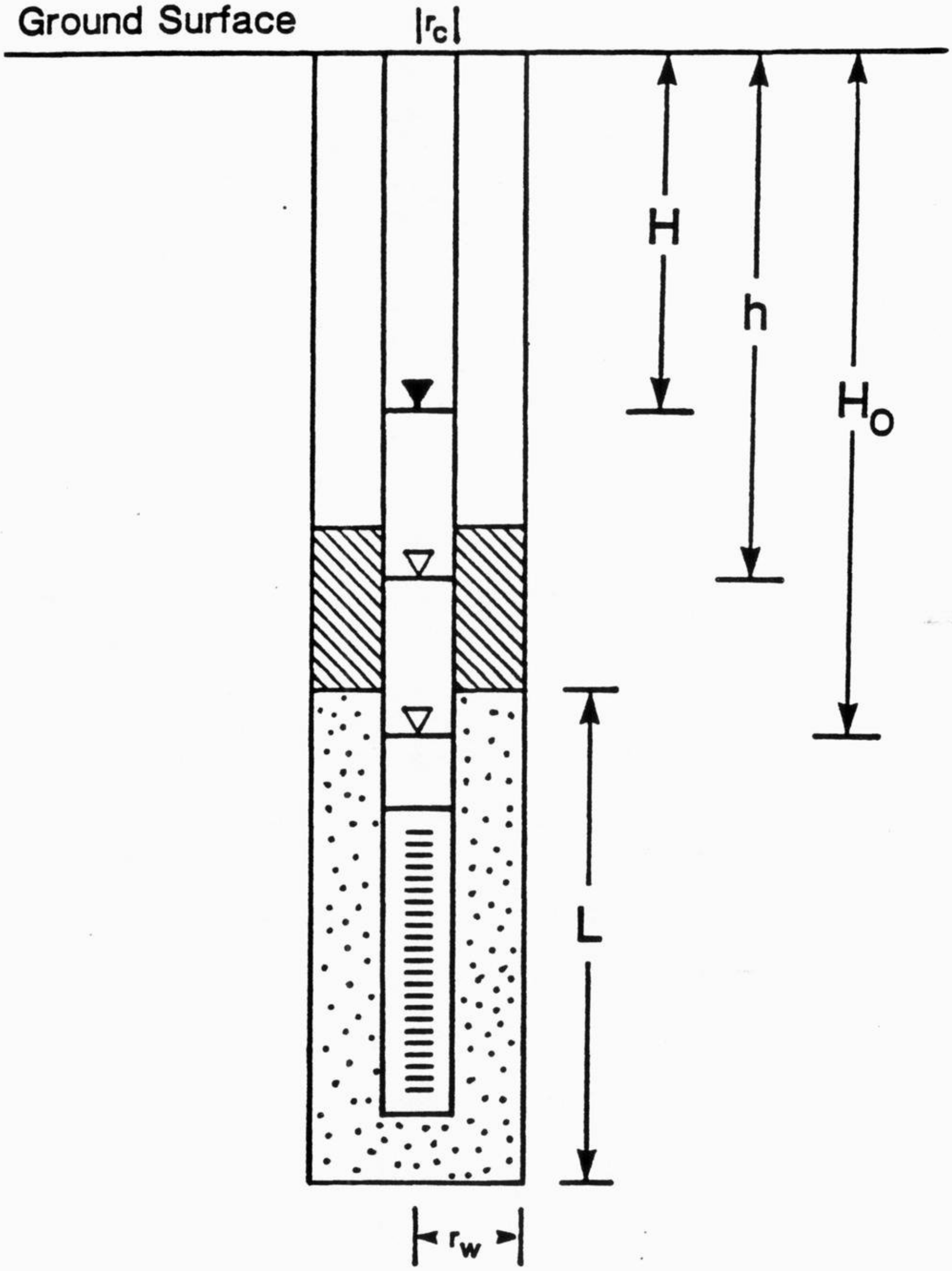

Figure 12. General piezometer configuration defining the piezometer parameters and head relationships used to determine hydraulic conductivity by the Hvorslev (1951) equation. H is the static water level at the start of the test, H_o is the lowest head produced after the slug has been removed, and h is an intermediate recovery value.

configuration is:

$$K_h = \frac{r_c^2 * \ln(m*L/r_w)}{2 * L * T_o} \qquad (6)$$

where:
r_c = standpipe radius (cm)
m = transformation ratio
= $(K_h/K_v)^{.5}$
L = sand pack length (cm)
r_w = borehole radius (cm)
T_o = basic time lag (sec)
K_h = horizontal permeability (cm/sec)
K_v = vertical permeability (cm/sec)

Equation (6) assumes the well configuration is constructed such that $(m*L/r_w)$ is greater than 4. If the formation is isotropic, the transformation ratio (m) is equal to unity and the basic time lag equation then simplifies to:

$$K_h = \frac{r_c^2 * \ln(L/r_w)}{2 * L * T_o} \qquad (7)$$

To determine the hydraulic conductivity of the formation using this formula, the slug test data are plotted on semi-log paper as the log of relative head versus time. The relative head (Fig. 12) is defined as the ratio of the unrecovered head difference (H - h) to the bailed head difference $(H - H_o)$. The basic time lag (T_o) is defined as the time that would be required for full recovery of the head difference $(H - H_o)$ if the initial rate of flow was maintained. Therefore, the basic time lag (T_o) is the time (t) at which $(H - h/H - H_o) = 0.37$ (Hvorslev,1951). The time (t) (Fig. 13) chosen at the intersection of

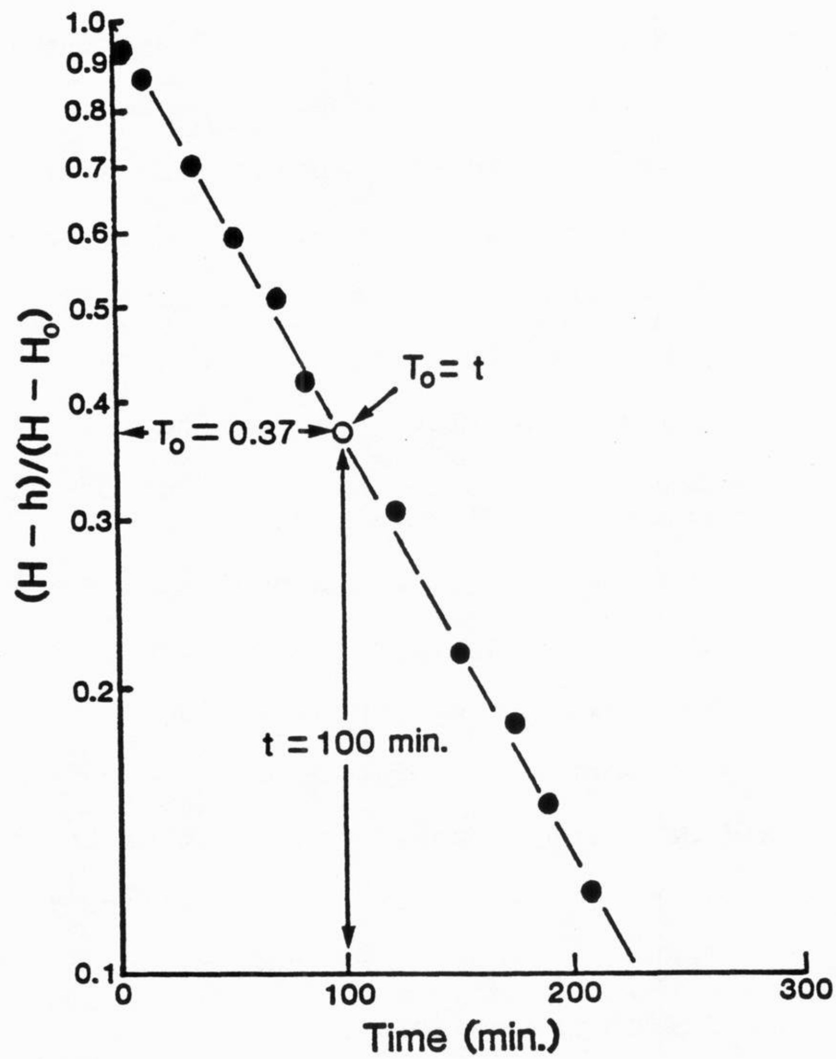

Figure 13. Hvorslev (1951) plot of relative head versus time illustrating the choice of basic time lag.

$\ln(H - h)/(H - H_o) = 0.37$ is then equal to the basic time lag T_o ($T_o = t$). The value of time lag is used in equation (7) and a value of hydraulic conductivity is calculated. The second method used to evaluate slug test data in this study is that of Cooper, et al. (1967) which was later expanded by Papadopulos, et al. (1973). They presented a solution for determining the transmissivity and storage coefficient of an aquifer material from slug test data. Their solution is based on most of the same assumptions as the Theis solution for non steady-state flow to a pumping well:

1) The medium is homogeneous, isotropic and of infinite horizontal extent,
2) The tested aquifer is confined,
3) The confining layer is non-leaky, and
4) The well is fully penetrating.

They present a set of type curves against which the field data are compared to determine the "aquifer" transmissivity (T) and storage coefficient (S). The hydraulic conductivity (K) of the medium can then be determined from the relationship:

$$K = T/b \tag{8}$$

Likewise specific storage (S_s) can be determined from its relationship to the storage coefficient (S):

$$S_s = S/b \tag{9}$$

where b is the thickness of the "aquifer" material being investigated.

In general the solution involves plotting the relative head versus time where the relative head $[(H - h)/(H - H_o)]$ is plotted on an arithmetic axis and time on a logarithmic axis. The field data are then superimposed on the type curves to obtain a best fit and a match point is chosen (Fig. 14). It is convenient to choose a match point such that $(T*t/r_c^2)$ equals 1.0. This value of time (t) is input into the rearranged equation to solve for transmissivity (Cooper, et al., 1967):

$$T = \frac{1.0*(r_c)^2}{t} \tag{10}$$

where: T = transmissivity (cm^2/sec)
r_c = standpipe radius (cm)
t = match point time (sec)

A storage coefficient for the "aquifer" material can be approximated for the formation material from the alpha parameters of the type curves (Fig. 14) and specific storage, S_s, can be calculated by eq. (9). The type curves presented are similar in slope, so the authors argue that the curve matching technique is much more reliable for the determination of transmissivity than for the storage coefficient (Cooper, et al. 1967).

A third method used to evaluate slug test data is by a technique developed by Bouwer and Rice (1976). The solution is based on the Thiem equation for steady state flow to a well assuming:

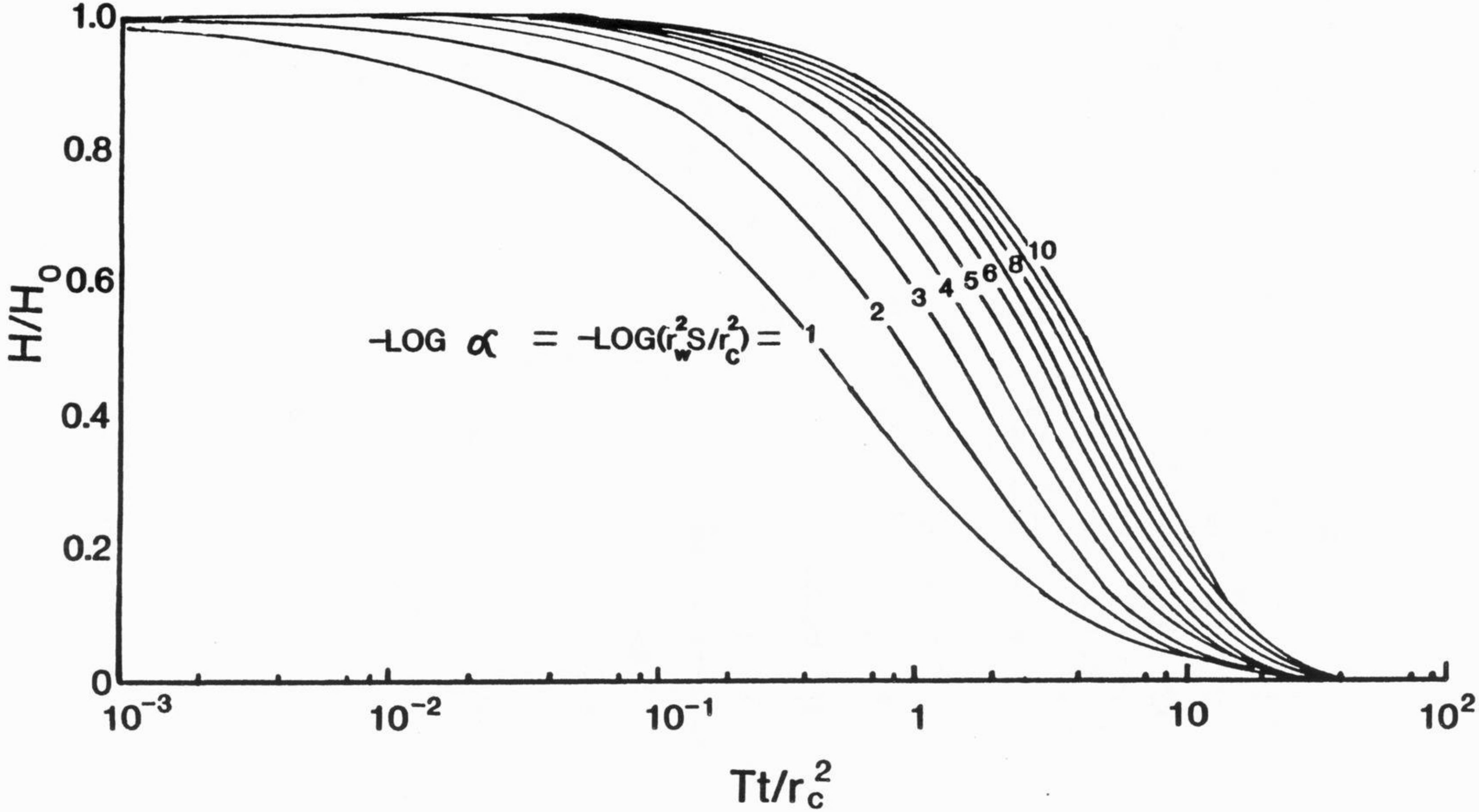

Figure 14. Type curves for slug test analysis based on the numerical solution of Cooper, et al. (1967) - Papadopulos, et al. (1973).

1) Drawdown of the water table is negligible,

2) Contribution from capillary water is negligible,

3) Well losses are negligible,

4) The medium is homogeneous, isotropic, and of infinite vertical extent, and

5) Change in storage is zero.

The basic equation which can be used to determine hydraulic conductivity of partially or fully penetrating wells is:

$$K = \frac{r_c^2 * \ln(R_e/r_w)}{2 * L} * \frac{1}{t} * \ln \frac{H_o}{h} \qquad (11)$$

where
r_c = stand pipe radius (cm)
R_e = effective radius of the well (cm)
r_w = borehole radius (cm)
L = sand pack length (cm)
H_o/h = relative head at time t
t = time of head measurement h (sec)

(from Bouwer and Rice, 1976).

This equation is very similar to the Hvorslev (1951) equation for the piezometer configuration in this study. The difference is the $\ln(R_e/r_w)$ term of the Bouwer and Rice equation where R_e is the effective radius or horizontal radius over which the head difference $(H - H_o)$ is dissipated. The equivalent term in the Hvorslev equation is $\ln(L/r_w)$ where the effective radius is assumed to be equal to the length of the sand pack (L). Bouwer and Rice (1976) have determined the effective radius term R_e with the use of an electrical resistance analog for different values of L, r_w, D, and H.

For the case of a partially penetrating well configuration where H < D (Fig. 15), an empirical relationship was derived with the use of the analog system to relate $\ln(R_e/r_w)$ to the geometry of the partially penetrating well configuration:

$$\ln \frac{R_e}{r_w} = \left[\frac{1.1}{\ln(H/r_w)} + \frac{A + B * \ln[(D - H)/r_w]}{L/r_w} \right]^{-1} \quad (12)$$

where R_e = effective radius of the well (cm)
r_w = borehole radius (cm)
D = saturated aquifer thickness (cm)
H = vertical distance from the water table to the piezometer bottom (cm)
L = sand pack length (cm)
A,B = dimensionless coefficients $f(L/r_w)$ (Fig. 16)

(from Bouwer and Rice, 1976).

Simulations of the analog system showed that where H << D, increases in D have no measureable effect of $\ln(R_e/r_w)$ and the effective upper limit of $\ln[(D - H)/r_w]$ is 6 (Bouwer and Rice, 1976).

In the case of a fully penetrating well, where D = H, the empirical relationship derived is:

$$\ln \frac{R_e}{r_w} = \left[\frac{1.1}{\ln(H/r_w)} + \frac{C}{L/r_w} \right]^{-1} \quad (13)$$

where: C = dimensionless coefficient, $f(L/r_w)$

(from Bouwer and Rice, 1976).

Simulations of the analog system were conducted assuming the bottom was closed; however, several simulations were made with open bottoms. Vertical flow was found to be negligible with the exception where L/r_w << 4. In this case, the flow from the bottom of

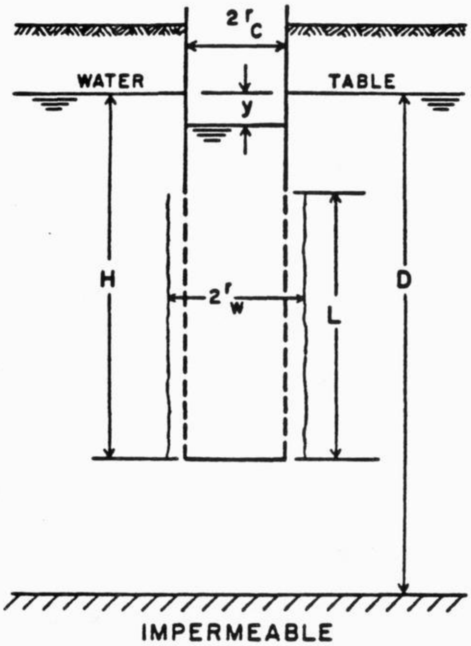

Figure 15. Geometry and symbols of a partially penetrating, partially perforated well in an unconfined aquifer with gravel pack or developed zone around perforated section. (From Bouwer and Rice, 1976).

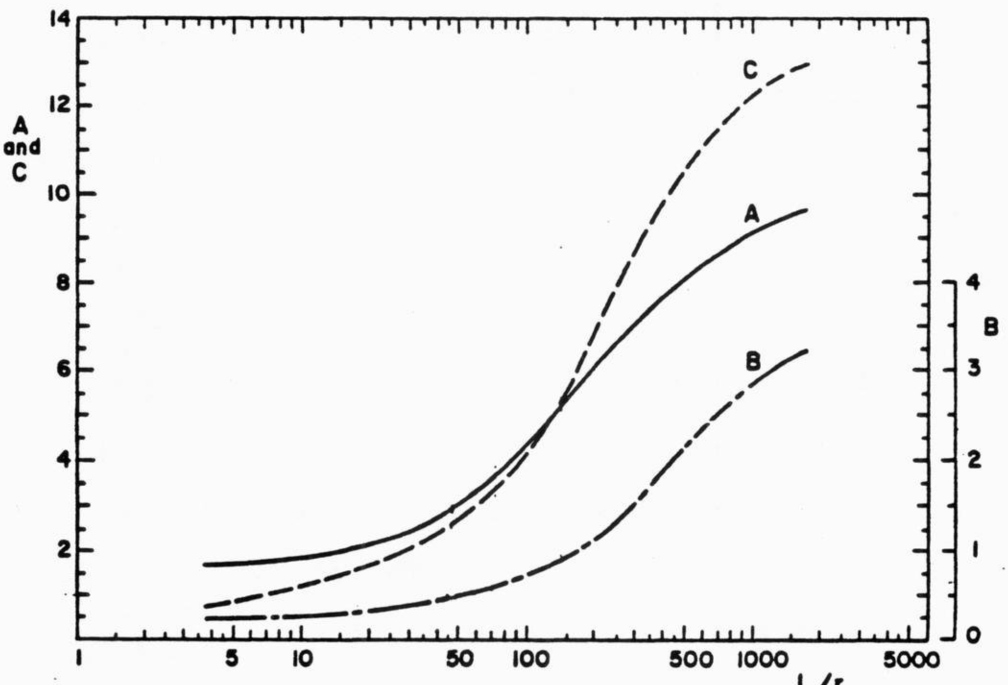

Figure 16. Curves relating coefficients A, B, and C to L/r_w. (From Bouwer and Rice, 1976).

the piezometer can be significant (Bouwer and Rice, 1976). This criterion is similar to the one used by Hvorslev (1951) for the use of the simplified equation (6) where $(m*L/r_w)$ is required to be greater than 4.

The partially penetrating well solution (eq. 12) where $\ln [(D - H)/r_w]$ is equal to 6 was chosen for this study because it best represents the parameters of a typical piezometer configuration. The sand pack length (L) and sand pack radius (r_w) of piezometers installed in this study were 8 feet (2.4 m) and 0.25 feet (0.076 m), respectively. Because the ratio L/r_w is approximately 32, the dimensionless coefficients A and B were graphically chosen as 2.5 and 0.4 respectively (Fig. 16). An iterative program with the solution of Bouwer and Rice (1976) was used to determine hydraulic conductivity. The iterative solution was checked with several hand calculations to assure the integrity of the program.

Interpretation of Bail Test Data

Numerous bail tests have been performed on the wells at site 1 and site 2 (Appendix III). The bail tests were initiated with the use of a bailer to "instantaneously" remove a slug of water. The bailing process took about three to five minutes to perform. Within a time frame of one week to three months for the well to recover to equilibrium, the bailing process can be considered instantaneous.

On May 1, 1986, the first bail test was performed at site 1. After the wells were bailed to the bottom, a fiber glass measuring tape with a metal "sounder" was used to measure water level reccovery in the well. The raw bail test data were first plotted by the method of Hvorslev (1951) (i.e. $\log(H/H_o)$ versus time). Plots of the raw data show two types of responses. A plot of well 1-1, which is the only 4 inch auger-bored hole at site 1, is essentially a straight line (Fig 17). This straight-line response is the typical Hvorslev response indicating that the water level is exponentially recovering to equilibrium. Such a straight line also indicates that the specific storage of the formation is negligible.

In contrast, wells 1-3 to 1-6, which are set in 6 inch bore holes, show a response similar to that of well 1-2 (Fig 18). The curved line response of the latter wells typically shows three rates of recovery: 1) rapid recovery in the very early stage of the test, 2) slow recovery in the intermediate stage of the test and 3) slightly increased recovery in the final stages of the test.

At site 2, well recovery was observed from the time the piezometer was finished until the well reached static equilibrium. Quick recovery of some wells did not allow for the collection of recovery data. However, Hvorslev plots of the remaining wells

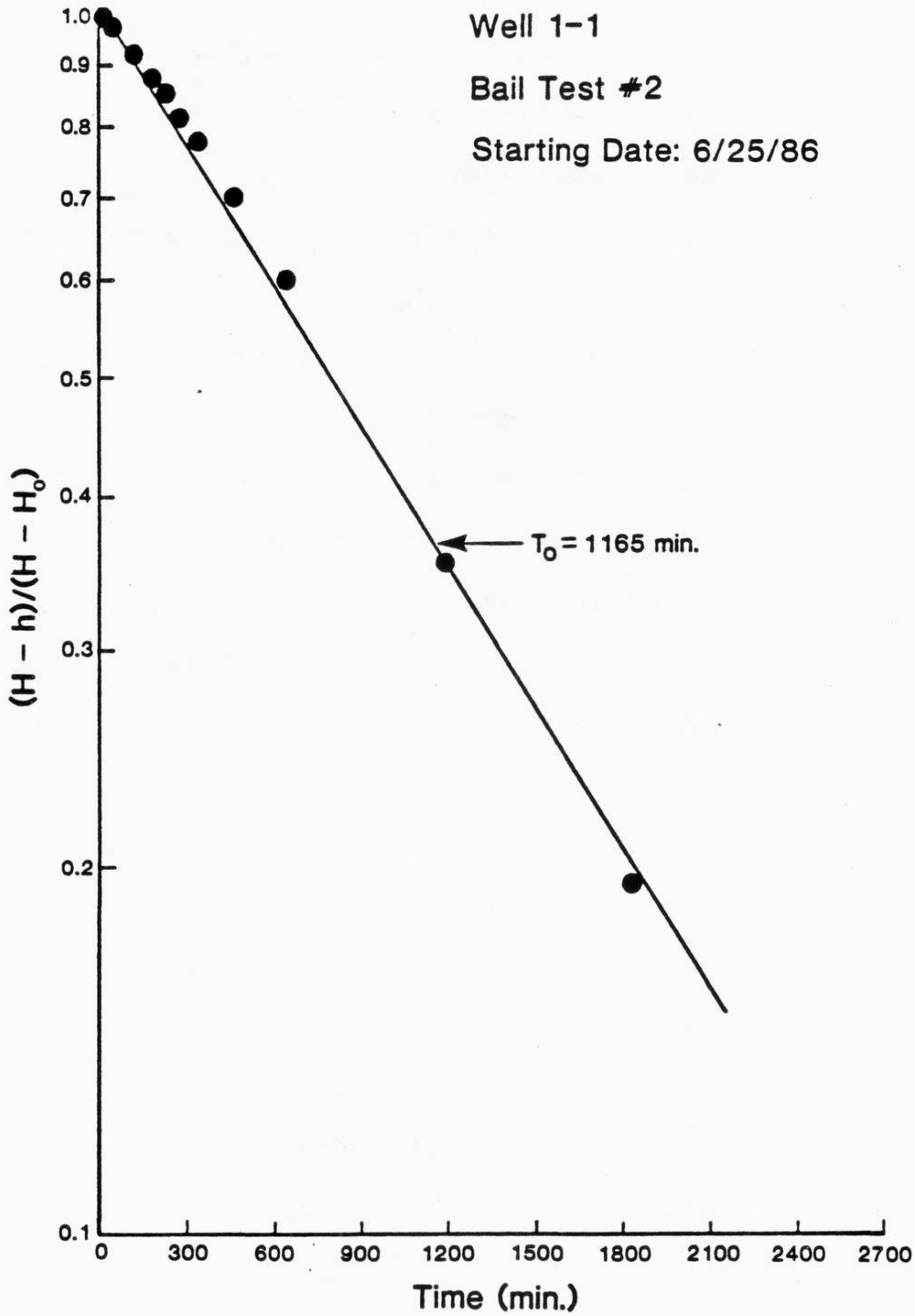

Figure 17. Hvorslev plot of well 1-1 showing a straight-line response.

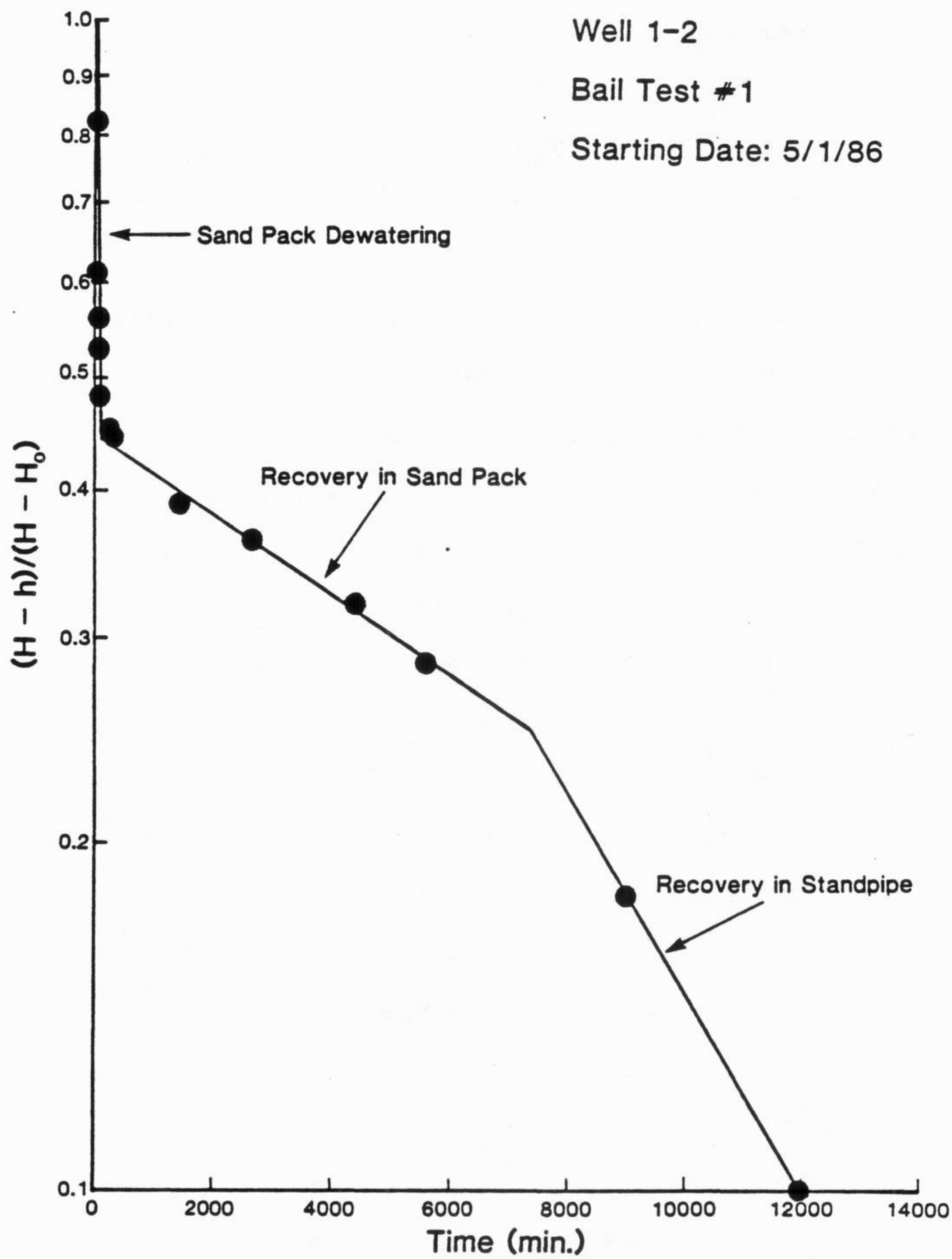

Figure 18. Hvorslev plot of well 1-2 showing three rates of recovery.

typically show two rates of recovery (Fig. 19):
1) slow recovery in the early stage of the test and
2) increased recovery in the final stage. This parallels the response in the latter stages of well 1-2 (Fig. 18).

To derive a correct hydraulic conductivity from these types of Hvorslev responses, one must understand what is physically happening in the monitoring well during the test. When a sand-packed piezometer is "instantaneously" bailed to the bottom of the screened interval, water is temporarily retained in the sand pack until the force of gravity allows it to drain freely. The result, as shown by a Hvorslev plot (Fig. 18), is rapid recovery of the bailed head in the early portion of the test. The sand pack dewatering was observed as far as an hour into the test.

After the sand pack has dewatered and equilibrated with the recovering head in the well, further volume recovery measured in the screened portion of the well is a function of water influx required to saturate the piezometer annulus, screen slots, and remaining pore spaces in the sand pack. Hence, recovery rates will be slower in the sand packed portion of the well than if the recovery were restricted to the annulus of the well. Therefore, the slope of the latter portion of the recovery curve (Fig. 18 and 19) is closest to the actual recovery rate of the piezometer.

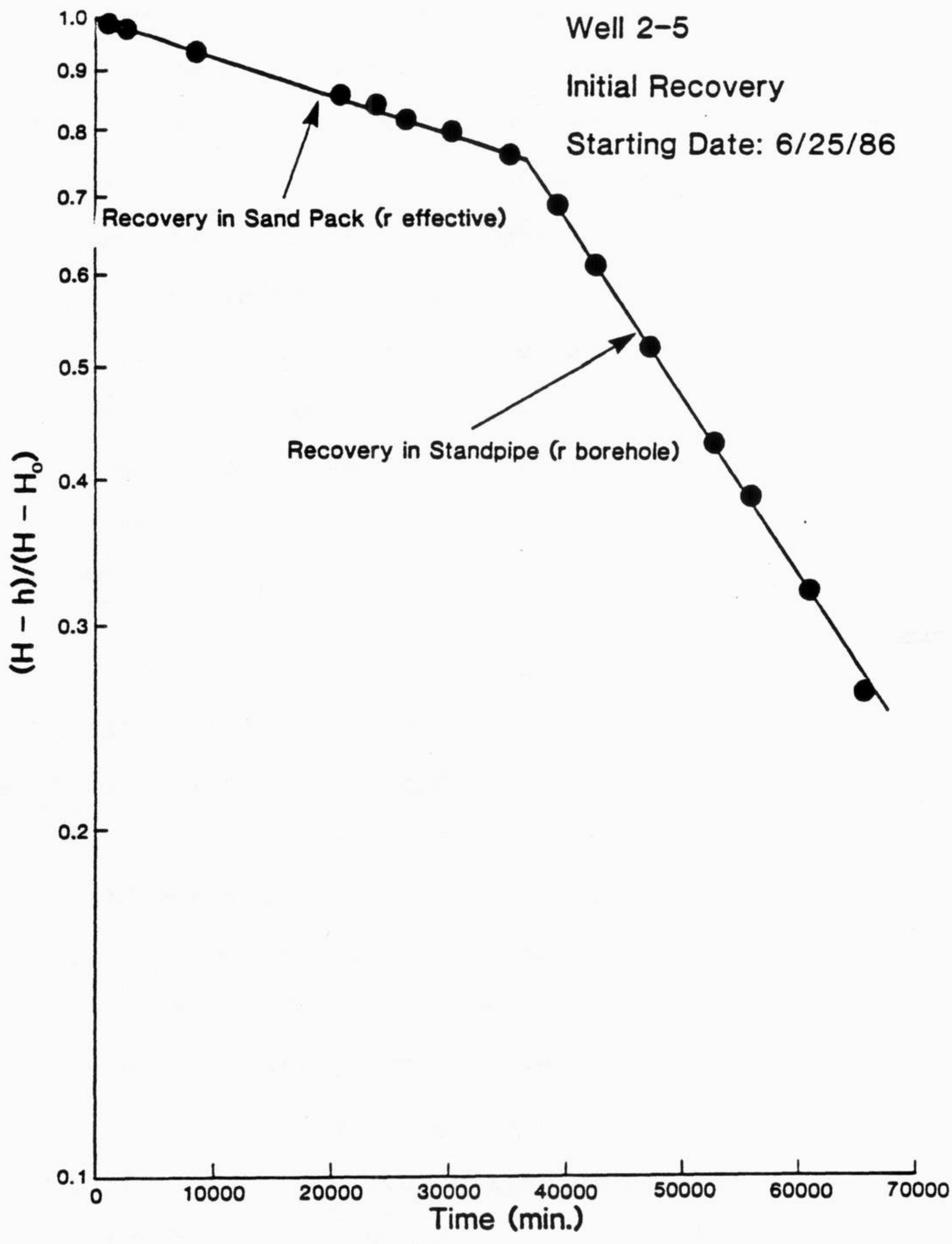

Figure 19. Hvorslev plot of well 2-5 showing the initial recovery response.

Therefore, to eliminate biases in calculated hydraulic conductivity due to sand pack dewatering (Fig. 18) and/or recovery within the sand pack (Fig. 18 and 19): 1) the raw data was initially plotted by the method of Hvorslev (1951), and 2) the data of the latter portion of the curve were replotted and evaluated to determine the hydraulic conductivity. These corrected data were also used for evaluation of hydraulic conductivity by the methods of Cooper, et al., (1967), Papadopulos, et al., (1973), and Bouwer and Rice (1976).

Bail Test Results

Hydraulic conductivity of the glacial till at site 1 and site 2 has been calculated from bail test data by the method of Hvorslev (1951), Cooper, et al., (1967) - Papadopulos, et al., (1973), and Bouwer and Rice (1976). At site 1, three bail tests and one slug test were performed on each well. The bail test was initiated with the use of a PVC bailer to instantaneously lower the head in the well. The slug test was initiated by injecting 2000 cm^3 of deionized water from the surface. The mean hydraulic conductivity as calculated by the method of Hvorslev (1951) at site 1 ranged from 2.0×10^{-7} to 8.6×10^{-7} cm/s (Table 6).

At site 2, all the piezometer screens and sand packs were set in essentially dry boreholes. For most of these wells, initial recovery data was recorded. In

Table 6. Hydraulic conductivities of wells at site 1 as calculated by the method of Hvorslev (1951). Geometric means for three sets of recovery data are presented for each well at site 1. In addition, means are presented for comparison of: 1) the overall means with consecutive bail tests, 2) the effect of screen-type chosen for piezometer construction, and 3) the effect of development with consecutive bail tests.

WELL NO.	BAIL TEST #1 (5/1/86) K, (cm/s)	BAIL TEST #2 (6/25/86) K, (cm/s)	SLUG TEST #1 (8/27/86) K, (cm/s)	BAIL TEST #3 (9/25/86) K, (cm/s)	Geometric Mean K, (cm/s)
1-1*	9.5E-7	6.7E-7	8.4E-7	1.0E-6	8.6E-7
1-2	7.6E-8	5.4E-8	1.0E-7	1.2E-7	8.4E-8
1-3*	3.0E-7	1.8E-6	1.9E-7	2.5E-7	4.0E-7
1-4	1.7E-7	1.7E-7	2.8E-7	4.2E-7	2.4E-7
1-5*	***	9.9E-7	4.4E-8	3.8E-7	2.5E-7
1-6	***	1.4E-7	1.9E-7	3.0E-7	2.0E-7
Geo. Mean	2.5E-7	3.4E-7	1.8E-7	3.4E-7	2.7E-7
FS	2.7E-7	1.9E-7	2.9E-7	3.5E-7	
FSWMW	2.3E-7	5.5E-7	2.3E-7	3.2E-7	
CS	***	3.7E-7	9.1E-8	3.4E-7	
Surged	5.3E-7	1.1E-6	1.9E-7	4.6E-7	
Bailed	1.1E-7	1.1E-7	1.7E-7	2.5E-7	

* Indicates development by surging. Others were developed by only bailing.

*** Insufficient recovery data to produce a Hvorslev plot.

FS = Factory Slot (1-1, 1-2)
FSWMW = Factory Slot with Mirafi Wrap (1-3, 1-4)
CS = Continuous Slot (1-5, 1-6)

addition, two bail tests were performed on the wells at site 2. The mean hydraulic conductivity as calculated by the method of Hvorslev (1951) at site 2 ranged from 1.8×10^{-8} to 2.5×10^{-7} cm/s for the standard piezometer screens (Table 7).

The results of the porous stone piezometers have not been included in the calculation of mean hydraulic conductivity at site 2 because it is felt that the porous stone results are very questionable. This was due to inappropriate construction of the silica flour pack for the porous stone piezometers. The sand pack of these piezometers (2-13 and 2-14) were sized such that the top of the pack extended more than 1 meter above the 0.62 meter long screen. When these piezometers were bailed, a large sand pack dewatering response was observed. It was difficult choosing a bailed head (H_o) that would produce a "straight - line" Hvorslev plot. The sand pack of these piezometers should have been extended less than 0.3 meters to avoid the sand pack dewatering phenomena and allow ample material to filter the bentonite sealant.

Therefore the bail test data from the standard piezometer screens at site 1 and site 2 have been analyzed to determine: 1) any differences in calculated hydraulic conductivity versus piezometer construction for the various sand pack - screen combinations, 2) any relationship between calculated hydraulic

Table 7. Hydraulic conductivities of wells at site 2 as calculated by the method of Hvorslev (1951). Geometric means for three sets of recovery data are presented for each well at site 2. In addition, means are presented for comparison of: 1) the overall means with consecutive bail tests, 2) the effect of screen-type chosen for piezometer construction, and 3) the effect of development with consecutive bail tests.

WELL NO.	INITIAL RECOVERY K, (cm/s)	BAIL TEST #1 (9/4/86)+ (9/23/86) K, (cm/s)	BAIL TEST #2 (9/23/86)+ (12/1/86) K, (cm/s)	Geometric Mean K, (cm/s)
2-1*	4.7E-8	2.3E-8 +	3.0E-8 +	3.2E-8
2-2	1.2E-7	2.3E-7 +	4.0E-7 +	2.2E-7
2-3*	***	1.3E-7 +	2.0E-7 +	1.6E-7
2-4	8.3E-8	3.8E-7 +	5.5E-7 +	1.4E-7
2-5*	2.5E-8	3.3E-8	3.1E-8	3.0E-8
2-6	1.9E-8	2.4E-8	2.2E-8	2.2E-8
2-7*	***	9.9E-8 +	1.1E-7 +	1.0E-7
2-8	1.6E-8	2.1E-8	1.8E-8	1.8E-8
2-9*	2.5E-8	4.1E-8	2.9E-8	3.1E-8
2-10	1.8E-8	2.3E-8	1.9E-8	2.0E-8
2-11*	2.0E-7	2.1E-7 +	3.8E-7 +	2.5E-7
2-12	3.7E-8	5.2E-8 +	6.0E-8 +	4.9E-8
2-13	***	1.9E-8 +	1.3E-7 +	
2-14	***	2.4E-8 +	5.4E-8 +	
Geo. Mean	4.1E-8	6.4E-8	7.3E-8	5.8E-8

FS	4.5E-8	5.8E-8	7.0E-8
FSWMW	3.3E-8	8.3E-8	8.8E-8
CS	4.3E-8	5.4E-8	6.3E-8

Surged	4.9E-8	6.6E-8	7.8E-8
Bailed	3.6E-8	6.1E-8	6.8E-8

+ Indicates test performed on (9/4/86) and (9/23/86).
* Indicates development by surging. Others were developed by only bailing.
*** Insufficient recovery data to produce a Hvorslev plot.

FS = Factory Slot (2-1, 2-2, 2-7, 2-8)
FSWMW = Factory Slot with Mirafi Wrap (2-3, 2-4, 2-9, 2-10)
CS = Continuous Slot (2-5, 2-6, 2-11, 2-12)

conductivity versus type and extent of well development (surging versus bailing - only), and 3) differences among the three types of solutions used in this study to evaluate bail test data (i.e. Hvorslev (1951), Cooper, et al., (1967) - Papadopulos, et al., (1973), and Bouwer and Rice (1976)).

The Effect of Piezometer Construction on Calculated Hydraulic Conductivity

In general, the hydraulic conductivity of piezometer screens for coarse and fine-grained materials are assumed to be much greater than the hydraulic conductivity of the formation. Screen hydraulic conductivities were calculated to range from 1.5 x 10^{-3} cm/sec for the continuous slot screens to 5.6 x 10^{-3} cm/sec for the porous stone piezometers (see Table 1). In contrast, the largest mean hydraulic conductivities calculated by the method of Hvorslev (1951) from bail test data ranged from 8.6 x 10^{-7} cm/s at site 1 to 2.5 x 10^{-7} cm/s at site 2. Because the screen hydraulic conductivities are approximately four orders of magnitude greater than the formation hydraulic conductivity, the choice of screen type should have no effect of the calculated screen hydraulic conductivity. The mean hydraulic conductivity as calculated by the method of Hvorslev (1951) versus bail (slug) tests for the various screen types for site 1 and 2 are conveniently summarized on Figures 20 and 21.

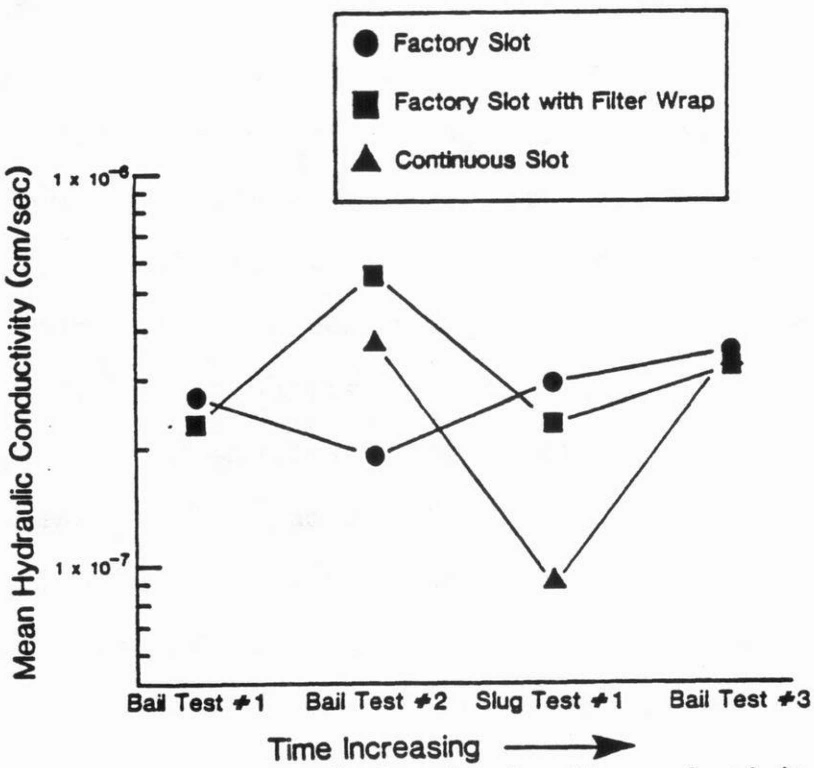

Figure 20. Bail-slug test calculated hydraulic conductivity (Hvorslev, 1951) versus time for the three screen types used at site 1.

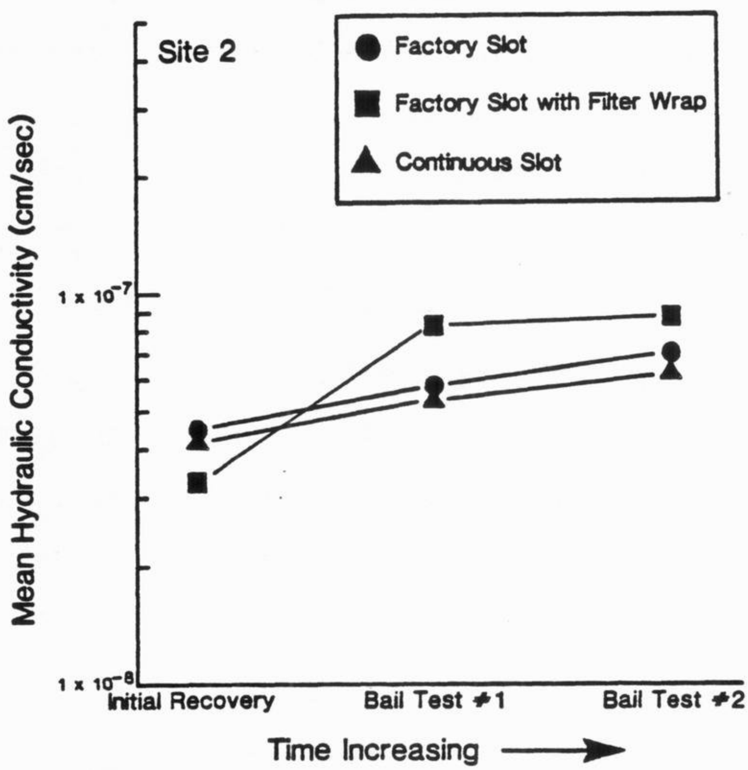

Figure 21. Bail test calculated hydraulic conductivity (Hvorslev, 1951) for the three standard screen types used at site 2.

Comparison of the mean hydraulic conductivities for the various screen types at site 1 (Fig. 20) shows no distinct trends in hydraulic conductivity with consecutive bail (slug) tests. This is expected given the argument that K(screen) >>> K(formation). However, hydraulic conductivity for the various screen types at site 2 show a parallel increasing trend with consecutive bail tests (Fig. 21). Because screen type is assumed to have no effect on the calculated hydraulic conductivity from bail tests, the extent of piezometer development is a possible explanation for the increasing trend.

The Effect of the Type and Extent of Piezometer Development on Calculated Hydraulic Conductivity

Well development by the method of surging and bailing should improve the sand pack to serve as a flow through system by removing air bubbles entrapped in the sand pack. If damage to the borehole wall has occurred during the drilling process, development by surging should be an effective means of developing the borehole wall. Turbidity results of this study suggest that the surging process is pulling formation material through the sand pack and into the piezometer annulus. Therefore, one would assume development of the bore hole wall is occurring and expect to observe increases in hydraulic conductivity with time.

The results of well development versus calculated hydraulic conductivity are conveniently summarized in Figures 22 and 23. At site 1 (Fig. 22), the results are very "noisy". A comparison of the first and last bail tests indicates a decrease of a factor of 1.2 for surged wells. In contrast, bailed only wells show an overall increase in hydraulic conductivity of a factor 2.3. Of the two methods of development at site 1, the bailed only wells show the only increasing trend of hydraulic conductivity with consecutive testing.

In contrast, the results from surged and bailed only wells at site 2 parallel each other very closely Fig. 23). Comparison of the initial recovery and last bail test calculated hyraulic conductivities shows a mean increase in hydraulic conductivity of a factor of 1.6 for surged wells versus 1.9 for bailed only wells. Well development by surging produces mean hydraulic conductivities which are within a factor of 2 from wells that were developed by bailing only. In fact, the results from site 2 suggest that the effect of development by bailing-only is equal to, or possibly better than development by surging in increasing the bail test calculated hydraulic conductivity.

However, the development process may not be solely responsible for the trend of increasing hydraulic conductivity with consecutive testing observed at site 2. A factor that may contribute to the increased hydraulic conductivity observed in this study could be

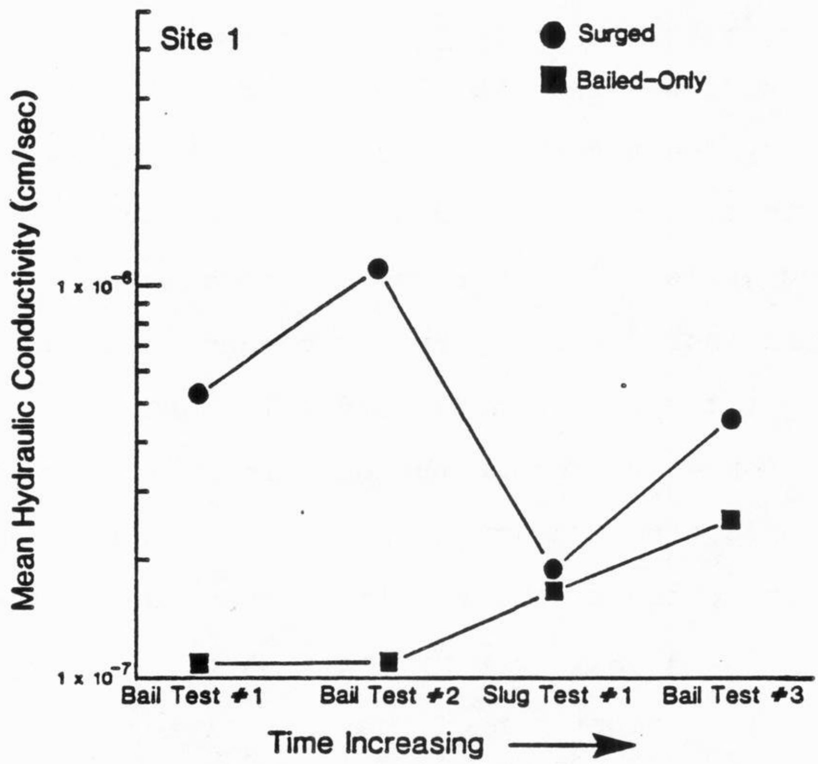

Figure 22. Graphical representation of bail-slug test calculated hydraulic conductivities (Hvorslev,1951) versus time for wells developed by surging and bailing-only at site 1.

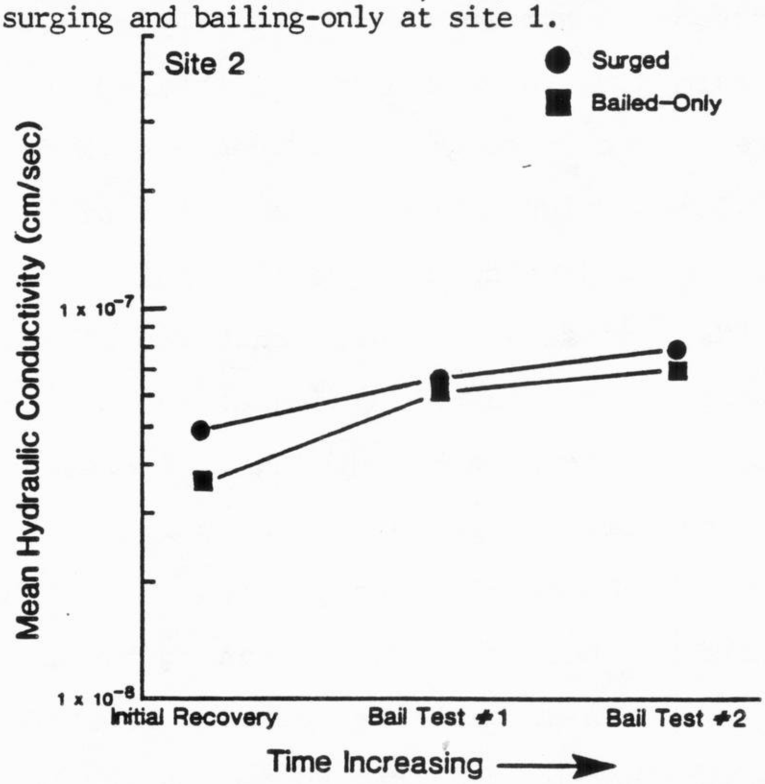

Figure 23. Graphical representation of bail test calulated hydraulic conductivities (Hvorslev, 1951) versus time for wells developed by surging and bailing-only at site 2.

the effect of a fluctuating water table over the duration of a bail test. Normally, rising or falling water tables are considered negligible during the slug tests of coarse grained materials. However, in fine-grained materials where a bail test may take several weeks to several months to perform, recharge events such as rainfall may influence computed hydraulic conductivity.

Because there was an abnormally high amount of rainfall during the summer and fall of 1986, a bail test was performed on 12/1/86 to determine any effects of the recharge on the calculated hydraulic conductivities. The calculated hydraulic conductivities of some the wells developed and bailed on 12/1/86 show a slight decrease from bail test #1 (Table 7). There two possible explanations. First, recharge events such as the periodic heavy rainfall events experienced during the summer and fall of 1986 may have significantly raised the water table over the duration of a slug test. This would result in hydraulic conductivities that are slightly greater than if the water table were to remain constant during the bail test.

Secondly, the bail test on 12/1/86 may have been performed during a period of falling water table conditions which may have occurred when recharge to the ground water system was limited by a surficial frost layer. However, it is assumed that the bail test of

12/1/86 was performed during conditions of limited water table fluctuations. In either case, the results of the 12/1/86 bail test produce hydraulic conductivities which are within a factor of 1.5 times the hydraulic conductivity of the previous bail test. This small change may be considered insignificant when trying to reproduce results which are within an order of magnitude (factor of 10) for a given monitoring point.

In general, the calculated hydraulic conductivities of both surged and bailed only wells at site 1 were less consistent with consecutive tests than those of site 2. This was because the static heads at site 1 were very close to the screened portion of the wells. The head differences at site 1 typically ranged from 0.3 to 1.0 meters for a given bail test. With such a small head difference, measurement error becomes important. In contrast, measurement error at site 2 becomes negligible with an average head difference of 3 meters. Therefore, the size of a bail or slug volume removed to initiate a test may be an important consideration in reference to the measurement error.

Differences in Bail - Slug Test Hydraulic Conductivity Calculated from Currently used Analytical Solutions

In addition to the Hvorslev (1951) analysis (Tables 6 and 7), hydraulic conductivities from bail - slug test data have been calculated by the methods of

Cooper, et al.,(1967) - Papadopulos, et al., (1973) (Tables 8 and 9) and Bouwer and Rice (1976) (Tables 10 and 11). A comparison of the overall means for the methods performed (Table 12 and 13) show that hydraulic conductivities as determined by Bouwer and Rice (1976) are less than those from Hvorslev (1951) and hydraulic conductivities as determined by Hvorslev (1951) are less than those from Cooper, etal., (1967) - Papadopulos, et al. (1973). A schematic comparing the hydraulic conductivity results of three slug testing solutions for sites 1 and 2 is conveniently summarized in Figures 24 and 25.

Hydraulic conductivities as calculated by Bouwer and Rice (1976) are less than the hydraulic conductivities calculated by Hvorslev (1951) because Hvorslev assumes the effective radius of a given well to be equal to the sand pack length L (eq. 7). This "effective" radius, Re, of the well as determined by electrical analog simulation was found to be less than the length of the sand pack (Bouwer and Rice,1976). Therefore the smaller effective radius in eq. (11) results in a lower calculated hydraulic conductivity.

Equations 12 and 13 presented earlier for calculating ln (R_e/r_w) were used to evaluate when the hydraulic conductivity computed using the Bouwer and Rice (1976) method will be less than the value computed using the Hvorslev (1951) method. In most

Table 8. Hydraulic conductivities of the wells at site 1 as calculated by the method of Cooper, et al., (1967) - Papadopulos, et al., (1973).

WELL NO.	BAIL TEST #1 (5/1/86) K, (cm/s)	BAIL TEST #2 (6/25/86) K, (cm/s)	SLUG TEST #1 (8/27/86) K, (cm/s)	BAIL TEST #3 (9/25/86) K, (cm/s)	Geometric Mean K, (cm/s)
1-1*	1.5E-6	1.6E-6	1.8E-6	2.8E-6	1.9E-6
1-2	8.8E-7	7.8E-8	3.4E-6	1.0E-7	3.9E-7
1-3*	***	5.2E-6	6.5E-7	8.9E-7	1.4E-6
1-4	***	***	5.3E-7	7.6E-7	6.3E-7
1-5*	***	2.9E-6	1.2E-6	9.6E-6	3.2E-6
1-6	***	***	6.4E-8	4.5E-7	1.7E-7
Geo. Means	1.1E-6	1.2E-6	7.4E-7	9.7E-7	9.4E-7

Table 9. Hydraulic conductivitity of wells at site 2 as calculated by the method of Cooper, et al., (1967) - Papadopulos, et al., (1973).

WELL NO.	INITIAL RECOVERY K, (cm/s)	BAIL TEST #1 (9/4/86)+ (9/23/86) K, (cm/s)	BAIL TEST #2 (9/23/86)+ (12/1/86) K, (cm/s)	Geometric Mean K, (cm/s)
2-1*	***	6.6E-8 +	*** +	6.6E-8
2-2	3.3E-7	4.5E-7 +	1.1E-6 +	5.5E-7
2-3*	***	3.0E-7 +	6.3E-7 +	4.3E-7
2-4	1.52E-7	8.2E-7 +	1.8E-6 +	6.0E-7
2-5*	4.7E-8	9.2E-8	5.1E-8	6.0E-8
2-6	5.0E-8	7.4E-8	3.9E-8	5.2E-8
2-7*	***	2.8E-7 +	3.8E-7 +	3.3E-7
2-8	4.2E-8	6.3E-8	3.5E-8	4.5E-8
2-9*	5.7E-8	1.1E-7	5.5E-8	7.0E-8
2-10	5.4E-8	3.9E-8	2.6E-8	3.8E-8
2-11*	3.9E-7	2.8E-7 +	1.2E-6 +	5.1E-7
2-12	8.8E-8	1.4E-7 +	1.9E-7 +	1.3E-7
2-13*	***	8.7E-8 +	3.3E-7 +	
2-14	***	5.8E-8 +	*** +	
Geo. Means	9.3E-8	1.5E-7	1.9E-7	1.4E-7

\+ Indicates tests performed on 9/4/86 and 9/23/86.
* Indicates development by surging. Others were developed by only bailing.
*** Insufficient data to produce a hydraulic conductivity.

Table 10. Hydraulic conductivities of the wells at site 1 as calculated by the method of Bouwer and Rice (1976).

WELL NO.	BAIL TEST #1 (5/1/86) K, (cm/s)	BAIL TEST #2 (6/25/86) K, (cm/s)	SLUG TEST #1 (8/27/86) K, (cm/s)	BAIL TEST #3 (9/25/86) K, (cm/s)	Geometric Mean K, (cm/s)
1-1	8.6E-7	5.0E-7	6.6E-7	8.8E-7	7.1E-7
1-2	4.7E-8	3.6E-8	6.6E-8	1.2E-7	6.0E-8
1-3	1.8E-7	1.8E-6	1.1E-7	1.6E-7	2.7E-7
1-4	1.3E-7	1.1E-7	1.8E-7	3.1E-7	1.7E-7
1-5	***	6.1E-7	2.8E-8	2.4E-6	3.4E-7
1-6	***	5.4E-8	1.2E-7	1.6E-7	1.0E-7
Geo. Mean	1.8E-7	2.2E-7	1.2E-7	2.4E-7	2.0E-7

Table 11. Hydraulic conductivities of the wells at site 2 as calculated by the method of Bouwer and Rice (1976).

WELL NO.	INITIAL RECOVERY K, (cm/s)	BAIL TEST #1 (9/4/86)+ (9/23/86) K, (cm/s)	BAIL TEST #2 (9/23/86)+ (12/1/86) K, (cm/s)	Geometric Mean K, (cm/s)
2-1	3.4E-8	1.7E-8 +	2.1E-8 +	2.3E-8
2-2	4.6E-8	1.4E-7 +	3.2E-7 +	1.3E-7
2-3	***	9.2E-8 +	1.4E-7 +	1.1E-7
2-4	5.5E-8	2.7E-7 +	4.8E-7 +	1.9E-7
2-5	1.7E-8	2.6E-8	2.1E-8	2.1E-8
2-6	1.3E-8	1.9E-8	1.6E-8	1.6E-8
2-7	***	7.7E-8 +	7.7E-8 +	7.7E-8
2-8	1.1E-8	1.5E-8	1.6E-8	1.4E-8
2-9	1.7E-8	2.8E-8	2.2E-8	2.2E-8
2-10	1.3E-8	1.4E-8	1.5E-8	1.4E-8
2-11	2.2E-7	1.4E-7 +	2.8E-7 +	2.1E-7
2-12	2.5E-8	3.6E-8 +	4.4E-8 +	3.4E-8
2-13	***	1.5E-8 +	8.7E-8 +	
2-14	***	2.1E-8 +	3.1E-8 +	
Geo. Mean	2.9E-8	4.5E-8	5.5E-8	4.3E-8

\+ Indicates tests performed on 9/4/86 and 9/23/86.
* Indicates development by surging. Others were developed by only bailing.
*** Insufficient data to produce a hydraulic conductivity.

Table 12. Comparison of mean hydraulic conductivity values obtained by the methods of Hvorslev (1951), Cooper, et al (1967) - Papadopulos, et al (1973) and Bouwer and Rice (1976) from the the three bail tests and one slug test performed on the wells at site 1.

WELL NO.	HVORSLEV METHOD K, (cm/s)	COOPER, ET AL. K, (cm/s)	BOUWER AND RICE K, (cm/s)
1-1	8.6E-7	1.9E-6	7.1E-7
1-2	8.4E-8	3.9E-7	6.0E-8
1-3	4.0E-7	1.4E-6	2.7E-7
1-4	2.4E-7	6.3E-7	1.7E-7
1-5	2.5E-7	3.2E-6	3.4E-7
1-6	2.0E-7	1.7E-7	1.0E-7
Geometric Mean K	2.7E-7	9.4E-7	2.0E-7

Table 13. Comparison of mean hydraulic conductivity values obtained from methods of Hvorslev (1951), Cooper, et al (1967) - Papadopulos, et al (1973), and Bouwer and Rice (1976) from initial recovery and two bail tests performed on the wells at site 2.

WELL NO.	HVORSLEV METHOD K, (cm/s)	COOPER, ET AL., K, (cm/s)	BOUWER AND RICE K, (cm/s)
2-1	3.2E-8	6.6E-8	2.3E-8
2-2	2.2E-7	5.5E-7	1.3E-7
2-3	1.6E-7	4.3E-7	1.1E-7
2-4	1.4E-7	6.0E-7	1.9E-7
2-5	3.0E-8	6.0E-8	2.1E-8
2-6	2.2E-8	5.2E-8	1.6E-8
2-7	1.0E-7	3.3E-7	7.7E-8
2-8	1.8E-8	4.5E-8	1.4E-8
2-9	3.1E-8	7.0E-8	2.2E-8
2-10	2.0E-8	3.8E-8	1.4E-8
2-11	2.5E-7	5.1E-7	2.1E-7
2-12	4.9E-8	1.3E-7	3.4E-8
2-13	5.0E-8	1.7E-7	3.6E-8
2-14	3.6E-8	5.8E-8	2.6E-8
Geometric Mean K	5.8E-8	1.4E-7	4.3E-8

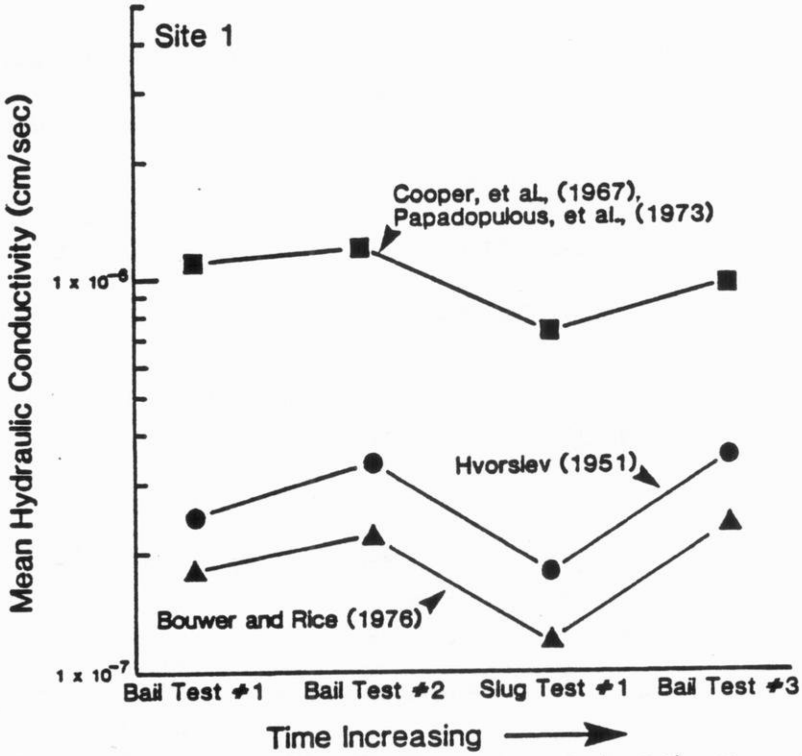

Figure 24. Comparison of Bouwer and Rice (1976), Hvorslev (1951), and Cooper, et al (1967) - Papadopulous, et al (1973) calculated hydraulic conductivities from bail-slug test data at site 1.

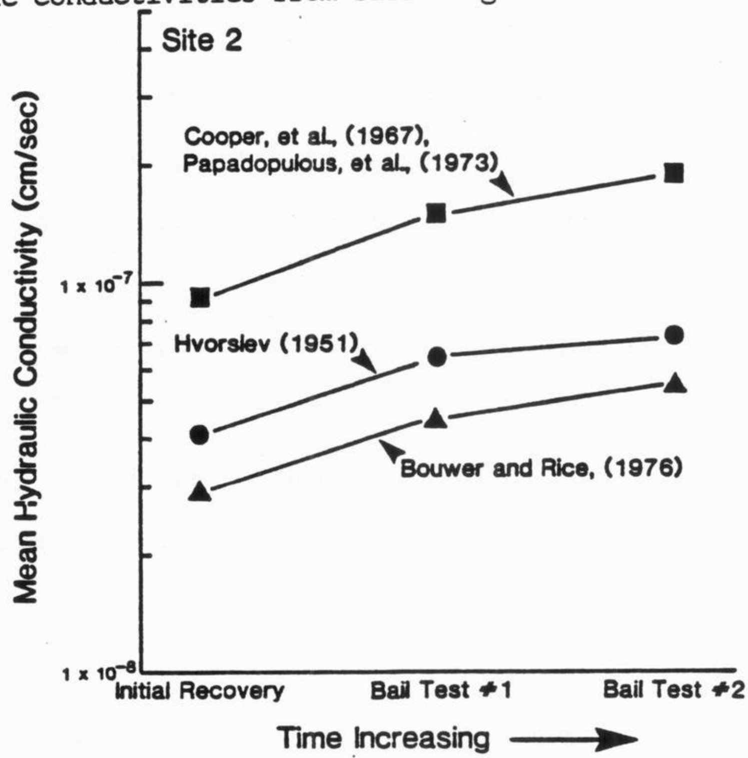

Figure 25. Comparison of Bouwer and Rice (1976), Hvorslev (1951), and Cooper, et al (1967) - Papadopulous, et al (1973) calculated hydraulic conductivities from bail-slug test data at site 2.

circumstances $\ln(R_e/r_w)$ will be less than $\ln(L/r_w)$. The only exceptions to this are when the piezometer is placed very close to the bottom of the aquifer, in contact with the impermeable boundary (i.e. when H = D). In those cases $\ln(R_e/r_w)$ will be greater than $\ln(L/r_w)$ if L/D is greater than about 0.2 and L/r_w is less than 64. The smallest values of $\ln(R_e/r_w)$ were about 0.6 times $\ln(L/r_w)$ and were obtained for monitoring wells that are installed at the water table. For the piezometers installed for the field portion of this study, $\ln(R_e/r_w)/\ln(L/r_w)$ values of about 0.70 to 0.75 were expected. This means that hydraulic conductivities as computed using the Bouwer and Rice (1976) method should be equal to about 0.70 to 0.75 times the hydraulic conductivities obtained with the Hvorslev (1951) method. Comparison of the overall mean Hvorslev (1951) and Bouwer and Rice (1976) hydraulic conductivities produces a $\ln(R_e/r_w)/\ln(L/r_w)$ value of 0.74 at both sites. This is in very good agreement with what should be expected from comparison of the two analytical solutions.

Hydraulic conductivities calculated by the method of Hvorslev and Bouwer and Rice are generally less than Cooper, et al., (1967) - Papadopulos, et al., (1973) by a factor of two to three (Table 13 and 14).
Values of Hvorslev (1951) hydraulic conductivity calculated in this study differ from Cooper, et al.,

(1967) - Papadopulos, et al., (1973) calculated hydraulic conductivities by a factor of 2.5 at site 1 to 3.5 at site 2. This difference in hydraulic conductivity has been observed in studies of glacial till in western New York (Prudic, 1982) and south central Saskatchewan (Keller, et al., 1986).

Prudic (1982) and Keller, et al. (1986) both attribute the greater hydraulic conductivities from the Cooper et al. (1973) method to the assumption of radial flow. The Hvorslev (1951) solution assumes an elliptical flow field around the well screen and the allowance for the additional flow in the vertical direction decreased the computed hydraulic conductivity. The Bouwer and Rice (1976) solution is more difficult to evaluate in this context because it uses a one-dimensional radial flow equation but computes the effective radius from a two-dimensional (r and z) analogue model. This mixing of dimensions creates some confusion in the physical interpretation of their solution.

A histogram of the mean hydraulic conductivity calculated from the wells at site 1 and site 2 versus frequency shows a bimodal distribution of hydraulic conductivity (Fig. 26). The results from site 1 seem to indicate a normal distribution. However, the results from site 2 indicate a bimodal distribution of hydraulic conductivity. The hydraulic conductivity results of site 1 seem to coincide with the second mode

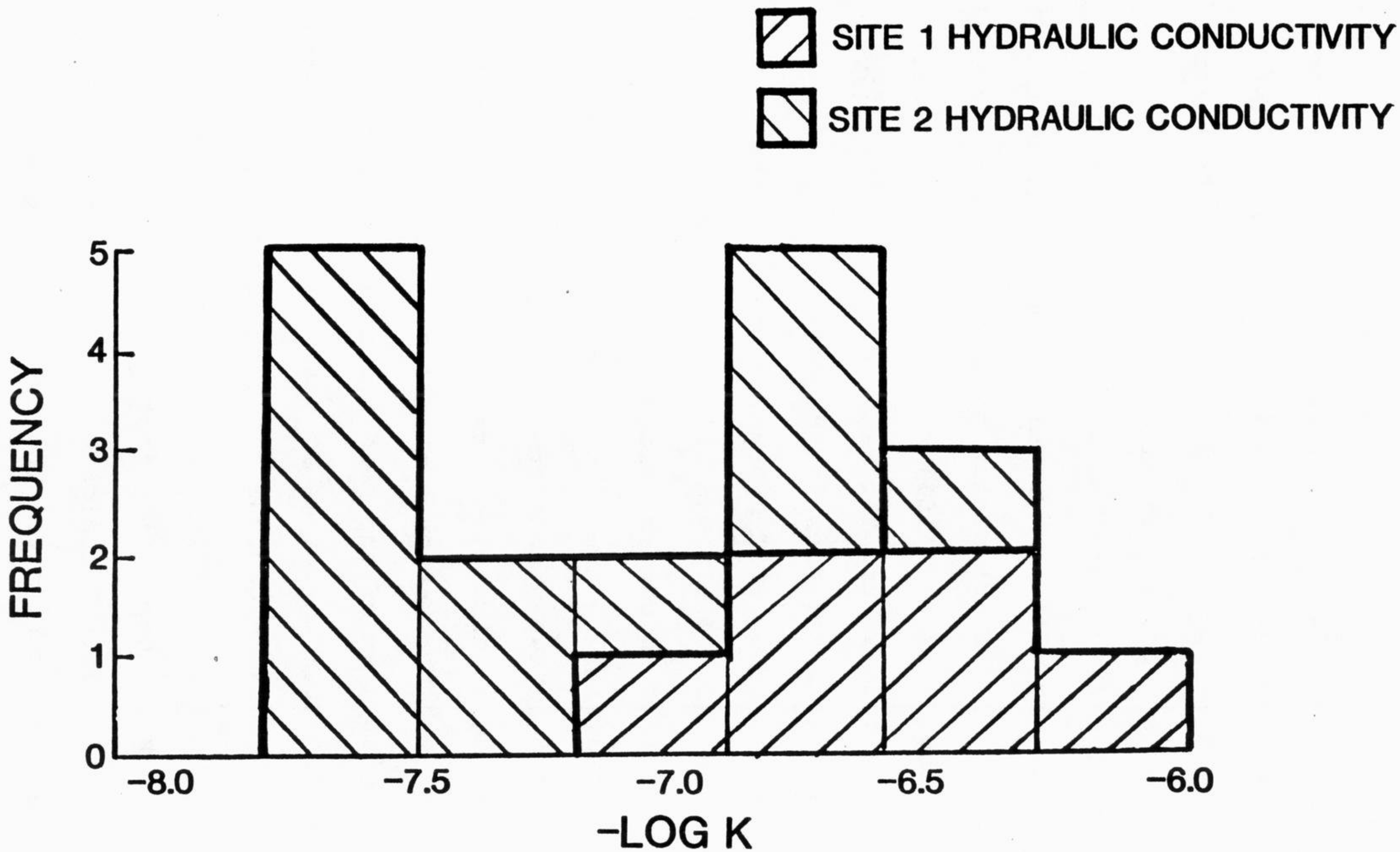

Figure 26. Histogram of the mean hydraulic conductivity results, K, for each well at site 1 and site 2 as calculated by the method of Hvorslev (1951). The hydraulic conductivity results obtained from the porous stone piezometers are excluded.

or the higher hydraulic conductivity results obtained from the wells at site 2.

Because of the bimodality of the hydraulic conductivity results, the physical properties and intergranular hydraulic conductivity of soil samples obtained from the two field sites have been evaluated. The laboratory results of sites 1 and 2 are used to make inferences to the continuity or discontinuity of the till present at both sites. In addition, comparisons of the field and laboratory hydraulic conductivity are used to evaluate the observed bimodal distribution of hydraulic conductivity observed at site 2.

LABORATORY TESTING OF SOIL SAMPLES

Laboratory tests were performed on the soil samples collected by Shelby tube at each of the field sites. Soil tests on these samples included engineering soils classification, one-dimensional consolidation testing, and triaxial tests. Engineering classification of the soils included specific gravity, grain-size, and Atterberg limits testing. These properties allow inferences to the homogeneity or heterogeneity of the soil. One-dimensional consolidation and triaxial cell tests of the soils provide hydraulic conductivity and specific storage values for the soil.

Engineering Soils Classification

Segments of each Shelby tube sample were analyzed to determine grain-size distribution, specific gravity, liquid limit, plastic limit, and shrinkage limit. Grain-size distribution was determined using a combined sieve and hydrometer analysis (ASTM Designation: D422). Total grain-size distribution curves for the tested intervals at site 1 and site 2 (Fig. 27 and 28) show consistent grain-size over the screened interval, and suggest a slight fining of the till units with depth. Because the distribution of grain-size is in good agreement and no stringers of sand of gravel were found over the sampled intervals, the till can be

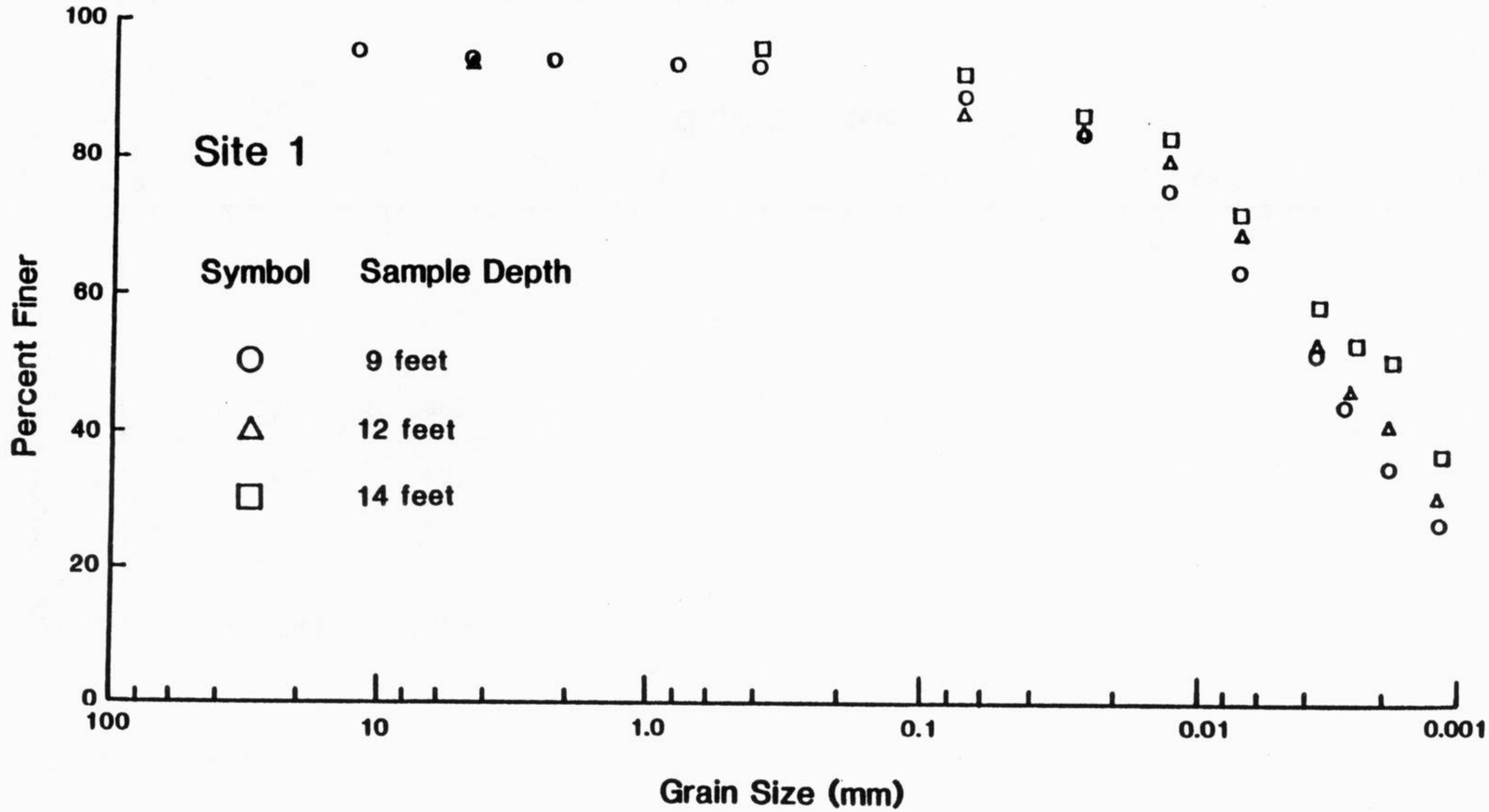

Figure 27. Grain-size distributions of the soil samples at site 1.

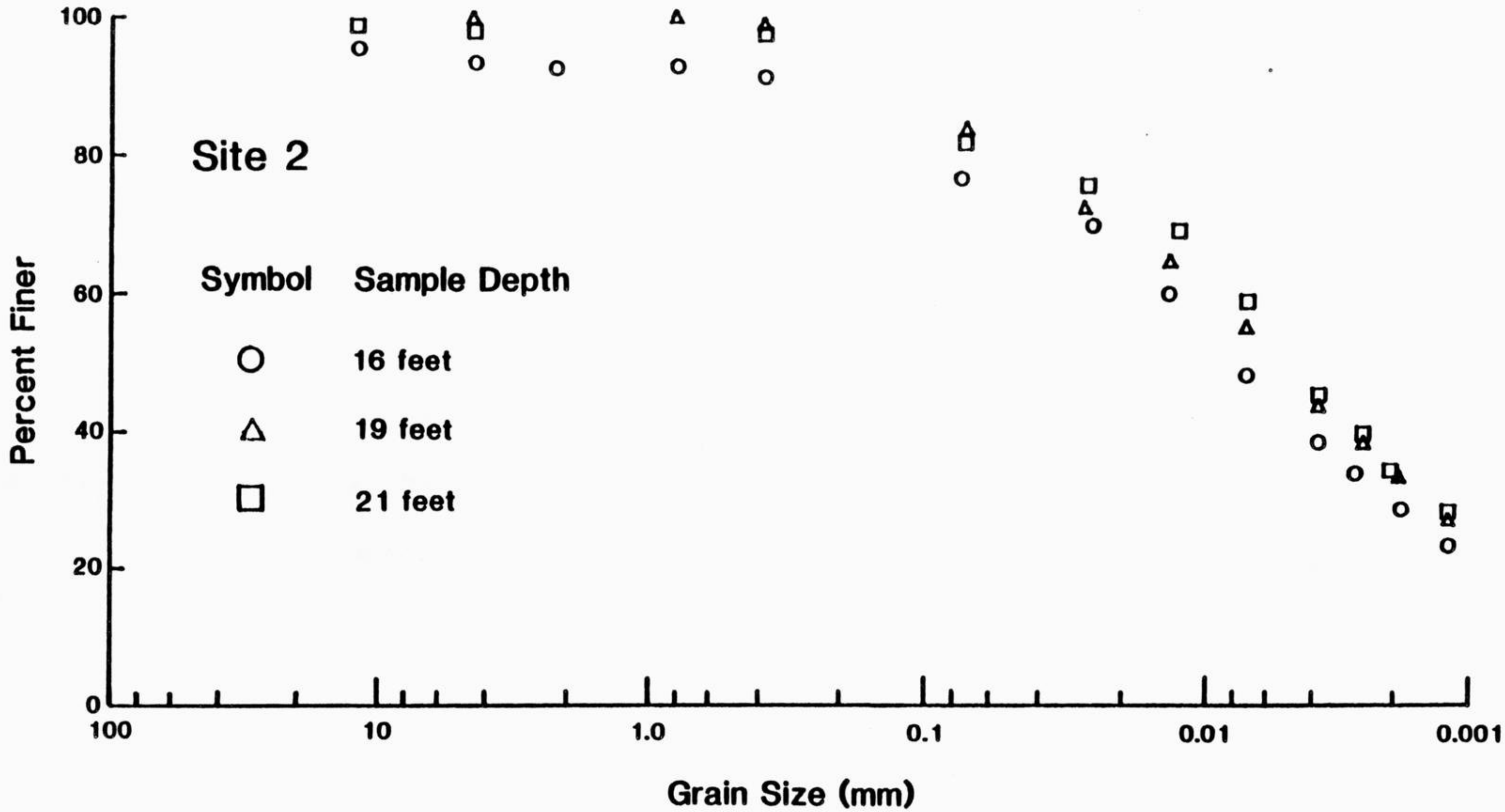

Figure 28. Grain-size distributions of the soil samples at site 2.

classified as being homogeneous. The Unified Soils Classification System name for the formation material is a gravelly-sandy lean clay. Comparison of the mean grain-size distributions shows the till units at sites 1 and 2 are very similar, but site 1 tends to have a greater amount of fines over the screened interval (Fig. 29).

Specific gravity of the soils was determined by the pycnometer method (ASTM Designation: D854). The specific gravity of soil solids is used in the calculations of total unit weight and void ratio in the consolidation testing. In general, the specific gravity of soil solids ranged from 2.72-2.75 gm/cm^3. There were no distinct trends of specific gravity with depth.

A relative measure of soil consistency with increasing moisture is defined by the Atterberg limits (ASTM Designation: D4318). The Atterberg limits, defined by the shrinkage, plastic, and liquid limit, show an increasing trend with depth at both sites 1 and 2 (Fig. 30 and 31). This trend compliments the increasing amount of fines with depth determined from the grain-size distribution (Fig. 27 and 28).

One Dimensional Consolidation Testing

One-dimensional time consolidation testing of soil samples was performed using a Wykeham Farrance Consolidation Test machine and the procedures outlined

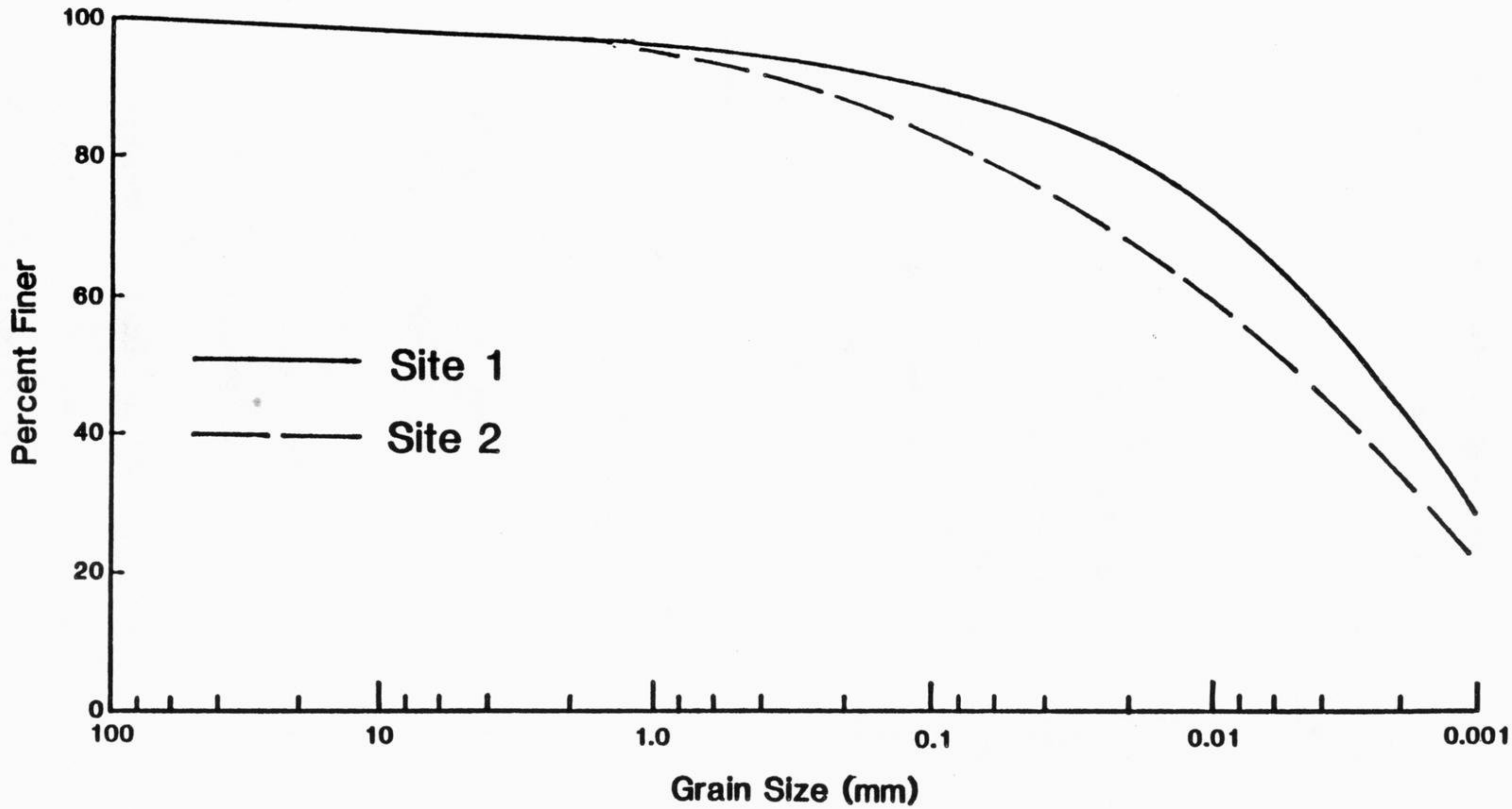

Figure 29. Comparison of the mean grain-size distribution curves for soil samples at sites 1 and 2.

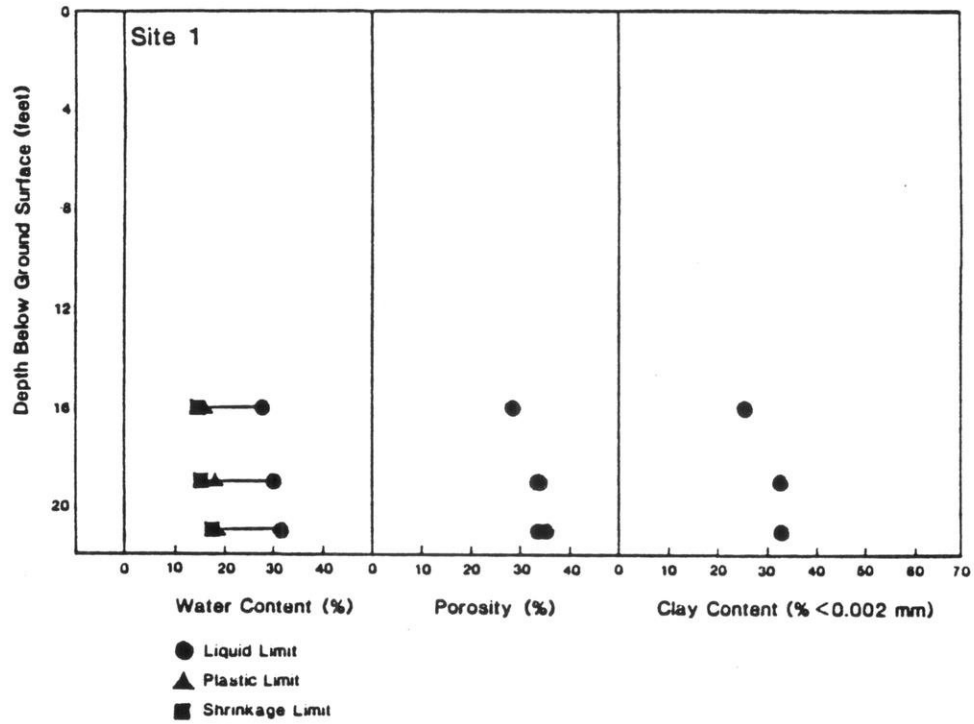

Figure 30. Laboratory results of Atterberg limits, porosity, and clay content of the soil samples at site 1.

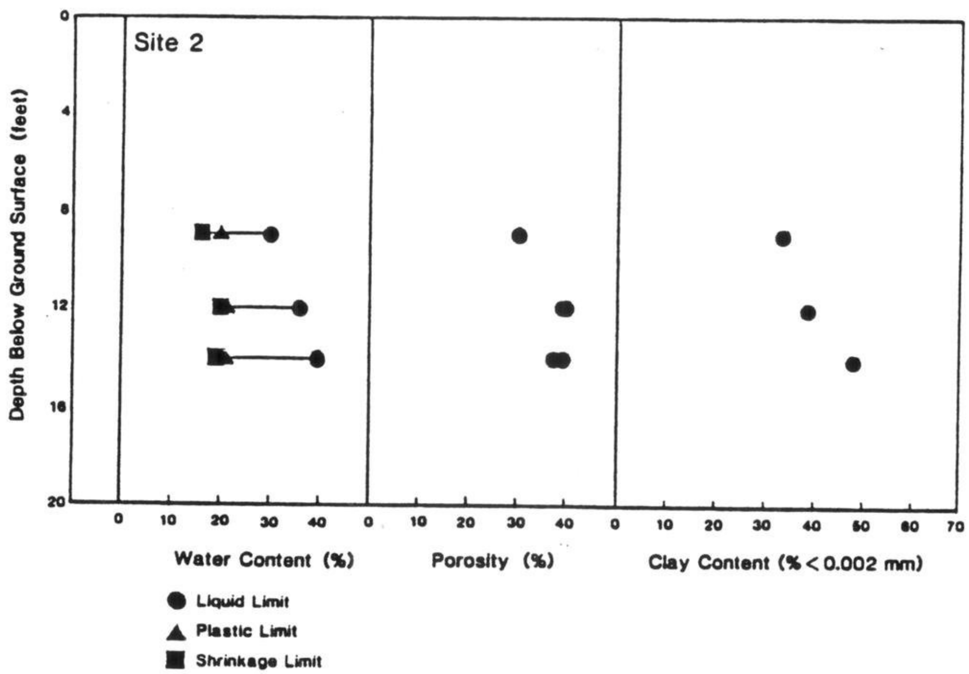

Figure 31. Laboratory results of Atterberg limits, porosity, and clay content of the soil samples at site 2.

in ASTM Designation: D2435. The consolidometers used in this study were designed such that the soil sample dimensions and load increments were in English units of inches and pounds, respectively. Standard conversions were used to determine values of hydraulic conductivity and specific storage in metric units of cm/sec and m^{-1}, respectively.

In general, a soil sample was trimmed to fit a stainless steel proving ring (3 inch diameter x 0.75 inch thick) and fitted to the consolidometer with an 11:1 lever arm ratio. The sample was then loaded with an initial load of 2 lbs. and the change in height of the sample was measured with time over a 24 hour period. Subsequent applied loads which doubled the working load (i.e. 2, 4, 8, 16, and 32 lbs.) were applied and the deformation-time information recorded. The sample was unloaded in increments of 32, 24 and 6 lbs. to obtain a rebound curve for the sample.

Theory of One-Dimensional Time-Consolidation

The underlying theory of one-dimensional time-consolidation testing by Terzaghi (1943) is governed by the following assumptions:

1) The soil is homogeneous, isotropic, and fully saturated,
2) Both soil and water are incompressible,
3) Water flow is only in one direction,
4) Volume change is only in one direction,

5) Water flow is laminar as defined by Darcy's Law, and,

6) Poisson's ratio and the elastic modulus are assumed constant.

The validity of one-dimensional time-consolidation results depend on the integrity of these stringent assumptions (Tavenas, et al., 1983a).

Calculations were made to determine the intitial void ratio, water content, total unit weight, and initial saturation of the samples from time-deformation information for the various applied loads. The initial void ratio, water content and unit weight of the soil were used to calculate the total stress on the in-situ sample. From calculations of the effective total stress of the in-situ samples, the in-situ void ratio was used to calculate hydraulic conductivity and specific storage of the soil samples (Appendix IV).

<u>Permeability from Consolidaton Data</u>

The coefficient of consolidation (c_v) and coefficient of volume compressibility (m_v) must be calculated for each load increment to determine the hydraulic conductivity of a soil sample. The coefficient of volume compressibility is simply the ratio of the change in strain over the change in stress for each load increment. The coefficient of consolidation was graphically determined by the Taylor Square Root of Time (Taylor, 1948) and Casagrande Logarithm of Time (Casagrande and Fadum, 1940) methods. For each method, the load increment data are plotted up as dial reading (in thousandths of an inch) versus time. For the Taylor Square Root of Time method, the data are plotted as the dial reading versus the square root of time. The coefficient of consolidation (c_v), based on the time for 90 percent primary consolidation to occur, was determined by the empirical relationship (Taylor, 1942):

$$c_v = \frac{T_{90} * (H/2)^2}{t_{90}} , \tag{14}$$

where:
- c_v = coefficient of consolidation (in^2/min)
- T_{90} = time factor for 90 percent consolidation = 0.848 (dimensionless)
- H = average height of the sample over the load increment (inches)
- t_{90} = graphically-derived time for 90 percent primary consolidation to occur (min)

The coefficient of consolidation was likewise determined for each load increment by this method.

The Casagrande Logarithm of Time method is based on the time for 50 percent of primary consolidation to occur. For this method, the load increment data are plotted on a semi-log plot of dial reading versus log time. The coefficient of consolidation is determined from the empirical relationship (Casagrande and Fadum, 1940):

$$c_v = \frac{T_{50} * (H/2)^2}{t_{50}} , \tag{15}$$

where: c_v = coefficient of consolidation (in_2/min)
T_{50} = time factor for 50 percent consolidation = 0.197 (dimensionless)
H = average height of the sample over the load increment (inches)
t_{50} = graphically-derived time for 50 percent primary consolidation to occur (min)

Again, the coefficient of consolidation was determined for each load increment.

The vertical permeability (k_v) of the sample for each load increment was determined from the empirical relationship (Terzaghi, 1943):

$$k_v = c_v * m_v * \gamma_w \ , \qquad (16)$$

where: k_v = vertical hydraulic conductivity (cm/s)
c_v = coefficient of consolidation (cm^2/sec)
m_v = coefficient of volume compressibility (cm^2/gm)
γ_w = unit weight of water (gm/cm^3)

For each method, the permeability is plotted versus void ratio for each increment (Fig. 32). The data should plot as a straight line, however, due to instrument error, operator error and/or graphical interpretation, this is generally not the case. The in-situ permeability is graphically chosen from the void-ratio - permeability line at its intersection with the estimated in-situ void ratio for a given sample.

In general, permeabilities determined by the Taylor Square Root of Time method are higher than those determined by the Casagrande Logarithm of Time method

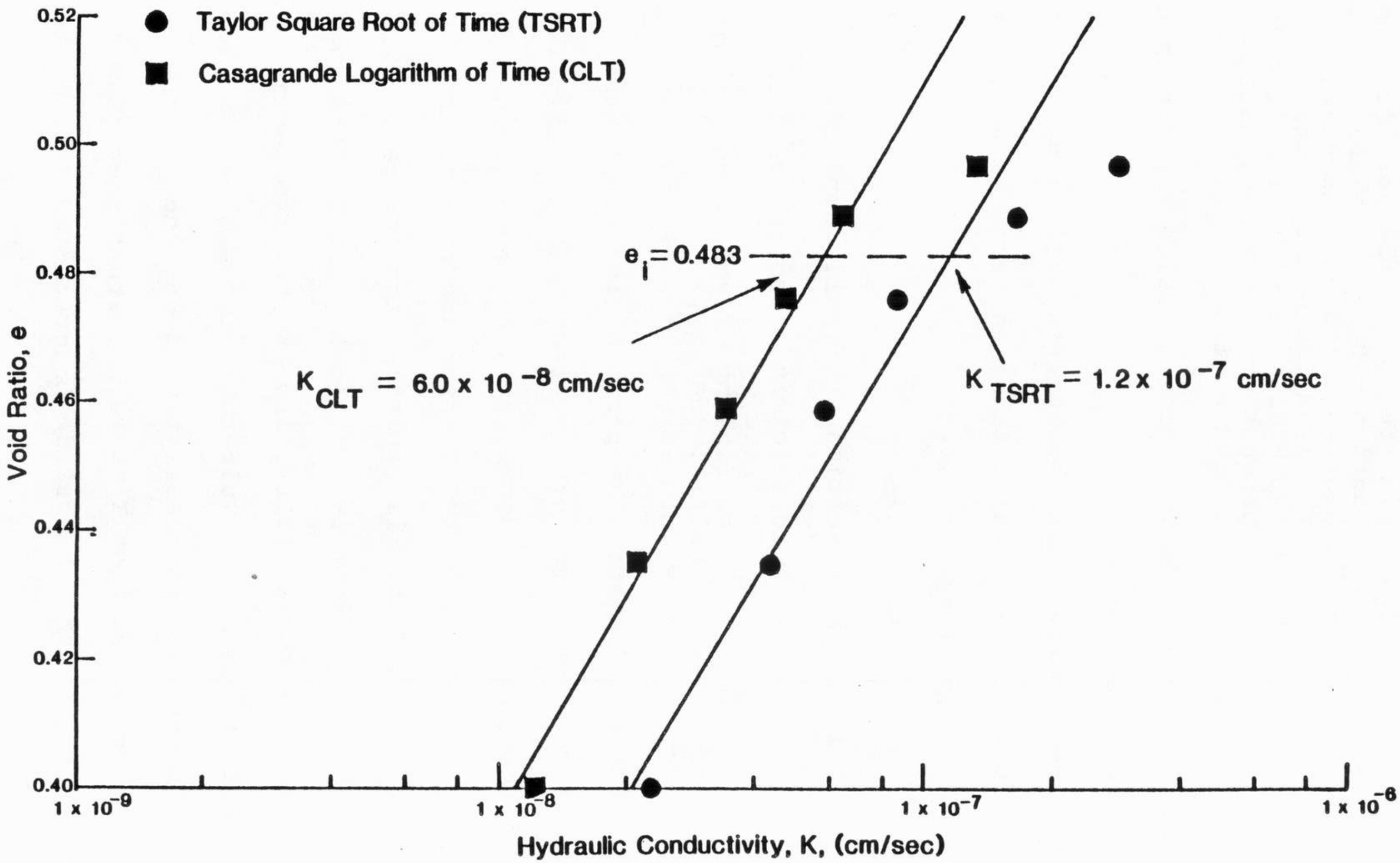

Figure 32. Determination of hydraulic conductivity from time-consolidation void ratio - permeability plot. The soil sample is from borehole 2-6 and depth of 19 feet. e_i is the in-situ void ratio.

(Tables 14 and 15). The former method is generally preferred because of the assumptions made in Casagrande's graphical interpretation (Tavenas, et al. 1983).

The results of 10 time-consolidation tests performed on the soil samples at sites 1 and 2 (Tables 14 and 15) (Appendix IV) do not show any trend with depth. The hydraulic conductivity as calculated by the two methods; however, do agree within a half of order of magnitude.

Specific Storage from Consolidation Data

Specific storage, S_s, is defined as the amount of water released from or taken into storage per unit change in head. Consolidation test data can be used to determine specific storage of the tested soil sample. Two relationships have been used to empirically determine the specific storage of the soil. The first method used involves the empirical relationship (Terzaghi, 1943):

$$c_v = k_v/S_s \tag{16}$$

or rewritten to define specific storage (S_s),

$$S_s = k_v/c_v \tag{17}$$

where: S_s = specific storage (cm^{-1}) or (m^{-1})
k_v = vertical hydraulic conductivity (cm/sec)
c_v = coef. of consolidation (cm^2/sec)

TABLE 14. Time-consolidation hydraulic conductivities from soil samples at site 1.

SAMPLE DEPTH (FEET)	TSRT[a] K, (cm/s)	CLTM[b] K, (cm/s)
9	2.4E-7	1.3E-7
12	1.6E-7	7.1E-8
	7.9E-8	3.8E-8
14	1.3E-7	6.0E-8
	7.4E-8	5.3E-8

a - Taylor Square Root of Time Method
b - Casagrande Log of Time Method

TABLE 15. Time-consolidation hydraulic conductivities from soil samples at site 2.

SAMPLE DEPTH (FEET)	TSRT[a] K, (cm/s)	CLTM[b] K, (cm/s)
16	8.2E-8	3.8E-8
19	7.6E-8	3.8E-8
	1.2E-7	6.0E-8
21	6.2E-8	3.0E-8
	1.9E-7	6.4E-8

a - Taylor Square Root of Time Method
b - Casagrande Log of Time Method

To determine the specific storage of the "in-situ" sample from this relationship, a plot of the void-ratio versus coefficient of consolidation is made for the various load increments. The in-situ void ratio is known from the void-ratio versus pressure plot and the coefficient of consolidation which meets the in-situ void-ratio is chosen (Fig. 33). The specific storage can be determined from eq. (17) knowing the in-situ permeability and coefficient of consolidation.

The second method to determine specific storage is developed from the empirical relationship (Terzaghi, 1943):

$$S_s = (a_v * \gamma_w)/(1 + e_i) \tag{18}$$

where:
S_s = specific storage (ft^{-1}) or (m^{-1})
a_v = coefficient of compressibility (psi^{-1})
γ_w = unit weight of water (lb/in^3)
e_i = in-situ void ratio

This method uses the void-ratio versus pressure plot (Fig. 34). The coefficient of compressibility, (a_v), is defined as the ratio of the change in void ratio to change in pressure along the void-ratio - pressure curve. Because the void-ratio - pressure plot is a curve, the coefficient of compression required is chosen as the tangent to the point equal to the in-situ total stress of the sample. The in-situ stress is required because a change in stress immediately away from the in-situ stress will cause water to be released from or taken into storage.

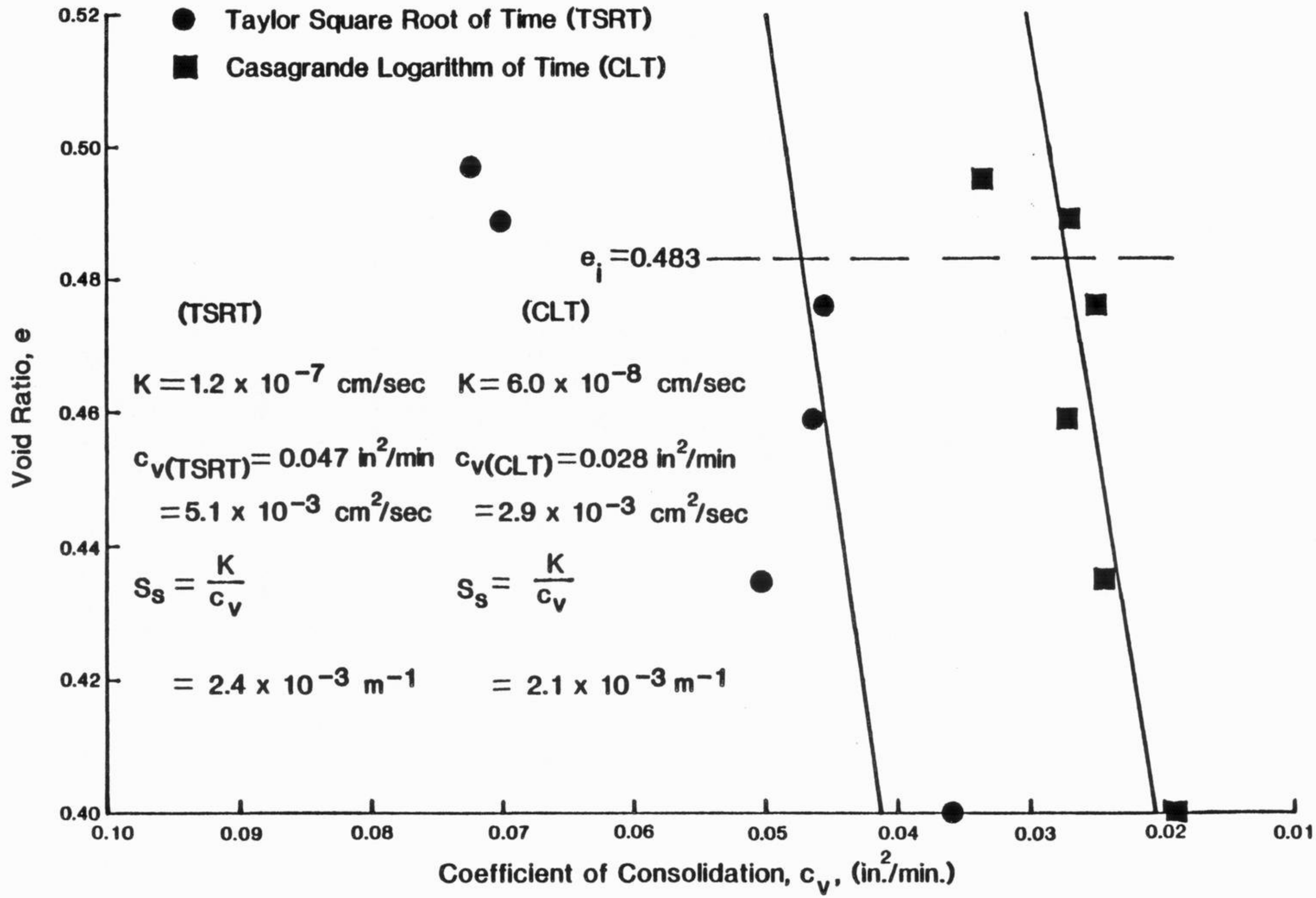

Figure 33. Determination of specific storage from a time-consolidation void ratio - coefficient of consolidation plot. The soil sample is from borehole 2-6 and depth of 19 feet. e_i is the in-situ void ratio.

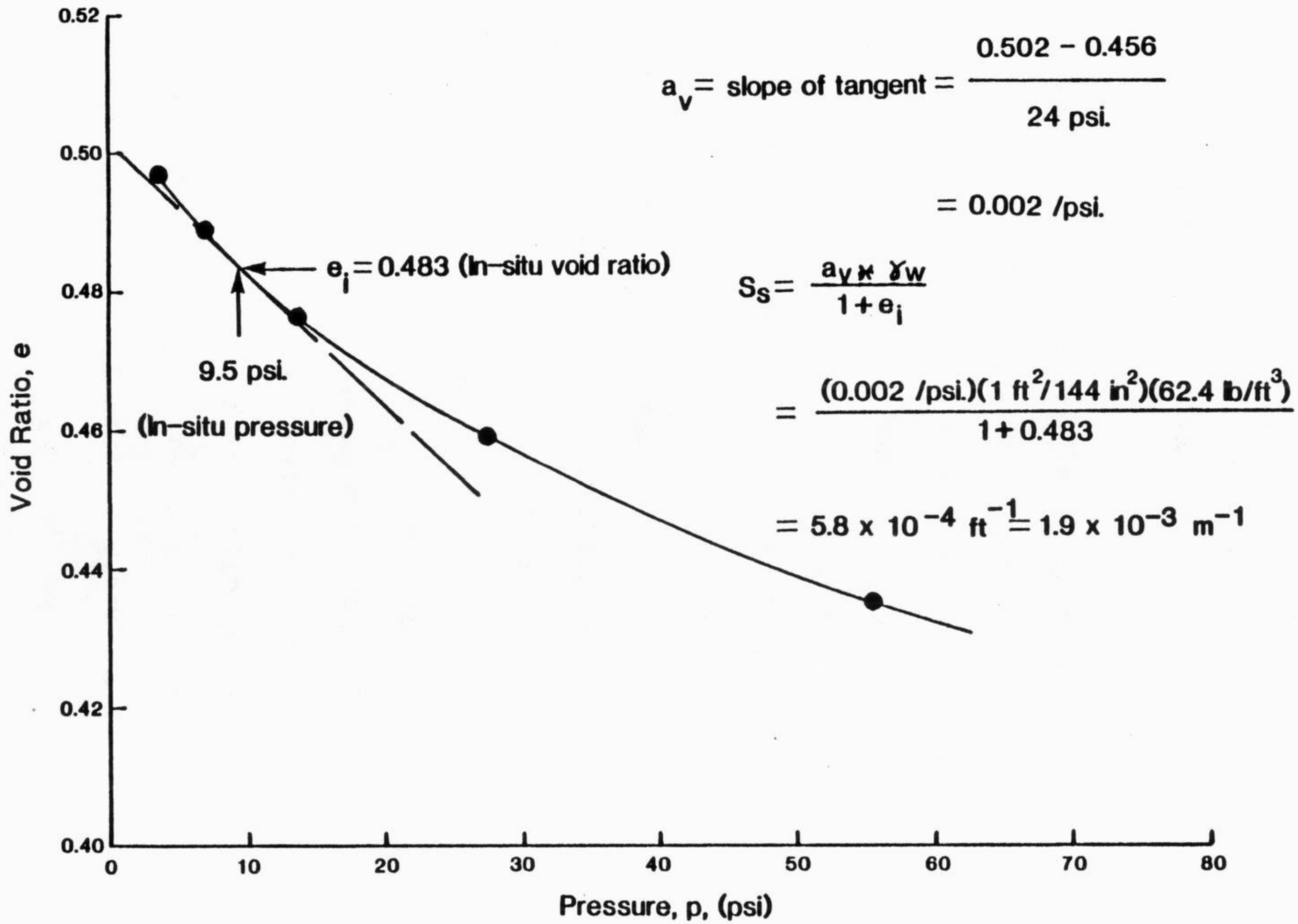

Figure 34. Determination of specific storage from a time-consolidation void ratio - pressure plot. The soil sample is from borehole 2-6 and depth of 19 feet.

The results of specific storage as determined from both of these methods are presented in Table 16 and 17. The specific storage results from the two empirically derived relationships agree within 20 percent.

Triaxial Permeability Testing

Method

The soil samples of site 1 and site 2 were also tested in the triaxial cell to determine the intergranular hydraulic conductivity of the soil. The general procedure followed the method and recommendations outlined by Tavenas, et al. (1983a). A length of Shelby tube sample was trimmed to a 3.5 cm diameter and length of 5.7 to 7.0 cm to fit the triaxial cell assembly. The cylindrical sample was then mounted underwater with porous stones at each end. The porous stones were previously boiled to assure proper saturation.

The sample and porous stones were enclosed with two snug fitting rubber membranes. A layer of petroleum jelly was applied between the membranes to prevent losses of water by diffusion through the rubber membrane. The mounted sample was enclosed in the triaxial cell assembly and the inner chamber was flooded. A three pot mercury system was used to apply a confining pressure on the sample and the gradient across the sample.

TABLE 16. Time-consolidation specific storage results from soil samples at site 1.

SAMPLE DEPTH (FEET)	S_s^a (a_v) (/m)	S_s^b $(k/cv)_T$ (/m)	S_s^c $(k/cv)_C$ (/m)
9	2.5E-3	3.6E-3	3.4E-3
12	2.3E-3	3.8E-3	2.8E-3
	3.3E-3	3.1E-3	4.8E-3
14	1.6E-3	2.1E-3	1.8E-3
	3.2E-3	3.1E-3	3.3E-3

a - Coefficient of Compressibility Method
b - Taylor Square Root of Time Method
c - Casagrande Log of Time Method

TABLE 17. Time-consolidation specific storage results from soil samples at site 2.

SAMPLE DEPTH (FEET)	S_s^a (a_v) (/m)	S_s^b $(k/c_v)T$ (/m)	S_s^c $(k/c_v)_C$ (/m)
16	1.7E-3	1.9E-3	1.7E-3
19	2.5E-3	2.7E-3	2.9E-3
	1.9E-3	2.4E-3	2.1E-3
21	1.9E-3	2.8E-3	2.4E-3
	2.6E-3	2.5E-3	2.8E-3

a - Coefficient of Compressibility Method
b - Taylor Square Root of Time Method
c - Casagrande Log of Time Method

A three channel Wykeham Farrance digital transducer (model 12724) was used to measure the confining cell and gradient pressures applied on the sample. The transducer was calibrated with the mercury pot system pressure gage prior to attaching the transducer to the triaxial cell assembly. The transducer was calibrated at 0, 5, and 10 psi. This is in the range of in-situ overburden pressures on the soil samples.

After calibration, the transducer was connected in series to the lines leading to the triaxial cell. A confining pressure was applied which was equal to the calculated in-situ overburden pressure (Appendix IV). A hydraulic gradient was then created by applying a pore water pressure difference across the sample. The greatest pore water pressure applied to one end of the sample had to be less than the confining pressure to prevent preferred flow between the sample and the rubber membrane. The sample was allowed to equilibrate for two days after application of the gradient as recommended by Tavenas, et al. (1983a). Flow was measured in a 100 cm^3 burrette assembled in series between the mercury pot pressure and the transducer. Flow measurements were taken for seven to ten days following the two day waiting period.

To calculate the sample hydraulic conductivity by Darcy's Law:

$$K = \frac{-Q}{i * A} , \qquad (19)$$

the flow rate (Q), hydraulic gradient (i), and cross-sectional area of the sample (A) must be determined (Appendix V).

The flow rate is calculated from a plot of recorded measurements of volume (cm^3) versus time (sec). The slope of this line is the average flow rate Q (cm^3/sec) for the duration of the test.

The hydraulic gradient (i) was calculated knowing the length of the sample, the pore water pressure difference, and the unit weight of water by the following equation (Tavenas, et al., 1983a):

$$i = \frac{u}{H * \gamma_w} , \tag{20}$$

where: i = hydraulic gradient (in/in)
u = pore water pressure difference (psi)
H = sample length (in)
γ_w = unit weight of water = 62.4 lb/ft^3
= 3.6×10^{-2} lb/in^3

Results

Intergranular hydraulic conductivities as determined in the triaxial cell ranged from 1.3 to 3.0 x 10^{-8} cm/s (Tables 18 and 19). The applied hydraulic gradients (30-50) are much higher than gradients found in nature. However due to the low overburden pressures, this could not be avoided. The range of hydraulic conductivities at site 1 (1.3 to 3.0 x 10^{-8} cm/sec) and site 2 (2.0 to 2.8 x 10^{-8} cm/sec) show very

Table 18. Triaxial cell permeabilities of soil samples at site 1.

Sample Depth (ft)	Overburden Pressure (psi)	Confining Pressure (psi)	Hydraulic Gradient	Hydraulic Conductivity (cm/s)
9	5.0	7.0	30.	1.8×10^{-8}
12	6.5	7.0	49.	3.0×10^{-8}
14	7.4	7.0	40.	1.3×10^{-8}

Table 19. Triaxial cell permeabilities of soil samples at site 2.

Sample Depth (ft)	Overburden Pressure (psi)	Confining Pressure (psi)	Hydraulic Gradient	Hydraulic Conductivity (cm/s)
16	8.0	8.0	40.	2.2×10^{-8}
19	9.5	9.5	49.	2.8×10^{-8}
21	10.6	10.5	40.	2.0×10^{-8}

good agreement for the same till unit at two different locations. However, the triaxial cell hydraulic conductivities show no trend with depth.

COMPARISON OF FIELD AND LABORATORY RESULTS

Hydraulic Conductivity

The previously determined laboratory time consolidation and triaxial cell hydraulic conductivities of the glacial till unit in this study are compared to their respective Hvorslev (1951) bail-slug test calculated values at site 1 (Fig. 35) and site 2 (Fig. 36). For each site, the laboratory hydraulic conductivity is plotted versus the respective sampling depth. The range of field hydraulic conductivity for each well is plotted relative to the screen midpoint. The screen midpoints are relative to the surface elevation (Appendix VI) of the first piezometer at each site (i.e. 1-1 and 2-1). Visual inspection of the field results does not suggest any trends of hydraulic conductivity relative to the position of the midpoint of the screen within the till.

Comparison of laboratory triaxial cell permeabilities and consolidation test permeabilities shows that the consolidation-derived permeabilities are 2 to 10 times greater than the triaxial cell values. Because triaxial cell tests are subject to a minimal amount of error compared to possible instrument error, operator error, and graphical error in time-consolidation tests (Tavenas, et al. 1983a), the triaxial cell permeabilities are a better estimate of

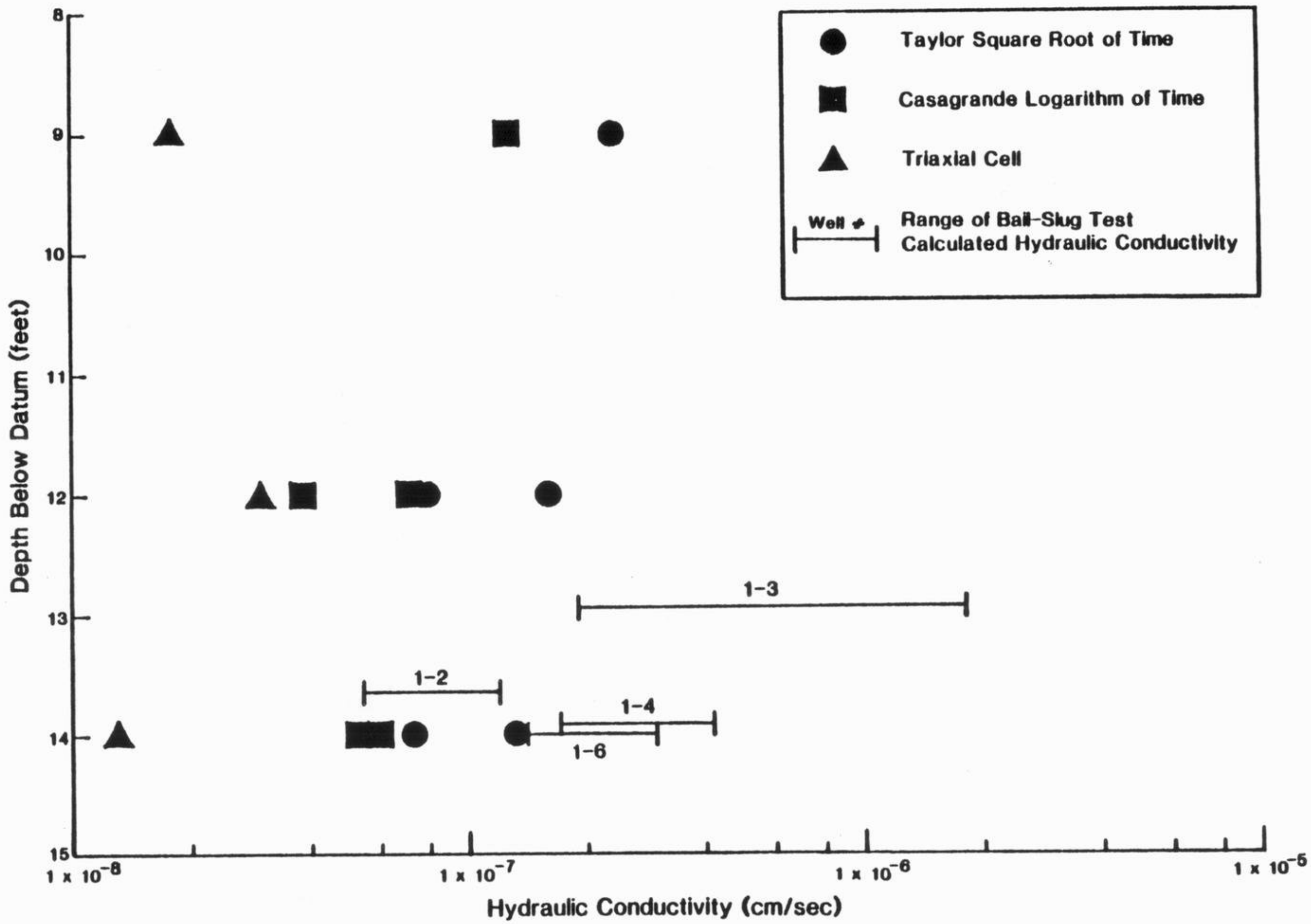

Figure 35. Range of Hvorslev (1951) hydraulic conductivities compared to laboratory time-consolidation and triaxial cell permeabilities at site 1. Hydraulic conductivities are plotted relative to the midpoints of the screens.

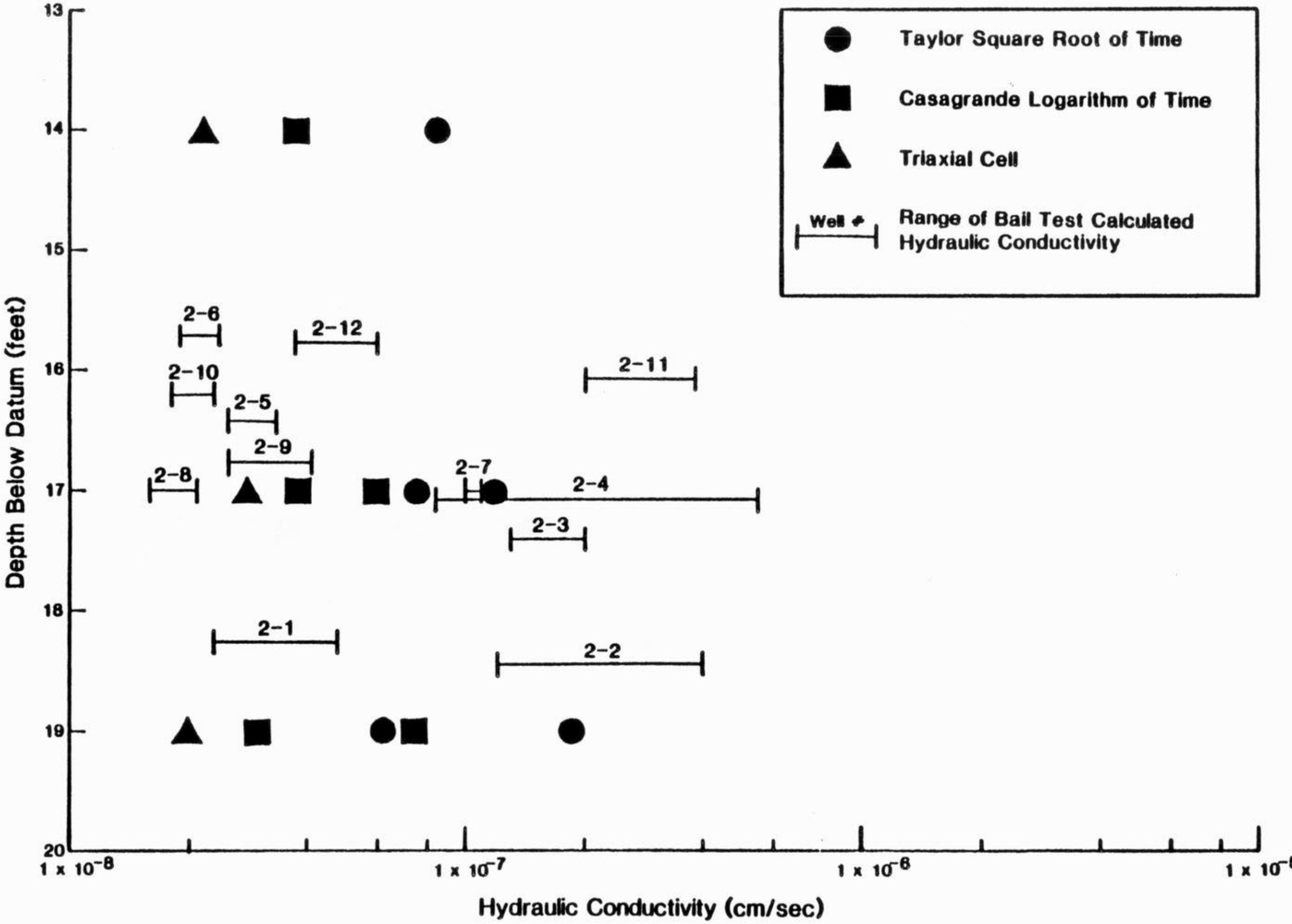

Figure 36. Range of Hvorslev (1951) hydraulic conductivities compared to laboratory time-consolidation and triaxial cell permeabilities at site 2. Hydraulic conductivities are plotted relative to the midpoints of the screens.

the intergranular hydraulic conductivity of the glacial till in this study.

A histogram of the mean hydraulic conductivity as calculated by the method of Hvorslev (1951) has shown a bimodal distribution of hydraulic conductivity (Fig. 37). Comparison of the mean triaxial cell results from sites 1 and 2 shows good agreement with the lower hydraulic conductivity results from site 2. However, comparison of the mean triaxial cell and bail-slug test hydraulic conductivities for site 1 seem to be unrelated. In fact, the triaxial cell results are approximately an order of magnitude lower than those calculated from bail - slug tests at site 1.

The bimodal distribution of hydraulic conductivity values at site 2 and an order of magnitude difference between the triaxial cell and bail-slug test results at site 1 suggests that there is some other factor than intergranular hydraulic conductivity controlling flow at these sites. Assuming the hydraulic conductivity of the bentonite seal and tamped clay are equal to or less than the hydraulic conductivity of the formation, primary or secondary structures in the till are controlling the observed higher hydraulic conductivities.

Primary structures include sand and gravel stringers present over the screened length of the well.

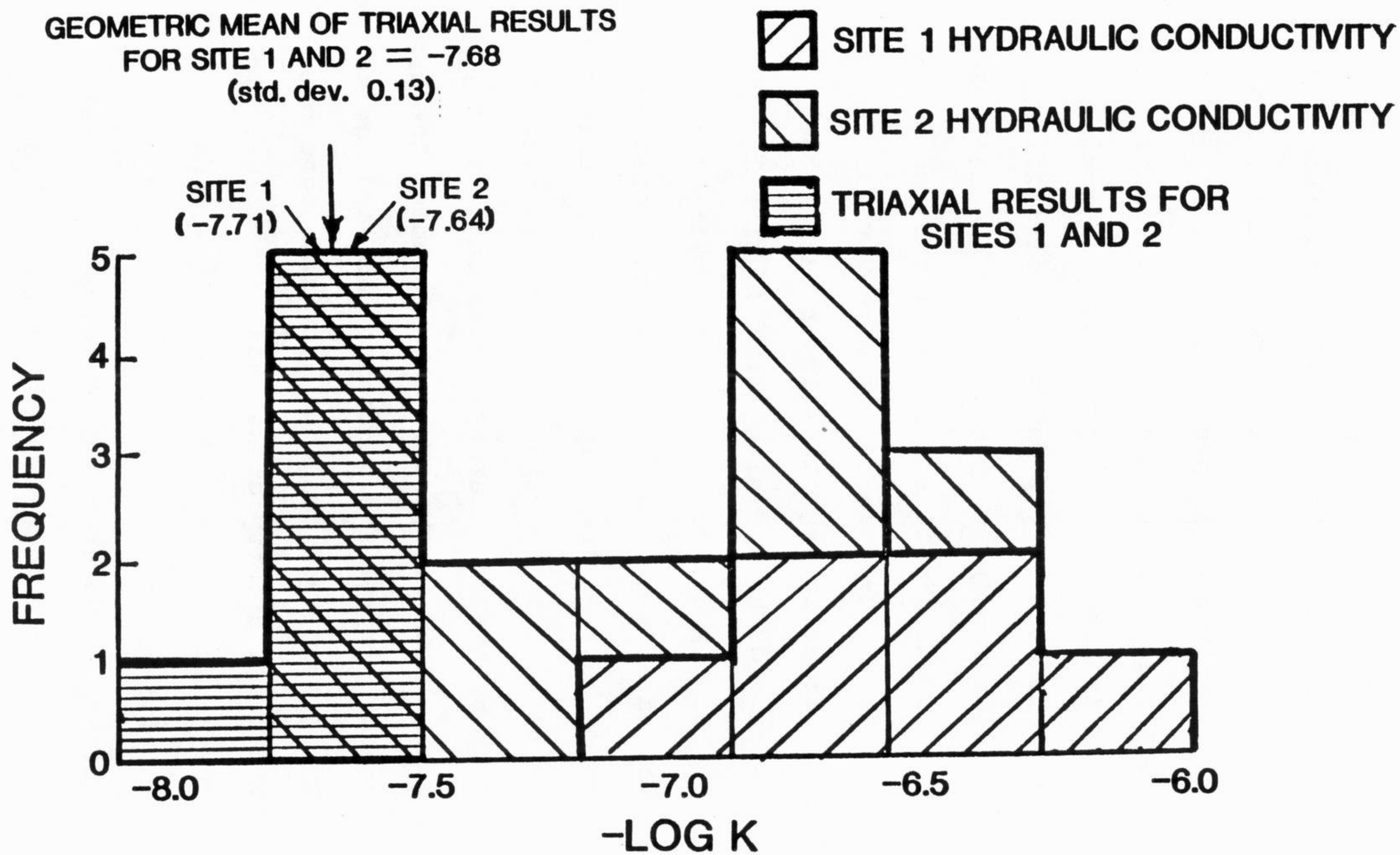

Figure 37. Histogram of mean hydraulic conductivity results of all slug-bail tests performed for each piezometer at sites 1 and 2 compared to the mean triaxial cell hydraulic conductivities of three soil samples at site 1 and three soil samples at site 2. Slug-bail test hydraulic conductivities are as calculated by Hvorslev (1951).

Evidence of such stringers was not observed in the trimmings from boreholes at sites 1 or 2 nor were they evident in the Shelby-tube samples of sites 1 or 2.

Secondary structures include fracturing or jointing of the till. In the laboratory, a Shelby tube sample taken from borehole 1-4 at a depth of 14 feet was observed to have one vertical fracture when the sample was being trimmed to fit the consolidometer. The fracture was noted to be lighter in color than the surrounding matrix. The fracture, however, could not be followed for the length of the Shelby-tube sample. Fractures were not observed in Shelby-tube samples taken from borehole 2-6 (depths of 16, 19 and 21 feet) at this site 2. However, well 2-6 produced hydraulic conductivities similar to the intergranular triaxial cell permeabilities (Fig. 36), so fractures would not be expected to be present in this borehole.

No visual evidence of fractures is present at site 2. However, fractures in the glacial till are suspected to control the hydraulic conductivity of the more highly conductive wells at sites 1 and 2 because: 1) a fracture was observed in hand sample from well 1-4, 2) comparison of the Hvorslev hydraulic conductivities from the wells at site 1 (which includes well 1-4) and range of higher Hvorslev hydraulic conductivities of site 2 are in very good agreement (Fig. 37), and 3) no stringers of sand or gravel were collected by Shelby tube or were observed in the drilling process from hand samples at sites 1 or 2.

Another line of evidence that suggest fracturing as the factor controlling the hydraulic conductivity of wells at site 1 and some of the wells at site 2 is the determination of preconsolidation pressures from consolidation test data (Appendix IV). The preconsolidation pressure presents the greatest pressure the soil was subjected to prior to sampling. The graphically determined pressures (Table 20) generally range from 35-40 psi. Note that these pressures range from 4 to 8 times the in-situ pressures of the samples. The enormous change in pressure due to glacial loading and unloading may have induced fractures in the glacial till unit.

Anisotropy is another possible explanation for the range of observed hydraulic conductivities at sites 1 and 2. Anisotropy will increase the hydraulic conductivity values obtained using the Hvorslev method, but a K_h/K_v ratio of 1000 is needed to increase the computed hydraulic conductivity by a factor of two. To increase the computed hydraulic conductivity by a factory of three, a K_h/K_v ratio of one million is required.

Anisotropy ratios for till units are not well

Table 20. Preconsolidation pressures of soil samples at site 1 and site 2 as calculated by the method of Casagrande (1940).

	Sample Depth (ft)	Overburden Pressure (psi)	Preconsolidation Pressure (psi)
Site 1			
	9	5.0	38.
	12	6.5	36. - 39.
	14	7.4	37. - 39.
Site 2			
	16	8.0	40.
	19	9.5	38. - 39.
	21	10.6	40. - 45.

known. Prudic (1982) argues that the fine-grained tills that he studied in New York state are isotropic. Studies of clays have produced anisotropy ratios ranging from 1 to 1.5 for marine clays and 1.5 to 40 for varved clays (Olson and Daniel, 1981, cited by Tavenas, et al., 1983b). An anisotropy value of 3 is reported for a varved clay (Chan and Kenny, 1973) and a value of 1 is reported for some Swedish clays (Larson, 1981). Tavenas, et al. (1983b) report an average ratio of 1.1 for the Champlain clays of Quebec and other Canadian clays and ratios of 2.2 to 2.5 for the Atchafalaya clay.

If the hydraulic conductivities obtained from the triaxial cell are assumed to be K_v and the value obtained from the Hvorslev (1951) analysis of the slug tests are K_h, anisotropy ratios of about 15 are possible. This hypothesis is not very satisfactory, however, because 1) the values from the slug tests are not consistent while those obtained from the triaxial cell are consistent, and 2) the anisotropy ratio of 15 can only account for a 39% difference in hydraulic conductivities and not the factors of 2 to 10 observed. Therefore, the differences between hydraulic conductivities obtained from triaxial tests and slug tests are more likely to be the result of fracture permeability than anisotropy.

The method of Cooper, et al. (1967) - Papadopulos, et al. (1973) has been shown to produce hydraulic conductivities which are approximately 3 times greater than hydraulic conductivities calculated by Hvorslev (1951). This means that the lower hydraulic conductivity values calculated by Cooper, et al. (1967) - Papadopulos, et al. (1973) will be three times greater than the triaxial cell values. Therefore, hydraulic conductivity as calculated by Hvorslev (1951) produces lower values which are in better agreement with the triaxial cell results.

In contrast, the method of Bouwer and Rice (1976) has been shown to produce hydraulic conductivity values which are 0.75 times those calculated by Hvorslev (1951). The lower values calculated by Bouwer and Rice (1976) will be in good agreement with the triaxial cell values.

The slug test results of this study suggest triaxial cell results are in good agreement with the lower range of hydraulic conductivities as calculated by Hvorslev (1951) and Bouwer and Rice (1976). However, if one is interested in determining the ability of this aquitard material to retain contaminants, the triaxial cell results are not the most conservative estimate of the bulk formation hydraulic conductivity. Bail test results of this study indicate field hydraulic conductivities which are 15 times the intergranular triaxial cell hydraulic conductivities.

Therefore, field tests should be given priority over the intergranular results when evaluating glacial till for waste containment.

Specific Storage

Specific storage has been determined in this study from laboratory time-consolidation test data and by the type curve approximation of Cooper, et al., (1967) - Papadopulos, et al., (1973). Specific storage determined from time-consolidation test data in this study ranged from 1.6×10^{-3} to 4.8×10^{-3} m^{-1} (Tables 17 and 18). These values cover the range of values observed by Domenico (1972) for stiff clay (Table 21). Similar time-consolidation test analysis of till in Manitoba, Saskatchewan, and Alberta which range from 9.2×10^{-4} to 2.2×10^{-3} m^{-1} (Grisak and Cherry, 1976) fall in the range of a stiff to plastic clay. These similarly derived values for specific storage are in very good agreement with the results of this study.

Specific storage has been determined by the method of Cooper, er al., (1967) - Papadopulous, et al., (1973). The values of specific storage ranged from 1.0×10^{-5} to 1.0×10^{-9} m^{-1} by the curve matching method. In general, the curve parameter "alphas" ranged from 10^{-4} to 10^{-8}. An alpha value of 10^{-3} would have to be used to obtain the range of specific storage values determined from consolidation test data. This was not

Table 21. Specific storage of various materials (from Domenico, 1972).

Material	S_s, m^{-1}
Plastic clay	2.0×10^{-2} - 2.6×10^{-3}
Stiff clay	2.6×10^{-3} - 1.3×10^{-3}
Medium-hard clay	1.3×10^{-3} - 9.2×10^{-4}
Loose sand	1.0×10^{-3} - 4.9×10^{-4}
Dense sand	2.0×10^{-4} - 1.3×10^{-4}
Dense sandy gravel	1.0×10^{-4} - 4.9×10^{-5}
Rock, fissured, jointed	6.9×10^{-5} - 3.3×10^{-6}
Rock, sound	Less than 3.3×10^{-6}

possible if the integrity of the curve matching technique was to be maintained. In general, the best match was obtained using the midpoints of the type-curves. Early portions of the field curves were generally steeper than the type curves. This may be a result of the assumptions and boundary conditions of the Cooper, et al., (1967) - Papadopulos, et al., (1973) analysis which pertains to determination of "aquifer" transmissivity and storage not holding true for the boundary conditions for such a well configuration in an aquitard material.

POTENTIAL SOURCES OF ERROR IN THE DETERMINATION OF HYDRAULIC PARAMETERS FROM BAIL-SLUG TESTS

There are many problems than may affect the quality of the hydraulic parameters obtained from slug tests on monitoring wells installed in fine-grained glacial tills. These problems have been previously summarized in Palmer and Paul (1987). Problems which may bias bail-slug test calculated hydraulic conductivity include:

1). choice of parameters L, r_w, r_c:
L = sand pack length
r_w = borehole radius
r_c = standpipe radius

2). bridging of bentonite seals

3). leaky joints above the sand-packed interval

4). formation of a low-permeability skin on the borehole wall

5). entrapped air in the sand pack above the slotted interval of the screen

6). the prescence of fractures in the formation

7). strain of the fine-grained formation material as a result of stress release around the borehole

8). partial penetration of the well

9). anistropy of the formation

10). fluctuation of the potentiometric surface during a bail test

11). boundary conditions

12). sand pack effects: sandpack dewatering, water level recovery within the sand pack

13). uncertainty in H_o

14). radius of influence of the test

Each of these points is discussed below.

Choice of Parameters L, r_w, and r_c

The analytical solutions evaluated in this study require proper application of the parameters of L, r_w, and r_c to those solutions if representative hydraulic conductivity values are to be obtained from slug tests. The choice of sand pack and screen size have been previously determined to have no effect on the hydraulic conductivity calculated from bail-slug tests performed on these wells. This was because the hydraulic conductivity of the screen and sand pack are approximately four orders of magnitude greater than that of formation. Therefore, any pressure change within the well should be rapidly propagated over the entire length of the screen and sand pack before much change will occur in the formation.

The proper choice of L and r_w should be the length of the sand pack and the radius of the borehole, respectively. The radius of the standpipe, r_c, is the appropriate value to use for flow within the standpipe. If hydraulic conductivity is determined from recovery data within the sand packed portion of the monitoring well, an effective radius must be used in place of r_c for the calculation of the hydraulic conductivity. This effective radius, r_e, is dependent on the volume of void space in the sand pack and can be written as:

$$r_e^2 = r_c^2(1-n) + nr_w^2 \qquad (21)$$

where n is the porosity of the sand pack (Sukop,1985).

Bridging of the Bentonite Seals

Anytime a vertical conduit such as a borehole is constructed in a low-permeability material, downward leakage is possible between the piezometer and bentonite used to seal the hole, the bentonite and the formation, or through the bentonite itself. Bridging of the bentonite pellets may occur between the piezometer and formation at heights above the sand pack. If bridging occurs, the vacant volume below the bridged area may add to the effective length of the sand pack. The result would be to produce hydraulic conductivity that is higher than the true formation hydraulic conductivity. The problem of bridging can be alleviated with the use of a bentonite slurry, cement grout, or a cement/grout mixture.

Leaky Joints

Another problem that may develop is leakage through joints connecting lengths of PVC standpipe. Joint leakage did occur in this study. The construction of well 2-1 involved a friction joint connecting two lengths of PVC near the surface. A Hvorslev plot (Fig. 38) shows a large increase in the relative head during a slug test started on September 4, 1986

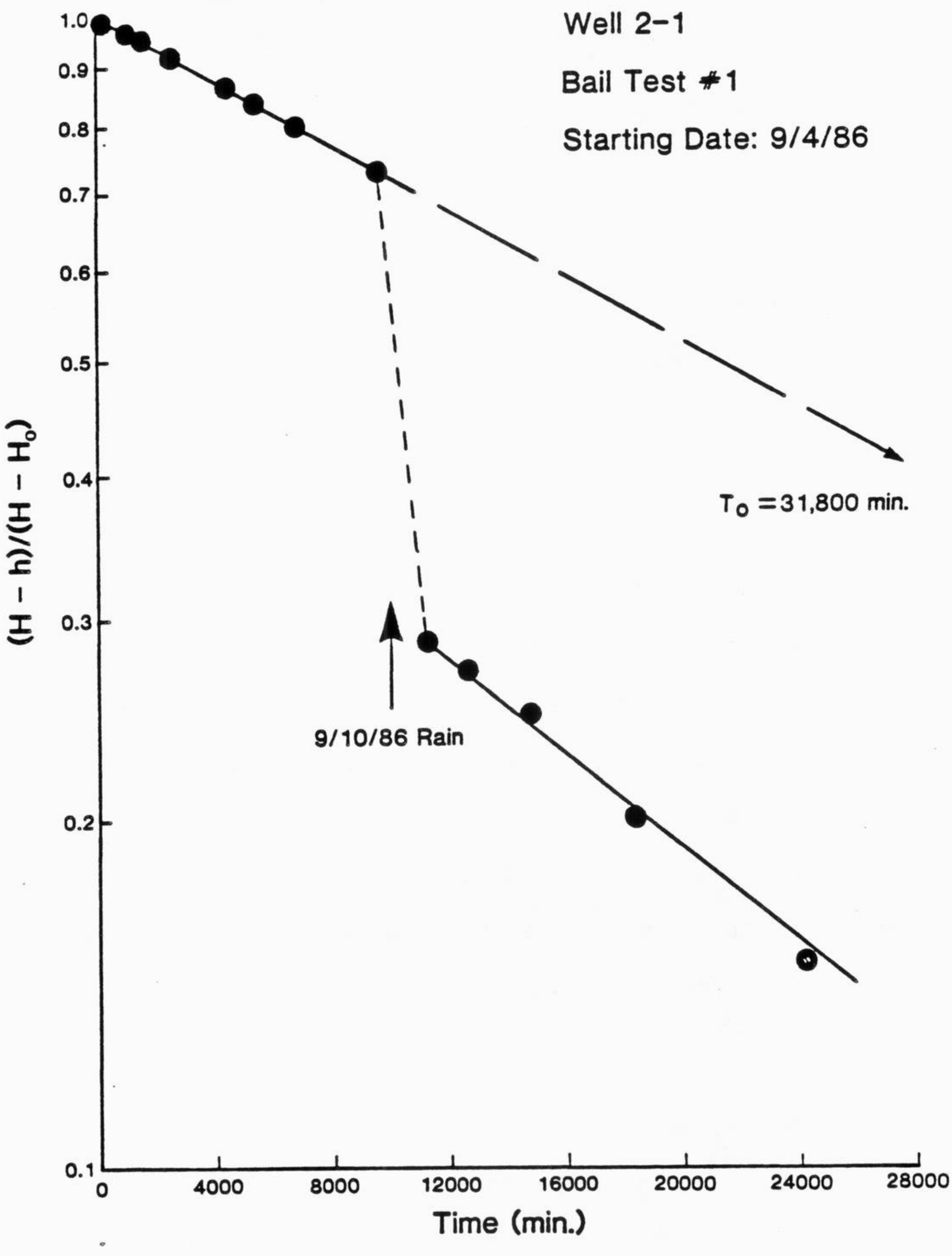

Figure 38. Hvorslev plot showing the effect of a rain fall event on the recovery response of well 2-1.

associated with a rainfall event on September 10, 1986. The shallow slope of the line is diagnostic of the true hydraulic conductivity. This problem did not occur in any of the other piezometers with the rainfall event, therefore, leakage through the near-surface joint is the suspected cause. The problem of leakage through the joint could have been alleviated by threading the coupling and grouting around the joint.

Formation of a Low-Permeability Skin

Bias may be contributed to the slug test interpretation by the piezometer installation procedure which may create a low permeability (smeared) skin on the wall of the borehole. Auger action may also smear in secondary fractures present in the till. The skin effect caused by the augering process is difficult to evaluate in the field. Smearing of the borehole was observed in this study. This smearing is the result of several factors: 1) looseness of joints connecting flights of auger causing a circular motion when rotated, 2) deflection of the auger flights off of subsurface boulders, and 3) retrieval of the auger itself. Because damage to the wellbore face cannot generally be avoided, development by surging is sometimes used to reduce the effects of the skin. In this study, however, this well development procedure was found to have minimal effect on the hydraulic conductivity and negative effects on the quality of the water samples because of the large increase in turbidity in the wells.

Numerical studies have shown that a low permeability skin of finite thickness can significantly affect the hydraulic conductivity obtained from slug tests (Faust and Mercer, 1984). Analytical solutions and type curves for the analysis of slug tests conducted on fully penetrating wells in confined aquifers with well skins has been presented by Sageev (1986). Unfortunately, the type curves presented are very similar and any curve fitting procedure may be prone to a large amount of uncertainty. However, it may be possible to derive a direct method for the calculation of the formation hydraulic conductivity, the thickness of the skin, and the product of the hydraulic conductivity and specific storage of the skin using the equations by Sageev (1986). This possibility is currently being investigated (Palmer, pers. com., 1987).

Entrapped Air above the Sand Pack

In fine-grained materials where a sand pack is used, air may become entrapped within the pack and above the screened portion of the piezometer when the bailed head recovers above the slotted interval. As the head in the well recovers above the screen, the head in the well above the sand pack will cause compression of the air remaining in the sand pack

(Keller and van der Kamp, 1987). Because 78% of atmospheric air is composed of nitrogen (e.g. Hem, 1970) and because nitrogen has a very low solubility in water, a large fraction of the air is expected to remain entrapped over the duration of the bail test. The effect of compressing the entrapped air during well recovery would be to decrease the rate of recovery. Accurate determination of the amount of entrapped air remaining in the pack could not be made for this study and the entrapped air error is assumed negligible. This appears reasonable in light of the observations by Keller et al. (1986) that only their smaller diameter piezometers (6 mm) and not their larger diameter (38 mm) piezometers exhibited significant effects of entrapped air. A methodology for the correction of slug test data for the effects of entrapped air is currently under development (Keller and van der Kamp, 1987).

The Presence of Fractures

The existence of fracture patterns in tills has been observed by various researchers (Grisak and Cherry, 1975; Keller et al., 1986) and has been used to account for the differences in the hydraulic conductivity between triaxial tests which measure the intergranular hyraulic conductivity, and slug tests which measure more the bulk properties of the formation. The slug test analysis, however, is theoretically based on a homogeneous porous medium and not a fractured medium. This is of concern since it has been suggested that conventional methods of analyzing slug test data may be unsuitable for analyzing data from piezometers installed in fractured rock (Schwartz, 1975). The point that must be addressed is whether or not the hydraulic parameters obtained from the slug test are representative of some equivalent porous medium and what bias there may be in the computed hydraulic parameters. Fortunately, this problem has been addressed by Barker and Black (1983) for an ideal set of equally spaced, horizontal fractures in a porous matrix.

The solution given by Barker and Black (1983) utilizes four dimensionless parameters: τ, α, β, and γ defined by:

$$\tau = Tt/r_c^2 \tag{22}$$

$$\alpha = Sr_w^2/r_c^2 \tag{23}$$

$$\beta = (2Nr_c/S)(S_sK/T)^{1/2} \tag{24}$$

$$\gamma = (d/r_c)(S_sT/K)^{1/2} \tag{25}$$

where:

- d = half the fissure separation (m)
- K = hydraulic conductivity of the rock matrix (m/s)
- N = number of fissures
- r_c = effective casing radius (m)
- r_w = radius of the borehole (m)
- S = fissure storage coefficient
- S_s = specific storage of the rock matrix (m^{-1})
- t = time (sec)
- T = total fissure transmissivity (m^2/s)

If α, β, and γ can be calculated, σ_τ, σ_β, and ($\sigma_\alpha/(1+\beta\gamma)$) can be obtained from Figure 39 (Figure 3 from Barker and Black, 1983), where:

σ_τ = ratio of equivalent homogeneous aquifer transmissivity to total fissure transmissivity

σ_α = ratio of equivalent homogeneous aquifer storage coefficient to fissure storage coefficient

$\sigma_\alpha/(1+\beta\gamma)$ = the factor by which the derived storage coefficient must be divided to obtain the total (fissure + matrix) storage coefficient

The parameters that were used in analysis of field site used in this study are given in Table 22. Transmissivity values of 10 and 100 times the matrix value were used. The fracture spacing (2d) is not known for the site and therefore values reported by Bradbury et al. (1985) and Keller et al. (1986) were used. The fracture width (B) was calculated from the cubic law for ground water flow in fractures (e.g. Freeze and Cherry, 1979) and the bulk hydraulic conductivity values:

$$B = [12bK\mu/(\rho gN)]^{1/3} \tag{26}$$

where:

ρ = density of the fluid (1000 kg/m^3)
g = acceleration of gravity (9.80 m/s^2)
μ = dynamic viscosity of water (0.00124 kg/(m·s))
K = the hydraulic conductivity of the formation (m/s)
N = number of fractures over the length of the sand pack
b = thickness of the formation (m)

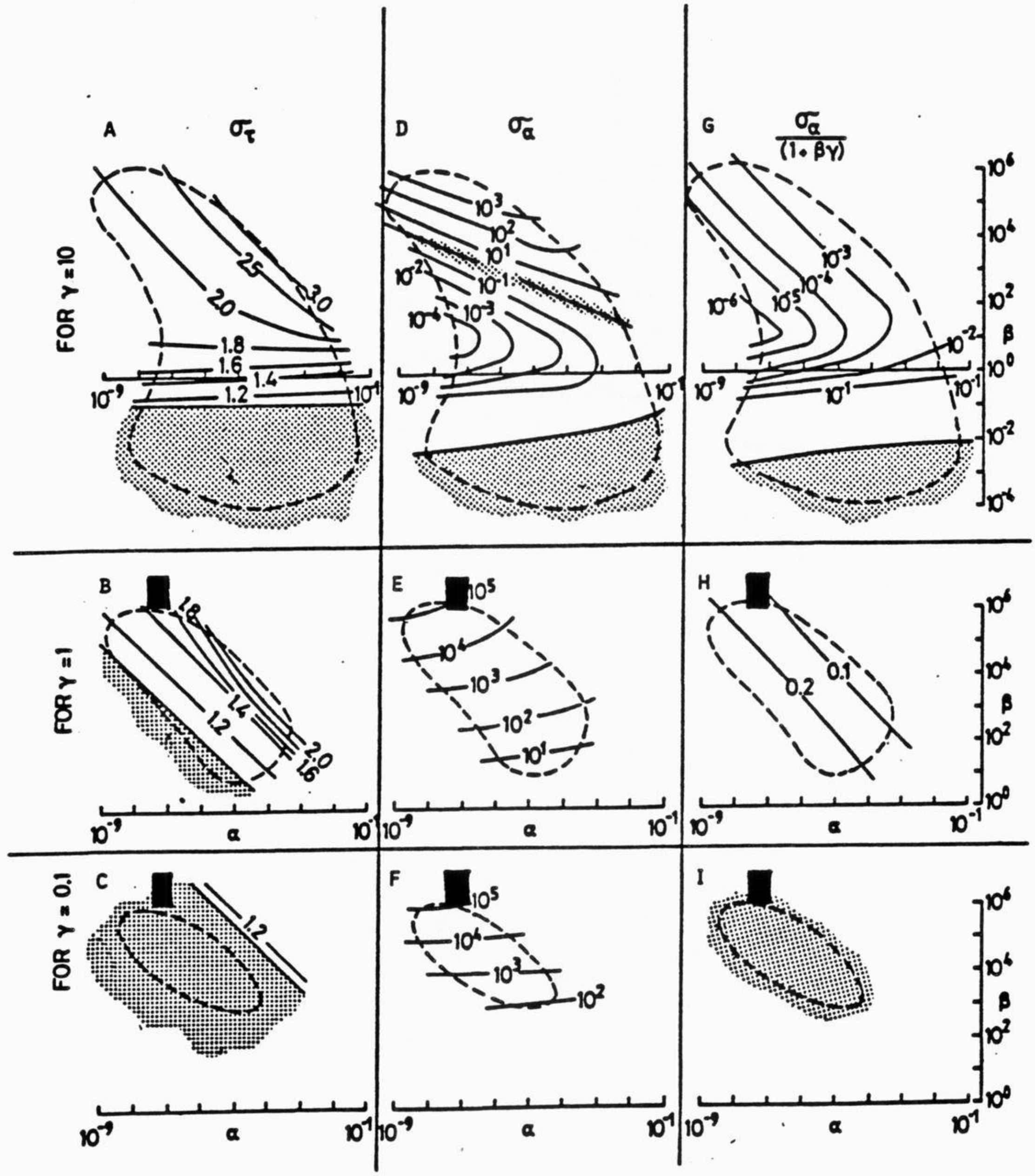

Figure 39. Scaling factors for porous media model versus ideal parallel fracture model. Stipling indicates areas where scaling factor is in the range of 0.9 to 1.1. ▬ indicates areas with parameters listed in Table 22. (Modified from Barker and Black, 1983).

Table 22 . Parameters used to estimate scaling factors for porous media versus parallel fracture models.

PARAMETER	VALUE	SOURCE	
K	3×10^{-10} m/s	traixial cell test (this study)	
S_s	0.002 /m	consolidation test (this study)	
T	7.32×10^{-9} m^2/s 7.32×10^{-8}	slug test (this study)	
r_w	0.0762 m		
r_c	0.0254 m		
d	0.0275 m 0.005 m	Bradbury et al. (1985) Keller et al. (1986)	
N	45	Using d from Bradbury et al. (1985)	
	244	Using d from Kellar et al. (1986)	
B	1.35×10^{-5}	$T=9 \times 10^{-9}$;	n=45 *
	7.71×10^{-6}		n=244
	2.92×10^{-5}	$T=9 \times 10^{-8}$;	n=45
	1.66×10^{-5}		n=244
S	2.88×10^{-9}	$T=9 \times 10^{-9}$;	n=45 **
	8.88×10^{-9}		n=244
	6.21×10^{-9}	$T=9 \times 10^{-9}$;	n=45
	1.91×10^{-8}		n=244

* Computed from cubic law and T (Eq. 12).
** Computed from Eq. 13.

LEGEND:

K = rock matrix hydraulic conductivity (m/s)
S_s = rock matrix specific storage (m^{-1})
T = total fissure transmissivity (m^2/s)
r_w = borehole radius (m)
r_c = standpipe radius (m)
N = number of fissures
B = fracture width (m)
S = fissure storage coefficient

The storage coefficient for the fractures was calculated from:

$$S = N\rho g b C_w \tag{27}$$

where C_w is the compressibility of water (4.8×10^{-10} $m \cdot s^2/kg$) and all other parameters are as defined above.

The range of values for α, β, and γ are given in Table 22 and those limits are marked on Fig. 39. From this figure, the values of σ_τ, σ_α, and $\sigma_\alpha/(1+\beta\gamma)$ are determined to be 1.0-1.5, 10^5-10^6, and 1.0-0.1, respectively. This means that the use of porous media equations to the slug test data obtained from the fractured till should yield results close to the actual value for the formation. The computed value of hydraulic conductivity will be at most a factor 1.5 greater than the actual formation hydraulic conductivity. The large values of σ_α indicate that the value of the storage coefficient does not represent the storage coefficient for the fractures. The values of $\sigma_\alpha/(1+\beta\gamma)$ (0.1 to 1.0) indicate that the results of the slug test should represent the specific storage of the till matrix or may underestimate those values by a factor of about 0.1.

Effect of Borehole Stress Release

When an open borehole is augered into till that has been over-consolidated, some stress release will occur around the borehole. This stress release will cause the till to expand and change its hydraulic properties. These effects may be permanent or transient. The "stress adjustment" time lag as defined by Hvorslev (1951) may be much greater than hydrostatic time lag in fine-grained materials (Hvorslev, 1951). However, no attempt was made to quantify the effect of stress adjustment time lag on the bail-slug test calculated hydraulic conductivities in this study.

Partial Penetration of the Well

Most monitoring wells will be screened over only a portion of the formation and not over the entire thickness. The Hvorslev (1951) and Bouwer and Rice (1976) solutions do account for this partial penetration, but they assume the specific storage of the formation is zero. The method proposed by Cooper, et al. (1967) does include the effect of a non-zero specific storage, but it assumes that the well is fully penetrating. The lower values of hydraulic conductivity obtained with the Hvorslev (1951) method compared to those obtained from by Cooper, et al. (1967) have been attributed to the inclusion of the vertical flow components due to partial penetration of the well. However, the Cooper, et al. method includes effects of a non-zero specific storage which the Hvorslev solution does not and it is not possible to differentiate the effects of partial penetration from the effects of the storage coefficient.

The empirical equations presented by Bouwer and Rice (1976) can be used to estimate the effects of partial penetration when the specific storage is equal to zero. For the monitoring wells used in this study L/r_w equals 32. A fully penetrating well screened at the bottom of a 200 foot thick aquifer should produce a hyraulic conductivity that is 0.4 times the hydraulic conductivity of a well fully screened over an eight foot thick aquifer. Thinner aquifers or other locations of the screen within the aquifer will produce factors closer to unity. It is not known whether the same factors would apply to formations with non-zero specific storage values.

A more recent method for the calculation of hydraulic parameters from slug tests have been presented by Nguyen and Pinder (1984). Their method is based on a three-dimensional, axisymmetric representation of the flow field and will accomodate partially penetrating wells and a non-zero specific storage. The specific storage is determined by plotting $\log(H/H_o)$ versus $\mathrm{Log}(t)$. The slope of this line (C_1) is determined and the specific storage is calculated from:

$$S_s = (r_c^2 C_1)/(r_w^2 bL) \tag{28}$$

where:
S_s = specific storage (m^{-1})
r_c = standpipe radius (m)
C_1 = slope of line (m/log cycle time)
r_w = borehole radius (m)
b = aquifer thickness (m)
L = screen length (m)

The hydraulic conductivity, K, is found by plotting ln(dH/dt) versus (1/t), determining the slope of the curve (C_2) and solving for K:

$$K = r_c^2 C_1/(4C_2L) \tag{29}$$

where: C_2 = slope of line (sec)
L = screen intake (m)

The derivative, dH/dt, can be approximated from the data by finite difference techniques. If sufficient data are available so that accurate estimations of the derivatives can be made, this method should provide the best estimates of hydraulic parameters from slug tests.

Anisotropy of the Formation

The only solution than accounts for anistropy in the aquifer is the Hvorslev (1951) equation. Use of the equation indicates that very large K_h/K_v ratios are needed to produce increases in computed hydraulic conductivity by factors of 2 or 3 ($K_h/K_v = 10^3$ and 10^6, respectively). Studies of clays (Olsen and Daniel, 1981; Chan and Kenny, 1973; Tavenas et al., 1983) and glacial tills (Prudic, 1982) indicate that anistropy ratios this great are not likely to be encountered. Therefore, anisotropy is not likely to have a significant effect in clay tills.

Fluctuations in the Potentiometric Surface

Methods for taking into account fluctuating regional potentiometric surfaces during a slug test are presented by Hvorslev (1951). Two cases are presented: 1) linearly varying potentiometric surface, and 2) sinsoidal varying potentiometric surface. For the case where the head is changing linearly with time, the elevation to which the head in the well must recover, z, can be written as

$$z = H_o + at \tag{30}$$

where H_o is the initial perturbation of the the head in the well, t is time, and a is the rate of change in the potentiometric surface with respect to time. The second term in eq. 30 can be neglected if

$$at/H_o << 1. \tag{31}$$

otherwise corrections such as those described by Hvorslev (1951) must be applied.

Boundary Conditions

The analytical solutions derived for the analysis of slug tests have been based on the idea that the test would be conducted in an aquifer. This concept has resulted in Cooper, et al. (1967) and Nguyen and Pinder (1984) choosing no-flow boundary conditions above and below the formation being tested. Similarly, Bouwer and Rice (1976) have chosen their bottom boundary condition to be a no-flow boundary. A much more likely scenario in the tesing of low-permeability tills is that the underlying or overlying material is a more highly permeable formation (e.g. sand and gravel). Under these circumstances a constant head boundary is more appropriate than a no-flow boundary. What effect this will have on the response of the monitoring well will depend on whether or not the radius of influence of the slug test reaches the boundary. Monitoring wells installed in the middle of thick clays are not likely to be affected by the boundary, regardless of its type. Monitoring wells, however are often installed near boundaries and in those circumstances, the choice of boundary condition may have some effect.

The potential effect of the different boundary conditions can be considered using the Bouwer and Rice (1976) equations. The bottom boundary in their analogue model is a no-flow boundary and their top boundary is a constant head. If both boundaries were no-flow, a well installed at the very top of the formation should produce the same result as one installed at the very bottom. The Bouwer and Rice solutions, however, produce different results.

Assuming a screen length of 8 feet, a borehole radius of 0.25 feet, and an aquifer thickness of 25 feet, the computed hydraulic conductivity for the well near the no-flow boundary is 1.5 times greater that the computed hydraulic conductivity for the well near the water table. This factor will increase with increasing formation thickness and decreasing screen length but will generally not exceed 2. Formation anisotropy will theoretically decrease these factors. It is not clear at this time what effect a non-zero specific storage value will have on these results.

Sand Pack Effects

There were two major sand pack effects observed in this study: 1) the sand pack dewatering phenonmenon and 2) the slow wetting of initially dry sand packs.

The sand pack dewatering phenomenon observed in this study has also been observed by Sukop (1986) This phenomenon masks the early response of the formation to the slug test and complicates data analysis. It can also introduce air into the sand pack which can affect the chemical quality of samples obtained from the wells. Therefore, sand pack dewatering should be avoided whenever possible.

In many cases, the recovery in a newly installed monitoring well is used to determine the hydraulic conductivity of the formation. In this study, the time that it took the monitoring wells to begin the recovery was observed to be dependent on the amount of moisture in the sand when it was installed. If the sand pack was installed very dry, it took as long as 2 weeks before water was detected in the bottom of the monitoring wells. If the sand was "damp" when installed, then only a few days were needed until water was detected in the bottom of the piezometer.

When piezometers are recovering through the sandpack, the effective radius of the well is larger than the radius of the standpipe. This effective radius can be calculated from consideration of the volume of pore space in the sandpack and the volume of water in the monitoring well (Sukop, 1986). Results from this study indicate that the initial recovery data can be used as long as 1) the correct effective radius is used, 2) the test is considered to start when water appears in the bottom of the monitoring well.

Uncertainty in H_o

Because of the slow rates of recovery of the water levels in monitoring wells installed in fine-grained glacial tills, there is often uncertainty concerning the value of H_o during the initial recovery. The hydraulic conductivity from the Hvorslev (1951) method can be written as:

$$K_T = C \ln(H/H_o{}^T) \qquad (32)$$

where:

$$C = r_c^2 \ln(mL/r_w)/(2Lt) \qquad (33)$$

and H_o^T represents the true value of H_o. If some other value of H_o is used K_T can be written as:

$$K_T = C \ln[(H/H_o)(H_o/H_o{}^T)] \qquad (34)$$

or

$$K_T = C \ln(H/H_o) + C \ln(H_o/H_o{}^T) \qquad (35)$$

or

$$K_T = K_g\{1 + [\ln(H_o/H_o{}^T)/\ln(H/H_o)]\} \qquad (36)$$

where K_g is the hydraulic conductivity based on the incorrect H_o value. If K is calculated from the point where H/H_o is 0.37, the eq. 35 reduces to:

$$K_T/K_g = [1-\ln(H_o/H_o^T)] \tag{37}$$

Therefore, even if H_o/H_o^T is equal to 0.5, the true value of the hydraulic conductivity will only be 1.7 time greater that the value estimated with the smaller value of H_o. For the tests associated with the present study, the maximum uncertainty in H_o is about 2 feet and the best guess of H_o is 18 feet so the uncertainty in H_o contributes only about a 10% uncertainty in the hydraulic conductivity.

Radius of Influence of the Test

The radius of influence of the test will depend on the how it is defined and the method of analysis. For the Hvorslev (1951) solution the effective radius of the test is equal to the length of the screen, L (in anisotropic media it is equal to mL). The radius of influence for the Bouwer and Rice (1976) method is based on the analogue simulations of Bouwer and Rice and is equal to their R_e value (see equations 12 and 13). For most practical situations $R_e < L$. The effective radius for the Cooper et al. (1969) solution has not been computed. It is expected, however that it will be less than R_e. This is because of the water that is released or taken into storage during the slug test will reduce the change in head at some given distance from the monitoring well. Thus the effective radius for any solution that has a non-zero specific storage will be expected to be less than that obtained from solutions based on steady-state ground water flow equations.

SUMMARY OF CONCLUSIONS

The results of this study are summarized as: 1) the effect of piezometer construction, installation, and development techniques on water sample turbidity, and 2) the effect of piezometer installation and development techniques on calculated hydraulic conductivity.

Turbidity Results

Major conclusions concerning the effect of monitoring well construction, installation, and development practices on the turbidity of water samples obtained from monitoring wells installed in fine-grained glacial till can be summarized as follows:

1. The turbidity of water samples obtained from wells that were installed after water had begun filling the bottom of the borehole was 50 to 200 times greater than in samples from wells that were installed in essentially dry boreholes.

2. Monitoring wells that were surged produced water samples with 3 to 100 times greater turbidity than wells that were only bailed.

3. For the given sand pack material, there are no inherent differences in water turbidity obtained from monitoring wells finished with factory slot, factory slot with Mirafi™ wrap or continuous slot screens.

4. The turbidity of water samples obtained from surged wells did not show a significant decrease with the second sampling, but the turbidity of samples obtained from wells that were bailed-only decreased by a factor of 3.

5. Commonly available well screens and sand packs are not capable of filtering out clay-sized particles in fine-grained glacial tills. The optimal well design will require a silt-sized sand pack and a very fine-meshed screen (< 0.05 mm).

Hydraulic Conductivity Results

Major conclusions concerning the effect of monitoring well construction, and development practices on hydraulic conductivity calculated from bail-slug test on monitoring wells installed installed in fine-grained glacial till can be summarized as follows:

1. The hydraulic conductivity of the screen and sand pack material used in this study have been determined to be approximately four orders of magnitude greater than the hydraulic conductivities determined from bail-slug tests in the field. There fore, the screen and sand pack materials used in this study have no effect on the bail-slug test calculated hydraulic conductivity.

2. Near-surface damage of the augered holes was observed during piezometer installation. Turbidity results suggest that well development by surging is effective in removing formation material on the borehole wall and pulling the suspended material into the well annulus. Development by surging, therefore, should be effective in reducing any auger-induced skin effects. However, the hydraulic conductivity results of this study indicate that development by surging has no significant effect of reducing skin effects and increasing the hydraulic conductivity calculated from field tests over development by only bailing.

3. Bail test recovery data plotted as relative head versus time should yield a straight line on a semi-log plot. Deviations in the early portion of the total water level recovery versus time curve may suggest effects of sand pack dewatering or unsaturated recovery within the sand pack itself. Evaluation of a non-linear plot by any of the currently used bail tests solutions will produce hydraulic conductivities that are not representative of formation material.

4. Formation hydraulic conductivities as calculated by the slug-bail test solution of Bouwer and Rice (1976) are consistently 0.75 times those calculated by the method of Hvorslev (1951). In contrast, formation hydraulic conductivities as calculated by the method of Cooper, et al. (1967) - Papadopulos, et al. (1973) are approximately three times those calculated by the method of Hvorslev (1951).

5. The range of hydraulic conductivities as determined from bail-slug tests indicates a bimodal distribution of values for the gray silty-clay till of the Oak Creek Formation. Evidence for lenses of sand and/or gravel was not observed in turnings from the drilling process or in Shelby tube soil samples collected in the field. However, a fracture was observed in hand sample. This suggests that fracture flow may occur in some of the wells, resulting in higher measured hydraulic conductivities.

6. Hydraulic conductivity determined from time-consolidation data has been shown to give results which are within an order of magnitude of triaxial permeabilities. However, due to unavoidable operator error, instrument error, and error intrinsic in the graphical interpretation of time consolidation tests, hydraulic conductivity as determined in the triaxial cell produces conductivities which are more truly representative of the intergranular hydraulic conductivity of the formation.

7. The triaxial cell hydraulic conductivity results of this study agree within a factor of two of the lower formation hydraulic conductivities as calculated by the methods of Hvorslev (1951) and Bouwer and Rice (1976). However, intergranular triaxial cell hydraulic conductivities may not be representative of the bulk hydraulic conductivity of the formation as shown by the bimodality of slug-bail test results in this study. If large (order of magnitude) discrepancies occur between laboratory triaxial cell and slug-bail test hydraulic conductivities of a fine-grained formation material ($K < 10^{-6}$ cm/sec), primary structures such as sand and gravel stringers and secondary structures such as fractures should be thoroughly evaluated when considering the material for waste containment.

RECOMMENDATIONS

The overall recommendations regarding monitoring well construction, installation, and development techniques necessary to obtain representative formation hydraulic conductivities from bail tests and sediment free water samples based on the results of this study are as follows:

1. A non-surged factory slot piezometer set in an essentially dry borehole and packed with TDS2150 sand is the optimal monitoring well design which will result in bail test hydraulic conductivities which are representative of the formation conductivity and produce essentially turbidity-free water.

2. Surging has little influence on calculated hydraulic conductivity measured from bail tests, but increases water sample turbidity. The effect is an overall increase in the cost of a sampling program. Therefore, surging is not recommended as a development technique in fine-grained materials.

3. Turbidity results from this study have shown no differences between the various screen-filter combinations used and their ablility to keep suspended material out of the well annulus. Therefore, factory slot screen alone is recommended because it is the cheapest method of obtaining representative water samples and formation hydraulic conductivities.

4. A monitoring well which is set in an essentially "dry" borehole is best for reducing water sample turbidity. Setting the piezometer in a borehole partially filled with water allows water carrying suspended material into the well annulus prior to setting the sand pack. The result is sediment build-up in the bottom the piezometer. This sediment was observed to be brought into suspension especially when the bailer was allowed to strike the bottom. The use of a bailer to remove such sediment is not effective. Some surging of the water present in the monitoring well may be necessary to suspend the bottom sediment so that it may be removed from the well annulus. The use of a sampling pump to remove bottom sediment may

also be an effective means of removing sediment present in the bottom of piezometers.

5. Hydraulic conductivity as calculated by the methods of Hvorslev (1951) and Bouwer and Rice (1976) have been shown to be within a factor of two of the intergranular triaxial cell permeabilities. Both methods are recommended for the determination of aquitard hydraulic conductivity. If a monitoring well is installed in a homogeneous medium of infinite vertical extent, the analytical solution of Hvorslev (1951) should be used. If the monitoring well is partially penetrating, the analytical solution of Bouwer and Rice should be used. The method of Cooper, et al. (1967) - Papadopulos, et al. (1973) is not recommended for evaluating bail test hydraulic conductivities because the assumptions and boundary conditions for the aquifer solution are not representative of a well installed in an aquitard. This method produces hydraulic conductivities which are a factor of 3 times greater than those calculated by the methods of Hvorslev (1951) and Bouwer and Rice (1976).

6. Downward leakage due to a near-surface friction fitting connecting two lengths of PVC was observed in this study. To avoid this problem and any biases that may result from such leakage, it is recommended that joints connecting lengths of PVC standpipe be threaded and grouted. Bentonite pellets as a sand pack sealant in fine-grained materials should not be used to avoid problems that may develop because of bridging and wetting-up of the pellets.

FUTURE WORK

During the course of this study several questions have not been addressed which indicate the need for further study. The first question is the physical analysis of the sand pack dewatering phenomenon. A model has been proposed (Sukop,1986) to account for volume changes which occur when a volume of water is instantaneously removed from a sand packed well screened across the water table. One of the criteria which is necessary and present in the conditions of this study is that the hydraulic conductivity of the sand pack be much greater than the hydraulic conductivity of the formation. The dewatering effect could be monitored with a fiberglass tape and metal sounder, however, a calibrated pressure transducer may produce results with better resolution.

The presence of a sand pack itself has posed many questions relating to the acquisition of representative water samples. Clean quartz sand packs are generally desired as a packing material as to not allow cation exchange and thus, potential bias of the water sample analysis and interpretation. Such quartz sand packs, however, are capable of retarding organic contaminants and metal ions (Palmer, et al., 1987). If a well is set with a dry sand pack in a formation with expected contamination, adsorption of organic constituents and /or metal on the packing material can be expected. The chemical analysis of early water samples taken from this well may be biased toward the low side or may not show contamination at all. Subsequent pore volumes removed may show increases in concentration of chemical constituents. This increase will occur until the pack comes into equilibrium with the chemical concentration of the formation.

This problem involves the quantitative analysis of the number of pore volumes of water which have to be removed to chemically equilibrate the sand pack with the formation. This is not expected to be a problem where the hydraulic conductivity of the formation is greater than 1 x 10^{-6} cm/s. However, when a well requires week or even months to recover, the pore volume analysis becomes very important.

Another problem developed by the sand pack is air entrapment in the sand pack. Physically, air entrapment has been shown to have negligible effects on the calculated hydraulic conductivity of piezometers with diameters greater than 38 mm (Keller, et al., 1986). However, air remaining in the sand pack has the possibility of volatizing organic chemicals and causing the precipitation of dissolved metal constituents. A chemical analysis of a water sample taken from such a well may result in lower observed chemical concentrations than actually representative of the formation. Dewatering a sand pack as such may cause the volatization of organic constituents or even cause temporary precipitation of metal constituents. If this occurs, the purging of the well below the screen may continually vacate adsorptive locations and result in chemical analyses which are biased toward the low side. In this case, the methods of obtaining water samples in low permeability environments should be closely evaluated.

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APPENDIX I

WELL SCREEN FLOW RATES AND WELL SCREEN DIMENSIONS USED TO DETERMINE SCREEN HYDRAULIC CONDUCTIVITIES BY A CONSTANT HEAD TEST

Screen Hydraulic Conductivity Data (see Fig. 1)

Temperature of Water to Calibrate (Fill) 20 gallon barrel = 18.6°C

Screen Type: Factory Slot
Screen Length (L): 1.43 meters
Head Difference (Δh): 0.54 meters
Screen Inside Radius (r_i): 0.024 meters
Screen Thickness (Δr): 0.006 meters

	Trial No.	5 gallons	10 gallons	15 gallons	20 gallons
Time *	1 *	61	120	175	229
(in *	2 *	57	115	173	230
sec.)*	3 *	58	117	173	231

20 gallon ave. time = 230 sec.

Screen Type: Factory Slot with Mirafi™ Filter Wrap
Screen Length (L): 1.43 meters
Head Difference (Δh): 0.54 meters
Screen Inside Radius (r_i): 0.024 meters
Screen Thickness (Δr): 0.006 meters

	Trial No.	5 gallons	10 gallons	15 gallons	20 gallons
Time *	1 *	55	110	165	221
(in *	2 *	56	110	164	219
sec.)*	3 *	55	111	163	220

20 gallon ave. time = 220 sec.

Screen Type: Continuous Slot
Screen Length (L): 0.61 meters
Head Difference (Δh): 0.54 meters
Screen Inside Radius (r_i): 0.027 meters
Screen Thickness (Δr): 0.0025 meters

	Trial No.	5 gallons	10 gallons	15 gallons	20 gallons
Time *	1 *	56	112	168	223
(in *	2 *	53	104	156	207
sec.)*	3 *	55	108	164	217

20 gallon ave. time = 216 sec.

Screen Type: Porous Stone
Screen Length (L): 0.61 meters
Head Difference (Δh): 0.67 meters
Screen Inside Radius (r_i): 0.0125 meters
Screen Thickness (Δr): 0.007 meters

	Trial No.	5 gallons	10 gallons	15 gallons	20 gallons
Time	1	62	125	197	292
(in	2	75	150	225	298
sec.)	3	72	143	214	286

20 gallon ave. time = 292 sec.

APPENDIX II

WELL CONSTRUCTION DATA FOR WELL INSTALLATION AT SITE 1 AND SITE 2

Screen-type abbreviations:

FS = Factory Slot
FSWMW = Factory Slot with Mirafi Wrap
CS = Continuous Slot
PS = Porous Stone

Field Site 1:

Well No.: 1-1
Date Installed: 4/4/86
Screen-type: FS
Borehole Dimensions
Borehole Radius (m): 0.051
Total Depth (m): 5.41
Piezometer Dimensions
Standpipe Radius (m): 0.024
Sandpack Length (m): 2.36
Screen Length (m): 1.43
Total Length (m): 7.65
2.46 meters above ground

Well No.: 1-2
Date Installed: 4/8/86
Screen-type: FS
Borehole Dimensions
Borehole Radius (m): 0.076
Total Depth (m): 5.33
Piezometer Dimensions
Standpipe Radius (m): 0.024
Sandpack Length (m): 2.13
Screen Length (m): 1.43
Total Length (m): 7.65
2.39 meters above ground

Well No.: 1-3
Date Installed: 4/8/86
Screen-type: FSWMW
Borehole Dimensions
Borehole Radius (m): 0.076
Total Depth (m): 5.44
Piezometer Dimensions
Standpipe Radius (m): 0.024
Sandpack Length (m): 1.98
Screen Length (m): 1.39
Total Length (m): 7.62
2.34 meters above ground

Well No.: 1-4
Date Installed: 4/12/86
Screen-type: FSWMW

Borehole Dimensions

Borehole Radius (m):	0.076
Total Depth (m):	5.33

Piezometer Dimensions

Standpipe Radius (m):	0.024
Sandpack Length (m):	2.67
Screen Length (m):	1.43
Total Length (m):	7.63

2.44 meters above ground

Well No.: 1-5
Date Installed: 4/18/86
Screen-type: CS

Borehole Dimensions

Borehole Radius (m):	0.076
Total Depth (m):	5.33

Piezometer Dimensions

Standpipe Radius (m):	0.024
Sandpack Length (m):	1.98
Screen Length (m):	1.45
Total Length (m):	7.65

2.45 meters above ground

Well No.: 1-6
Date Installed: 4/18/86
Screen-type: CS

Borehole Dimensions

Borehole Radius (m):	0.076
Total Depth (m):	5.33

Piezometer Dimensions

Standpipe Radius (m):	0.024
Sandpack Length (m):	1.98
Screen Length (m):	1.45
Total Length (m):	7.65

2.40 meters above ground

<u>Field Site 2:</u>

Well No.: 2-1
Date Installed: 6/5/86
Screen-type: FS
<u>Borehole Dimensions</u>
  Borehole Radius (m): 0.076
  Total Depth (m): 6.73
<u>Piezometer Dimensions</u>
  Standpipe Radius (m): 0.024
  Sandpack Length (m): 2.32
  Screen Length (m): 1.42
  Total Length (m): 7.65
  1.01 meters above ground

Well No.: 2-2
Date Installed: 6/5/86
Screen-type: FS
<u>Borehole Dimensions</u>
  Borehole Radius (m): 0.076
  Total Depth (m): 6.74
<u>Piezometer Dimensions</u>
  Standpipe Radius (m): 0.024
  Sandpack Length (m): 2.21
  Screen Length (m): 1.42
  Total Length (m): 7.59
  0.90 meters above ground

Well No.: 2-3
Date Installed: 6/9/86
Screen-type: FSWMW
<u>Borehole Dimensions</u>
  Borehole Radius (m): 0.076
  Total Depth (m): 6.46
<u>Piezometer Dimensions</u>
  Standpipe Radius (m): 0.024
  Sandpack Length (m): 2.41
  Screen Length (m): 1.43
  Total Length (m): 7.39
  0.81 meters above ground

Well No.: 2-4
Date Installed: 6/9/86
Screen-type: FSWMW
<u>Borehole Dimensions</u>
Borehole Radius (m): 0.076
Total Depth (m): 6.46
<u>Piezometer Dimensions</u>
Standpipe Radius (m): 0.024
Sandpack Length (m): 2.41
Screen Length (m): 1.46
Total Length (m): 7.42
0.81 meters above ground

Well No.: 2-5
Date Installed: 6/9/86
Screen-type: CS
<u>Borehole Dimensions</u>
Borehole Radius (m): 0.076
Total Depth (m): 6.25
<u>Piezometer Dimensions</u>
Standpipe Radius (m): 0.024
Sandpack Length (m): 2.68
Screen Length (m): 1.46
Total Length (m): 7.24
0.82 meters above ground

Well No.: 2-6
Date Installed: 6/9/86
Screen-type: CS
<u>Borehole Dimensions</u>
Borehole Radius (m): 0.076
Total Depth (m): 6.37
<u>Piezometer Dimensions</u>
Standpipe Radius (m): 0.024
Sandpack Length (m): 2.62
Screen Length (m): 1.45
Total Length (m): 7.33
0.81 meters above ground

Well No.: 2-7
Date Installed: 6/12/86
Screen-type: FS
<u>Borehole Dimensions</u>
Borehole Radius (m): 0.076
Total Depth (m): 6.58
<u>Piezometer Dimensions</u>
Standpipe Radius (m): 0.024
Sandpack Length (m): 2.62
Screen Length (m): 1.40
Total Length (m): 7.50
0.80 meters above ground

Well No.: 2-8
Date Installed: 6/12/86
Screen-type: FS
Borehole Dimensions
    Borehole Radius (m): 0.076
    Total Depth (m): 6.52
Piezometer Dimensions
    Standpipe Radius (m): 0.024
    Sandpack Length (m): 2.47
    Screen Length (m): 1.41
    Total Length (m): 7.53
    0.82 meters above ground

Well No.: 2-9
Date Installed: 6/12/86
Screen-type: FSWMW
Borehole Dimensions
    Borehole Radius (m): 0.076
    Total Depth (m): 6.49
Piezometer Dimensions
    Standpipe Radius (m): 0.024
    Sandpack Length (m): 2.47
    Screen Length (m): 1.42
    Total Length (m): 7.43
    0.81 meters above ground

Well No.: 2-10
Date Installed: 6/12/86
Screen-type: FSWMW
Borehole Dimensions
    Borehole Radius (m): 0.076
    Total Depth (m): 6.58
Piezometer Dimensions
    Standpipe Radius (m): 0.024
    Sandpack Length (m): 2.29
    Screen Length (m): 1.42
    Total Length (m): 7.15
    0.76 meters above ground

Well No.: 2-11
Date Installed: 6/18/86
Screen-type: CS
Borehole Dimensions
    Borehole Radius (m): 0.076
    Total Depth (m): 6.73
Piezometer Dimensions
    Standpipe Radius (m): 0.024
    Sandpack Length (m): 2.56
    Screen Length (m): 1.44
    Total Length (m): 7.42
    0.78 meters above ground

Well No.: 2-12
Date Installed: 6/18/86
Screen-type: CS

Borehole Dimensions

Borehole Radius (m): 0.076
Total Depth (m): 6.74

Piezometer Dimensions

Standpipe Radius (m): 0.024
Sandpack Length (m): 2.59
Screen Length (m): 1.45
Total Length (m): 7.42
0.81 meters above ground

Well No.: 2-13
Date Installed: 7/2/86
Screen-type: PS

Borehole Dimensions

Borehole Radius (m): 0.051
Total Depth (m): 6.64

Piezometer Dimensions

Standpipe Radius (m): 0.0095
Sandpack Length (m): 1.83
Screen Length (m): 0.61
Total Length (m): 7.41
0.80 meters above ground

Well No.: 2-14
Date Installed: 7/2/86
Screen-type: PS

Borehole Dimensions

Borehole Radius (m): 0.051
Total Depth (m): 6.71

Piezometer Dimensions

Standpipe Radius (m): 0.0095
Sandpack Length (m): 1.98
Screen Length (m): 0.61
Total Length (m): 7.36
0.83 meters above ground

APPENDIX III

SLUG AND BAIL TEST DATA FROM SITES 1 AND 2 USED TO DETERMINE FIELD HYDRAULIC CONDUCTIVITY

Bail Test: #1
Well #: 1-1
Date: 5/1/86
Static Head, H (m): 3.31
Bailed Head, Ho (m): 5.76
Starting Time: 10:26 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.76	0	1.00
5.68	12	0.96
5.63	35	0.95
5.57	62	0.92
5.47	96	0.88
5.39	130	0.85
5.26	186	0.79
5.00	297	0.69
3.51	1365	0.08
3.46	1460	0.06
3.19	2671	-0.05

Bail Test: #1
Well #: 1-2
Date: 5/1/86
Static Head, H (m): 4.72
Bailed Head, Ho (m): 5.11
Starting Time: 2:56 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.57	-256	
5.43	-253	
5.35	-242	
5.21	-233	
5.17	-207	
5.13	-157	
5.12	-120	
5.11	-82	
5.11	0	1.00
5.06	1185	0.89
5.03	2408	0.81
5.00	4114	0.71
4.97	5308	0.63
4.88	8770	0.41
4.81	11734	0.22

Bail Test: #1
Well #: 1-3
Date: 5/1/86
Static Head, H (m): 4.15
Bailed Head, Ho (m): 4.73
Starting Time: 10:48 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.84	-1425	
5.76	-1417	
5.69	-1391	
5.59	-1356	
5.51	-1322	
5.43	-1286	
5.29	-1213	
5.20	-1156	
4.73	0	1.00
4.53	1224	0.66
4.34	2929	0.33
4.27	4124	0.22
4.25	7586	0.18
4.28	9108	0.23

Bail Test: #1
Well #: 1-4
Date: 5/1/86
Static Head, H (m): 4.25
Bailed Head, Ho (m): 4.86
Starting Time: 3:27 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.64	-243	
5.45	-233	
5.14	-200	
4.99	-165	
4.93	-129	
4.88	-55	
4.86	0	1.00
4.72	1162	0.77
4.63	2380	0.62
4.44	4086	0.32
4.28	5280	0.04

Bail Test: #2
Well #: 1-1
Date: 6/25/86
Static Head, H (m): 3.35
Bailed Head, Ho (m): 5.56
Starting Time: 10:11 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.56	0	1.00
5.55	2	1.00
5.54	5	0.99
5.54	10	0.99
5.52	25	0.98
5.49	44	0.97
5.45	74	0.95
5.41	95	0.93
5.38	120	0.92
5.35	151	0.90
5.29	186	0.87
5.23	227	0.85
5.15	282	0.81
5.07	345	0.78
4.90	469	0.70
4.67	647	0.60
4.13	1191	0.35
3.78	1843	0.19
3.41	3428	0.03

Bail Test: #2
Well #: 1-2
Date: 6/25/86
Static Head, H (m): 4.46
Bailed Head, Ho (m): 5.12
Starting Time: 2:54 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.58	-172	
5.29	-165	
5.21	-152	
5.18	-134	
5.15	-96	
5.14	-55	
5.12	0	1.00
5.12	63	0.99
5.11	188	0.98
5.10	365	0.96
5.07	910	0.92
5.04	1561	0.88
5.00	3146	0.81
4.96	4011	0.76
4.90	6111	0.65
4.82	8196	0.55

Bail Test: #2
Well #: 1-3
Date: 6/25/86
Static Head, H (m): 4.54
Bailed Head, Ho (m): 5.39
Starting Time: 10:39 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.39	0	1.00
5.28	14	0.86
5.19	44	0.76
5.11	69	0.67
5.07	89	0.62
5.04	124	0.58
4.98	161	0.52
4.94	202	0.47
4.90	257	0.42
4.86	319	0.37
4.80	444	0.30
4.71	622	0.20
4.59	1167	0.05
4.56	1817	0.01
4.57	3402	0.03
4.55	4146	0.01
4.53	6367	-0.02

Bail Test: #2
Well #: 1-4
Date: 6/25/86
Static Head, H (m): 4.60
Bailed Head, Ho (m): 4.88
Starting Time: 1:21 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.42	-100	
5.33	-97	
5.23	-92	
5.04	-75	
4.96	-61	
4.90	-36	
4.88	0	1.00
4.87	41	0.98
4.86	96	0.91
4.85	159	0.90
4.84	184	0.85
4.83	462	0.82
4.80	1006	0.73
4.78	1656	0.66
4.74	3242	0.49
4.70	4104	0.37
4.58	6205	-0.05

Bail Test: #2
Well #: 1-5
Date: 6/25/86
Static Head, H (m): 3.46
Bailed Head, Ho (m): 4.54
Starting Time: 12:18 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
4.95	-68	
4.85	-66	
4.72	-63	
4.66	-58	
4.61	-47	
4.58	-27	
4.56	-13	
4.54	0	1.00
4.51	29	0.97
4.48	64	0.94
4.45	106	0.91
4.38	160	0.85
4.30	203	0.78
4.18	328	0.66
4.04	506	0.54
3.79	1051	0.31
3.65	1720	0.18

Bail Test: #2
Well #: 1-6
Date: 6/25/86
Static Head, H (m): 4.10
Bailed Head, Ho (m): 4.83
Starting Time: 12:48 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.25	-86	
4.94	-78	
4.87	-56	
4.85	-45	
4.85	-32	
4.83	0	1.00
4.83	35	0.99
4.82	77	0.98
4.82	132	0.98
4.81	194	0.96
4.79	319	0.95
4.78	497	0.93
4.74	1043	0.88
4.71	1691	0.83
4.63	3277	0.73
4.58	4137	0.66
4.31	6241	0.29
4.12	8322	0.03

Slug Test: #1
Well #: 1-1
Date: 8/27/86
Static Head, H (m): 3.53
Bailed Head, Ho (m): 2.48
Starting Time: 8:40 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
2.48	0	1.00
2.49	8	0.99
2.52	35	0.96
2.56	65	0.93
2.61	111	0.88
2.69	182	0.80
2.92	463	0.59
3.02	634	0.48
3.34	1611	0.18
3.44	3020	0.09
3.47	4685	0.06
3.47	6185	0.05
3.49	7440	0.04
3.53	10190	0.00

Slug Test: #1
Well #: 1-2
Date: 8/27/86
Static Head, H (m): 4.36
Bailed Head, Ho (m): 3.47
Starting Time: 8:46 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
3.47	0	1.00
3.47	31	0.99
3.48	60	0.98
3.49	107	0.98
3.50	177	0.97
3.65	458	0.80
3.75	629	0.68
3.86	1606	0.55
3.95	3014	0.46
4.03	4679	0.37
4.08	6179	0.31
4.12	7434	0.26
4.19	10184	0.19
4.22	11534	0.16
4.24	12944	0.13
4.28	14524	0.09
4.32	17329	0.04
4.32	18724	0.04

Slug Test: #1
Well #: 1-3
Date: 8/27/86
Static Head, H (m): 4.75
Bailed Head, Ho (m): 3.70
Starting Time: 8:53 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
3.70	0	1.00
3.72	55	0.99
3.72	101	0.98
3.74	171	0.97
3.81	452	0.90
3.85	623	0.86
4.03	1600	0.69
4.22	3007	0.51
4.38	4672	0.36
4.48	6172	0.26
4.55	7427	0.19
4.62	10177	0.12
4.66	11527	0.09
4.69	12937	0.06
4.73	14517	0.03
4.78	17322	-0.02
4.80	18717	-0.05

Slug Test: #1
Well #: 1-4
Date: 8/27/86
Static Head, H (m): 4.54
Bailed Head, Ho (m): 3.58
Starting Time: 9:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
3.58	0	1.00
3.60	40	0.97
3.62	95	0.95
3.65	164	0.92
3.76	446	0.81
3.82	617	0.75
4.06	1594	0.50
4.26	3000	0.29
4.38	4665	0.17
4.43	6165	0.12
4.46	7420	0.09
4.50	10170	0.04
4.51	11520	0.03
4.54	12930	0.01
4.57	14510	-0.03

Slug Test: #1
Well #: 1-5
Date: 8/27/86
Static Head, H (m): 3.58
Bailed Head, Ho (m): 2.93
Starting Time: 9:51 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
2.93	0	1.00
2.94	45	0.99
2.95	115	0.96
3.01	396	0.88
3.03	567	0.84
3.12	1544	0.71
3.16	2949	0.64
3.20	4614	0.58
3.23	6114	0.53
3.26	7369	0.50
3.29	10119	0.45
3.30	11469	0.43
3.32	12879	0.40
3.33	14459	0.38
3.38	17324	0.31
3.39	18719	0.29

Slug Test: #1
Well #: 1-6
Date: 8/27/86
Static Head, H (m): 4.01
Bailed Head, Ho (m): 3.12
Starting Time: 9:12 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
3.12	0	1.00
3.14	41	0.98
3.15	85	0.97
3.17	155	0.95
3.23	436	0.87
3.27	607	0.83
3.44	1585	0.65
3.59	2988	0.48
3.71	4653	0.34
3.78	6153	0.26
3.83	7408	0.20
3.90	10158	0.13
3.92	11508	0.11
3.96	12918	0.06
3.99	14498	0.03
4.02	17363	-0.01

Bail Test: #3
Well #: 1-1
Date: 9/25/86
Static Head, H (m): 2.84
Bailed Head, Ho (m): 5.08
Starting Time: 10:03 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.08	0	1.00
5.04	18	0.98
4.93	59	0.93
4.78	118	0.87
4.62	182	0.79
4.48	240	0.73
3.76	522	0.41
2.83	1347	-0.01

Bail Test: #3
Well #: 1-2
Date: 9/25/86
Static Head, H (m): 4.26
Bailed Head, Ho (m): 5.03
Starting Time: 1:06 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.23	-166	
5.07	-123	
5.04	-64	
5.03	0	1.00
5.02	58	0.99
4.99	339	0.95
4.91	1164	0.85
4.76	2904	0.65
4.51	4584	0.33
4.32	6059	0.09
4.23	6954	-0.03

Bail Test: #3
Well #: 1-3
Date: 9/25/86
Static Head, H (m): 4.47
Bailed Head, Ho (m): 4.98
Starting Time: 1:07 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.15	-183	
5.10	-164	
5.04	-123	
5.00	-64	
4.98	0	1.00
4.96	58	0.97
4.91	338	0.87
4.84	1163	0.72
4.69	2903	0.44
4.60	4583	0.26
4.52	6058	0.11
4.49	6953	0.05
4.46	8458	-0.02

Bail Test: #3
Well #: 1-4
Date: 9/25/86
Static Head, H (m): 4.09
Bailed Head, Ho (m): 4.75
Starting Time: 1:08 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.00	-170	
4.81	-123	
4.77	-64	
4.75	0	1.00
4.73	58	0.97
4.65	337	0.86
4.36	1162	0.41
4.06	2902	-0.04

Bail Test: #3
Well #: 1-5
Date: 9/25/86
Static Head, H (m): 2.46
Bailed Head, Ho (m): 4.52
Starting Time: 10:24 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
4.70	-24	
4.52	0	1.00
4.19	42	0.84
3.78	101	0.64
3.43	165	0.47
3.19	223	0.36
2.57	501	0.05
2.37	1326	-0.04

Bail Test: #3
Well #: 1-6
Date: 9/25/86
Static Head, H (m): 3.52
Bailed Head, Ho (m): 4.71
Starting Time: 1:10 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
4.82	-171	
4.74	-122	
4.72	-64	
4.71	0	1.00
4.69	58	0.99
4.65	335	0.95
4.53	1160	0.85
4.06	2900	0.45
3.81	4580	0.24
3.66	6055	0.12
3.59	6950	0.06
3.52	8455	0.00

Initial Recovery
Well #: 2-1
Date: 7/10/86
Static Head, H (m): 2.54
Bailed Head, Ho (m): 5.23
Starting Time: 6:30 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.93	-43090	
6.89	-41715	
6.84	-40135	
6.69	-33750	
6.64	-31264	
6.59	-29565	
6.53	-27225	
6.49	-25465	
6.41	-22350	
6.37	-20890	
6.34	-19515	
6.30	-17940	
6.10	-11040	
5.23	0	1.00
4.99	3210	0.91
4.47	5760	0.72
4.25	9470	0.64
3.58	14500	0.39
3.44	18740	0.34
3.33	22010	0.30
3.23	26590	0.26
3.12	32150	0.22
3.07	35390	0.20

Initial Recovery
Well #: 2-2
Date: 6/12/86
Static Head, H (m): 2.57
Bailed Head, Ho (m): 5.28
Starting Time: 9:35 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.34	-2955	
6.06	-1580	
5.28	0	1.00
3.52	6385	0.35
3.23	8870	0.25
3.10	10570	0.20
3.02	12910	0.17
2.98	14670	0.15
2.97	19245	0.15
2.57	29095	0.00

Initial Recovery
Well #: 2-3
Date: 6/16/86
Static Head, H (m): 2.28
Bailed Head, Ho (m): 3.42
Starting Time: 8:00p.m.

Head, h (m)	Time (min)	H - h/H- Ho
7.07	-10920	
6.67	-7965	
6.25	-6385	
3.42	0	1.00
3.21	2485	0.82
3.23	4185	0.83
3.25	6525	0.85
3.26	8285	0.85

Initial Recovery
Well #: 2-4
Date: 6/16/86
Static Head, H (m): 2.41
Bailed Head, Ho (m): 5.85
Starting Time: 8:00 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
7.20	-10920	
6.86	-7965	
6.58	-6385	
5.85	0	1.00
4.97	2485	0.75
4.44	4185	0.59
4.02	6525	0.47
3.73	8285	0.38
3.40	11400	0.29
3.29	12860	0.25
3.21	14235	0.23
3.13	15810	0.21
2.79	22710	0.11
2.64	33750	0.07
2.61	36960	0.06

Initial Recovery
Well #: 2-5
Date: 7/20/86
Static Head, H (m): 2.46
Bailed Head, Ho (m): 5.43
Starting Time: 7:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.81	-39630	
6.75	-38255	
6.70	-36680	
6.51	-29780	
6.17	-18740	
6.08	-15530	
6.00	-12980	
5.91	-9270	
5.76	-4240	
5.43	0	1.00
5.09	3270	0.88
4.71	7850	0.76
4.31	13410	0.62
4.13	16650	0.56
3.84	21560	0.46
3.60	25990	0.38
3.43	30310	0.32

Initial Recovery
Well #: 2-6
Date: 8/3/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.35
Starting Time: 6:30 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
7.25	-61190	
7.19	-59815	
7.16	-58240	
7.02	-51340	
6.76	-40300	
6.68	-37090	
6.62	-34540	
6.53	-30830	
6.40	-25800	
6.28	-21560	
6.19	-18290	
6.06	-13710	
5.92	-8150	
5.76	-4910	
5.35	0	1.00
5.01	4430	0.88
4.74	8750	0.79

Initial Recovery
Well #: 2-7
Date:
Static Head, H (m): 0.00
Bailed Head, Ho (m): 0.00
Starting Time:

Head, h (m)	Time (min)	H - h/H- Ho

** NO INITIAL RECOVERY DATA **

Initial Recovery
Well #: 2-8
Date: 8/10/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.66
Starting Time: 8:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
7.48	-67080	
7.37	-64245	
7.21	-55770	
6.90	-41520	
6.77	-35260	
6.54	-25990	
6.35	-18140	
6.14	-9340	
5.66	0	1.00
5.37	4320	0.91
5.00	10080	0.80
4.64	17280	0.68
4.29	24690	0.58
4.07	30720	0.51
3.91	34720	0.46
3.74	40870	0.40
3.56	47700	0.35
3.35	54735	0.28

Initial Recovery
Well #: 2-9
Date: 7/20/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.30
Starting Time: 8:30 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
7.23	-44065	
7.09	-39965	
6.93	-35390	
6.84	-32440	
6.24	-14500	
6.07	-8740	
5.30	0	1.00
4.90	4240	0.86
4.60	7510	0.76
4.29	12090	0.65
3.98	17650	0.54
3.84	20890	0.49
3.62	25800	0.41
3.40	30230	0.34
3.26	34550	0.29
3.09	40310	0.23
2.87	54920	0.15
2.78	60950	0.12
2.73	64970	0.10
2.72	71100	0.10
2.67	77930	0.08
2.53	84965	0.03

Initial Recovery
Well #: 2-10
Date: 8/3/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.70
Starting Time: 8:20 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
7.04	-64955	
6.96	-60855	
6.87	-56280	
6.81	-53330	
6.43	-35390	
6.30	-29630	
6.12	-20890	
5.95	-13380	
5.70	0	1.00
5.37	4910	0.90
5.07	9340	0.80
4.82	13660	0.73
4.49	19420	0.63
4.16	26620	0.53
3.86	34030	0.44
3.65	40060	0.37
3.52	44080	0.33
3.36	50210	0.28
3.19	57040	0.23
3.01	64075	0.17

Initial Recovery
Well #: 2-11
Date: 6/21/86
Static Head, H (m): 2.94
Bailed Head, Ho (m): 6.00
Starting Time: 8:45 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.83	-2340	
6.00	0	1.00
4.44	1760	0.49
3.17	4875	0.08
2.96	6335	0.01
2.94	7710	0.00

Initial Recovery
Well #: 2-12
Date: 7/10/86
Static Head, H (m): 2.12
Bailed Head, Ho (m): 5.82
Starting Time: 6:30 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
7.12	-22350	
7.04	-20890	
6.98	-19515	
6.90	-17940	
6.54	-11040	
5.82	0	1.00
5.25	3210	0.85
4.90	5760	0.75
4.42	9470	0.62
3.87	14500	0.47
3.52	18740	0.38
3.26	22010	0.31
2.99	26590	0.24
2.76	32150	0.17
2.66	35390	0.15
2.51	40300	0.11
2.40	44730	0.08
2.33	49050	0.06
2.23	54810	0.03
2.19	62010	0.02

Bail Test: #1
Well #: 2-1
Date: 9/4/86
Static Head, H (m): 2.54
Bailed Head, Ho (m): 6.04
Starting Time: 2:36 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.77	-256	
6.53	-253	
6.39	-250	
6.25	-245	
6.17	-238	
6.09	-198	
6.07	-165	
6.04	-95	
6.04	0	1.00
6.03	50	1.00
6.00	228	0.99
5.91	1104	0.96
5.86	1544	0.95
5.75	2629	0.92
5.57	4474	0.87
5.48	5434	0.84
5.35	6834	0.80
5.09	8809	0.73
3.54	11304	0.29
3.47	12689	0.27
3.40	14399	0.25
3.24	18395	0.20
3.06	24440	0.15

Bail Test: #1
Well #: 2-2
Date: 9/5/86
Static Head, H (m): 2.20
Bailed Head, Ho (m): 5.25
Starting Time: 9:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.96	-1347	
6.78	-1344	
6.64	-1340	
6.30	-1320	
6.24	-1304	
6.18	-1267	
6.14	-1198	
6.11	-1164	
6.06	-1102	
6.03	-947	
5.88	-875	
5.25	0	1.00
4.94	440	0.90
4.32	1525	0.70
3.44	3370	0.41
3.12	4330	0.30
2.81	5730	0.20
2.64	7705	0.14
1.90	10200	-0.10

Bail Test: #1
Well #: 2-3
Date: 9/5/86
Static Head, H (m): 2.28
Bailed Head, Ho (m): 4.90
Starting Time: 9:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.49	-1323	
6.11	-1317	
5.94	-1310	
5.93	-1301	
5.91	-1283	
5.86	-1249	
5.81	-1197	
5.77	-1162	
5.71	-1100	
5.67	-1052	
5.51	-874	
4.90	0	1.00
4.66	44	0.91
4.20	1525	0.73
3.65	3370	0.52
3.45	4330	0.45
3.23	5730	0.36
2.98	7705	0.27
2.40	10200	0.05
2.26	11585	-0.01

Bail Test: #1
Well #: 2-4
Date: 9/4/86
Static Head, H (m): 2.41
Bailed Head, Ho (m): 5.54
Starting Time: 6:27 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.49	-449	
6.24	-442	
6.19	-436	
6.14	-427	
6.09	-408	
6.05	-377	
5.99	-323	
5.97	-287	
5.88	-226	
5.85	-178	
5.54	0	1.00
4.42	873	0.64
4.01	1313	0.51
3.25	2398	0.27
2.55	4243	0.04
2.41	5203	0.00

Bail Test: #1
Well #: 2-5
Date: 9/23/86
Static Head, H (m): 2.46
Bailed Head, Ho (m): 5.77
Starting Time: 7:14 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.89	-472	
5.87	-436	
5.85	-378	
5.82	-283	
5.81	-231	
5.77	0	1.00
5.64	896	0.96
5.56	1376	0.94
5.43	2226	0.90
5.35	2831	0.87
5.01	5396	0.77
4.81	7081	0.71
4.64	8551	0.66
4.53	9446	0.63
4.36	10956	0.57
3.92	15346	0.44
3.49	20911	0.31
3.01	29431	0.17
2.49	49876	0.01

Bail Test: #1
Well #: 2-6
Date: 9/23/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.94
Starting Time: 7:15 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.03	-473	
6.00	-437	
5.98	-377	
5.97	-282	
5.94	0	1.00
5.85	895	0.97
5.79	1375	0.96
5.69	2225	0.93
5.62	2830	0.91
5.36	5395	0.83
5.18	7080	0.78
5.03	8550	0.74
4.96	9445	0.72
4.82	10955	0.68
4.43	15345	0.57
4.03	20910	0.45
3.53	29430	0.31
2.85	49875	0.12

Bail Test: #1
Well #: 2-7
Date: 9/5/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.22
Starting Time: 9:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.36	-1288	
6.11	-1285	
6.04	-1287	
6.01	-1271	
5.99	-1251	
5.93	-1192	
5.90	-1159	
5.85	-1098	
5.82	-1050	
5.70	-872	
5.22	0	1.00
5.03	440	0.93
4.63	1525	0.79
4.12	3370	0.60
3.91	4330	0.53
3.66	5730	0.44
3.33	7705	0.32
2.79	10200	0.12
2.60	11585	0.06
2.42	13295	-0.01

Bail Test: #1
Well #: 2-8
Date: 9/25/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.94
Starting Time: 6:25 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.23	-3510	
6.20	-3457	
6.19	-3362	
6.18	-3302	
6.17	-3070	
6.12	-2180	
6.10	-1700	
6.00	-605	
5.94	0	1.00
5.70	2565	0.93
5.53	4250	0.88
5.39	5720	0.84
5.32	6615	0.82
5.20	8125	0.79
4.83	12515	0.68
4.41	18080	0.56
3.93	26600	0.42
3.24	47045	0.23

Bail Test: #1
Well #: 2-9
Date: 9/23/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.96
Starting Time: 7:19 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.07	-432	
6.05	-376	
6.03	-280	
6.02	-230	
5.96	0	1.00
5.80	891	0.95
5.71	1371	0.93
5.57	2221	0.89
5.47	2826	0.86
5.09	5391	0.75
4.86	7076	0.69
4.66	8546	0.63
4.57	9441	0.60
4.39	10951	0.55
3.94	15341	0.43
3.49	20906	0.30
3.05	29426	0.17
2.58	49871	0.04

Bail Test: #1
Well #: 2-10
Date: 9/23/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.87
Starting Time: 7:18 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.92	-432	
5.89	-379	
5.89	-283	
5.88	-232	
5.87	0	1.00
5.82	892	0.99
5.81	1372	0.98
5.78	2222	0.97
5.73	2827	0.96
5.60	5392	0.92
5.45	7077	0.88
5.31	8547	0.84
5.24	9442	0.82
5.12	10952	0.78
4.76	15342	0.68
4.36	20907	0.56
3.86	29427	0.42
3.12	49872	0.20

Bail Test: #1
Well #: 2-11
Date: 9/4/86
Static Head, H (m): 1.89
Bailed Head, Ho (m): 5.41
Starting Time: 6:29 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.52	-392	
6.30	-388	
6.06	-382	
5.99	-376	
5.95	-357	
5.88	-319	
5.83	-286	
5.72	-225	
5.65	-178	
5.41	0	1.00
4.53	871	0.75
4.20	1311	0.66
3.61	2396	0.49
2.98	4241	0.31
2.77	5201	0.25
2.55	6601	0.19
2.31	8576	0.12
1.64	11071	-0.07

Bail Test: #1
Well #: 2-12
Date: 9/4/86
Static Head, H (m): 2.12
Bailed Head, Ho (m): 5.95
Starting Time: 6:30 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.41	-502	
6.25	-498	
6.15	-490	
6.10	-477	
6.05	-439	
6.04	-406	
6.01	-345	
5.98	-298	
5.95	0	1.00
5.73	750	0.94
5.59	1190	0.91
5.30	2275	0.83
4.89	4120	0.72
4.71	5080	0.68
4.46	6480	0.61
4.11	8455	0.52
3.80	10950	0.44
3.65	12335	0.40
3.48	14045	0.36
3.13	17785	0.27
2.73	23830	0.16

Bail Test: #1
Well #: 2-13
Date: 9/5/86
Static Head, H (m): 2.47
Bailed Head, Ho (m): 3.69
Starting Time: 9:10 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.71	-1230	
6.65	-1227	
6.55	-1220	
6.46	-1213	
6.31	-1197	
6.21	-1185	
6.05	-1164	
5.82	-1127	
5.72	-1104	
5.57	-1072	
5.51	-1056	
4.94	-878	
3.69	0	1.00
3.47	435	0.82
3.15	1525	0.56
2.91	3360	0.36
2.86	4320	0.32
2.79	5720	0.26

Bail Test: #1
Well #: 2-14
Date: 9/5/86
Static Head, H (m): 2.35
Bailed Head, Ho (m): 4.37
Starting Time: 9:10 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.60	-1211	
6.54	-1204	
6.42	-1186	
6.32	-1163	
6.20	-1126	
6.12	-1103	
6.02	-1071	
5.97	-1055	
5.55	-876	
4.37	0	1.00
4.04	435	0.84
3.54	1525	0.59
3.12	3360	0.38
2.98	4320	0.31
2.83	5720	0.24

Bail Test: #2
Well #: 2-1
Date: 9/23/86
Static Head, H (m): 2.54
Bailed Head, Ho (m): 5.95
Starting Time: 7:11 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.16	-477	
6.11	-441	
6.07	-379	
6.03	-284	
6.01	-232	
5.95	0	-0.212
5.81	899	-0.224
5.75	1379	-0.230
5.32	2229	-0.268
5.07	2834	-0.290
4.57	5399	-0.335
4.43	7084	-0.348
3.84	8554	-0.401
3.57	9449	-0.424
3.32	10959	-0.447
3.05	15349	-0.471
2.74	20914	-0.499
2.57	29434	-0.514
2.51	49879	-0.519

Bail Test: #2
Well #: 2-2
Date: 9/23/86
Static Head, H (m): 2.20
Bailed Head, Ho (m): 5.77
Starting Time: 2:28 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.14	-193	
6.07	-157	
5.95	-95	
5.77	0	-0.124
5.69	52	-0.132
5.31	284	-0.164
4.13	1184	-0.265
3.63	1664	-0.308
2.82	2514	-0.377
2.37	3119	-0.415
1.49	4244	-0.490

Bail Test: #2
Well #: 2-3
Date: 9/23/86
Static Head, H (m): 2.28
Bailed Head, Ho (m): 5.71
Starting Time: 2:29 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.93	-193	
5.88	-157	
5.81	-96	
5.71	0	-0.157
5.66	58	-0.162
5.44	284	-0.181
4.78	1184	-0.240
4.48	1664	-0.267
3.98	2514	-0.311
3.70	3119	-0.336
2.89	5684	-0.408
2.64	7369	-0.431
2.37	8839	-0.454
2.27	8294	-0.463

Bail Test: #2
Well #: 2-4
Date: 9/23/86
Static Head, H (m): 1.70
Bailed Head, Ho (m): 5.24
Starting Time: 2:30 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.89	-193	0.027
5.75	-157	0.015
5.54	-96	-0.003
5.24	0	-0.029
5.09	58	-0.042
4.50	284	-0.093
2.95	1184	-0.227
2.46	1664	-0.269
1.75	2514	-0.330
1.61	3119	-0.342

Bail Test: #2
Well #: 2-5
Date: 12/1/86
Static Head, H (m): 2.39
Bailed Head, Ho (m): 5.03
Starting Time: 11:05 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.03	0	-0.324
4.79	1290	-0.351
4.62	2730	-0.371
4.50	4350	-0.385
4.15	8685	-0.425
3.73	14700	-0.473
3.30	24700	-0.523
2.71	60465	-0.592

Bail Test: #2
Well #: 2-6
Date: 12/1/86
Static Head, H (m): 2.54
Bailed Head, Ho (m): 5.57
Starting Time: 11:06 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.57	0	-0.279
5.38	1290	-0.298
5.23	2730	-0.313
5.11	4350	-0.325
4.77	8685	-0.359
4.36	14700	-0.401
3.88	24700	-0.449
3.06	60465	-0.532

Bail Test: #2
Well #: 2-7
Date: 9/23/86
Static Head, H (m): 2.02
Bailed Head, Ho (m): 5.79
Starting Time: 7:20 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
6.17	-440	-0.037
6.10	-387	-0.043
6.01	-292	-0.050
5.97	-232	-0.053
5.79	0	-0.068
5.26	890	-0.111
5.01	1370	-0.131
4.63	2220	-0.161
4.40	2825	-0.180
3.64	5390	-0.242
3.26	7075	-0.272
2.99	8545	-0.294
2.86	9440	-0.305
2.65	10950	-0.321
2.23	15340	-0.356
1.96	20905	-0.377

Bail Test: #2
Well #: 2-8
Date: 12/1/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.29
Starting Time: 11:09 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.29	0	-0.289
5.11	1290	-0.309
4.99	2730	-0.322
4.90	4350	-0.331
4.63	8685	-0.360
4.29	14700	-0.396
3.89	24700	-0.439
3.15	60465	-0.519

Bail Test: #2
Well #: 2-9
Date: 12/1/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.31
Starting Time: 11:10 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.31	0	-0.286
5.08	1290	-0.311
4.90	2730	-0.330
4.76	4350	-0.344
4.37	8685	-0.385
3.93	14700	-0.433
3.44	24700	-0.484
2.75	60465	-0.558

Bail Test: #2
Well #: 2-10
Date: 12/1/86
Static Head, H (m): 2.44
Bailed Head, Ho (m): 5.75
Starting Time: 11:11 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
5.75	0	-0.206
5.60	1290	-0.220
5.46	2730	-0.233
5.36	4350	-0.243
5.03	8685	-0.273
4.64	14700	-0.309
4.14	24700	-0.355
3.16	60465	-0.445

Bail Test: #2
Well #: 2-11
Date: 9/23/86
Static Head, H (m): 1.38
Bailed Head, Ho (m): 5.68
Starting Time: 2:34 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
20.37	-147	1.123
19.48	-96	1.060
18.63	0	1.000
18.20	51	0.969
16.43	283	0.844
11.81	1183	0.516
10.13	1663	0.397
7.69	2513	0.224
6.33	3118	0.127
4.12	4243	-0.030

Bail Test: #2
Well #: 2-12
Date: 9/23/86
Static Head, H (m): 2.12
Bailed Head, Ho (m): 5.68
Starting Time: 7:16 p.m.

Head, h (m)	Time (min)	H - h/H- Ho
19.62	-428	1.084
19.38	-354	1.063
19.15	-259	1.044
19.05	-232	1.035
18.64	0	1.000
17.59	894	0.910
17.70	1374	0.920
16.36	2224	0.805
15.89	2829	0.765
14.19	5394	0.620
13.23	7079	0.538
12.44	8549	0.470
12.05	9444	0.437
11.41	10954	0.382
9.79	15344	0.244
8.41	20909	0.126
7.11	29429	0.015

Bail Test: #2
Well #: 2-13
Date: 9/24/86
Static Head, H (m): 2.50
Bailed Head, Ho (m): 3.30
Starting Time: 10:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
17.11	-1256	
16.80	-1235	
16.45	-1208	
16.06	-1176	
15.66	-1141	
15.38	-1110	
13.68	-891	
10.84	0	1.000
10.08	490	0.712
9.12	1340	0.348
8.68	1945	0.182
7.70	4510	-0.189

Bail Test: #2
Well #: 2-14
Date: 9/24/86
Static Head, H (m): 2.13
Bailed Head, Ho (m): 3.62
Starting Time: 10:00 a.m.

Head, h (m)	Time (min)	H - h/H- Ho
19.05	-1255	2.469
18.85	-1234	2.428
18.62	-1206	2.381
18.35	-1175	2.326
18.10	-1140	2.275
17.85	-1109	2.223
16.39	-891	1.924
11.88	0	1.000
11.77	490	0.977
10.41	1340	0.699.
9.75	1945	0.564
8.02	4510	0.209

APPENDIX IV

ONE-DIMENSIONAL CONSOLIDATION TEST DATA USED TO DETERMINE IN-SITU VOID RATIO, LABORATORY HYDRAULIC CONDUCTIVITY, AND PRECONCOLIDATION PRESSURE OF SOIL SAMPLES FROM FIELD SITES 1 AND 2

**CALCULATIONS AND GRAPHICAL INTERPRETATIONS USED TO CALCULATE IN-SITU VOID RATIO, HYDRAULIC CONDUCTIVITY AND PRECONSOLIDATION PRESSURES FOR THE SOIL SAMPLES AT SITES 1 AND 2 WILL BE ON FILE AT THE DEPARTMENT OF GEOLOGICAL AND GEOPHYSICAL SCIENCES OFFICE, UNIVERSITY OF WISCONSIN MILWAUKEE.

CONSOLIDATION TEST * #1 *

Project Field Site 1 Boring No. 1-3 Sample Depth 12'

Soil Description Gray silty-clay

Ring Diameter 2.50 in. Ring Height 1.00 in. Ring Weight 113.59 gm

	Before Test	After Test
Wt Wet Soil and Ring	278.69 gm	277.05 gm
Wt Dry Soil and Ring	246.05 gm	246.05 gm
Wt Water	32.64 gm	31.00
Wt Dry Soil	132.46 gm	132.46
Water Content	24.64 %	23.40 %

	Initial Conditions
Volume of Ring, V	4.909 in.^{3} = 80.44 cm^3
Volume of Solids, Vs	2.972 in.^{3} = 48.70 cm^3
Volume of Voids, Vv	1.937 in.^{3} = 31.74 cm^3
Initial Void Ratio, e_o	.651
Initial Saturation, S	100%

Water Content Trimmings	
Tin and Wet Sample	46.35 gm
Tin and Dry Sample	40.24 gm
Wt. Water	6.11 gm
Tin Wt.	15.30 gm
Dry Sample Wt.	24.94 gm
Water Content	24.50%

CONSOLIDATION TEST- Calculation Sheet * #1 *

Sample Diameter 2.50 in. Sample Area 4.909 in^2

Initial Sample Height 1.00 in. Initial void ratio .651

Initial dial reading 0.0000 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	1.54	3.45	0.0123	.0123	1.23E-2	.631	2.1E-7	2.3E-7
2	3.07	6.87	0.0192	.0192	1.92E-2	.620	1.8E-7	7.5E-8
3	6.14	13.75	0.02876	.02876	2.88E-2	.604	7.6E-8	5.0E-8
4	12.31	27.58	0.04287	.04287	4.29E-2	.581	5.9E-8	2.9E-8
5	24.73	55.41	0.06189	.06189	6.19E-2	.550	4.8E-8	1.6E-8
6	49.56	111.05	0.08709	.08709	8.71E-2	.508	2.5E-8	9.3E-9
7	24.73	55.41	0.08291	.08291	8.29E-2	.514		
8	6.14	13.75	0.07107	.07107	7.11E-2	.534		
9	1.54	3.45	0.06005	.06005	6.01E-2	.552		

Load #	Sq. Rt.of Time Method t_{90}	c_v	m_v	Log. of Time Method t_{50}	c_v	m_v
1	5.38	.0392	3.57E-3	1.18	.042	3.57E-3
2	3.50	.0588	2.02E-3	1.99	.024	2.02E-3
3	2.19	.0357	1.39E-3	2.00	.0235	1.39E-3
4	1.34	.0380	1.02E-3	2.45	.0188	1.02E-3
5	4.00	.046	6.83E-4	2.73	.0157	6.83E-4
6	4.75	.0364	4.53E-4	3.00	.0134	4.53E-4

CONSOLIDATION TEST * #2 *

Project Field Site 1 Boring No. 1-4 Sample Depth 14'

Soil Description Gray silty-clay

Ring Diameter 2.50 in. Ring Height 1.00 in. Ring Weight 112.79 gm

	Before Test	After Test
Wt Wet Soil and Ring	282.97 gm	282.56 gm
Wt Dry Soil and Ring	250.09 gm	250.09 gm
Wt Water	32.88 gm	32.47 gm
Wt Dry Soil	137.30 gm	137.30 gm
Water Content	23.95 %	23.65 %

	Initial Conditions
Volume of Ring, V	4.909 in^3 = 80.44 cm^3
Volume of Solids, Vs	3.078 in^3 = 50.44 cm^3
Volume of Voids, Vv	1.831 in^3 = 30.00 cm^3
Initial Void Ratio, e_o	.593
Initial Saturation, S	109 %

Water Content Trimmings	
Tin and Wet Sample	60.27 gm
Tin and Dry Sample	52.22 gm
Wt. Water	8.05 gm
Tin Wt.	15.54 gm
Dry Sample Wt.	36.48 gm
Water Content	22.07 %

CONSOLIDATION TEST- Calculation Sheet * #2 *

Sample Diameter 2.50 in. Sample Area 4.909 in.^{2}

Initial Sample Height 1.00 in. Initial void ratio .593

Initial dial reading 0.0000 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	1.54	3.45	.00698	.00698	6.98E-3	.581	1.8E-7	1.1E-7
2	3.07	6.88	.0116	.0116	1.16E-2	.574	2.0E-7	6.1E-8
3	6.14	13.76	.01855	.0186	1.86E-2	.562	9.5E-8	3.7E-8
4	12.34	27.65	.03063	.0306	3.06E-2	.543	6.1E-8	2.0E-8
5	24.77	55.50	.04741	.0474	4.74E-2	.516	3.6E-8	1.2E-8
6	49.46	110.83	.07105	.0711	7.11E-2	.478	2.5E-8	1.3E-8
7	24.77	55.50	.06668	.0667	6.67E-2	.487		
8	6.14	13.76	.06366	.0637	6.37E-2	.491		
9	1.54	3.45	.04140	.0414	4.14E-2	.527		

Load #	Sq. Rt. of Time Method t_{90}	c_v	m_v	Log. of Time Method t_{50}	c_v	m_v
1	3.53	.0594	2.02E-3	1.42	.0346	2.02E-3
2	2.19	.095	1.35E-3	1.63	.030	1.35E-3
3	3.35	.0613	1.01E-3	1.98	.0242	1.01E-3
4	4.37	.046	8.70E-4	3.18	.0148	8.70E-4
5	4.97	.0393	6.03E-4	3.40	.0134	6.03E-4
6	4.93	.038	4.27E-4	2.40	.0183	4.27E-4

CONSOLIDATION TEST * #3 *

Project Field Site 2 Boring No. 2-6 Sample Depth 14'

Soil Description Gray silty-clay

Ring Diameter 2.800 in. Ring Height .7495 in. Ring Weight 91.82 gm

	Before Test	After Test
Wt Wet Soil and Ring	265.28 gm	265.76 gm
Wt Dry Soil and Ring	242.05 gm	242.05 gm
Wt Water	23.23 gm	23.51 gm
Wt Dry Soil	150.23 gm	150.23 gm
Water Content	15.46 %	15.65 %

	Initial Conditions
Volume of Ring, V	4.615 in.^{3} = 75.64 cm^3
Volume of Solids, Vs	3.321 in.^{3} = 54.42 cm^3
Volume of Voids, Vv	1.294 in.^{3} = 21.21 cm^3
Initial Void Ratio, e_o	.394
Initial Saturation, S	109%

Water Content Trimmings	
Tin and Wet Sample	33.39 gm
Tin and Dry Sample	30.77 gm
Wt. Water	2.62 gm
Tin Wt.	15.25 gm
Dry Sample Wt.	15.52 gm
Water Content	16.88 %

CONSOLIDATION TEST- Calculation Sheet * #3 *

Sample Diameter 2.800 in. Sample Area 6.158 $in.^2$

Initial Sample Height 0.7495 in. Initial void ratio .394

Initial dial reading 2.2000 $*H = R_o - R$

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	2.21	3.95	2.17833	.0217	2.90E-2	.354	***	1.9E-7
2	4.41	7.88	2.17378	.0262	3.50E-2	.346	***	****
3	8.82	15.76	2.16850	.0315	4.20E-2	.336	7.5E-8	3.1E-8
4	17.66	31.55	2.16179	.0382	5.10E-2	.324	3.2E-8	1.6E-8
5	35.32	63.09	2.15010	.0499	6.66E-2	.302	2.6E-8	1.1E-8
6	70.55	125.20	2.13365	.0664	8.85E-2	.271	1.6E-8	8.2E-9
7	35.32	63.09	2.13608	.0639	8.53E-2	.276		
8	8.82	15.76	2.14305	.0560	7.47E-2	.290		
9	2.21	3.95	2.15107	.0489	6.53E-2	.302		

Load #	Sq. Rt.of Time Method			Log. of Time Method		
	t_{90}	c_v	m_v	t_{50}	c_v	m_v
1	***	***	7.34E-3	1.65	.0167	7.34E-3
2	***	***	1.53E-3	***	***	1.53E-3
3	1.99	.0555	8.88E-4	1.13	.0227	8.88E-4
4	2.99	.0363	5.70E-4	1.39	.0182	5.70E-4
5	3.13	.0338	4.95E-4	1.68	.0147	4.95E-4
6	3.28	.0295	3.53E-4	1.57	.0151	3.53E-4

CONSOLIDATION TEST * #4 *

Project Field Site 2 Boring No. 2-6 Sample Depth 17'

Soil Description Gray silty-clay

Ring Diameter 2.800 in. Ring Height .7495 in. Ring Weight 87.04 gm

	Before Test	After Test
Wt Wet Soil and Ring	254.00 gm	252.01 gm
Wt Dry Soil and Ring	226.10 gm	226.10 gm
Wt Water	27.90 gm	25.91 gm
Wt Dry Soil	139.06 gm	139.06 gm
Water Content	20.06 %	18.63 %

	Initial Conditions
Volume of Ring, V	4.615 in.^{3} = 75.64 cm^3
Volume of Solids, Vs	3.085 in.^{3} = 50.56 cm^3
Volume of Voids, Vv	1.530 in.^{3} = 25.07 cm^3
Initial Void Ratio, e_o	.495
Initial Saturation, S	111 %

Water Content Trimmings	
Tin and Wet Sample	31.85 gm
Tin and Dry Sample	29.15 gm
Wt. Water	2.70 gm
Tin Wt.	15.22 gm
Dry Sample Wt.	13.93 gm
Water Content	19.40 %

CONSOLIDATION TEST- Calculation Sheet * #4 *

Sample Diameter 2.800 in. Sample Area 6.158 $in.^2$

Initial Sample Height .7495 in. Initial void ratio .495

Initial dial reading 2.2500 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	2.19	3.91	2.24253	.0075	9.96E-3	.480	1.3E-7	5.1E-8
2	4.39	7.84	2.23705	.0130	1.73E-2	.469	6.9E-8	3.2E-8
3	8.79	15.70	2.22702	.0230	3.07E-2	.449	6.4E-8	3.1E-8
4	17.34	30.97	2.21775	.0323	4.30E-2	.430	3.4E-8	1.4E-8
5	34.86	62.27	2.20322	.0468	6.24E-2	.401	2.5E-8	1.2E-8
6	70.11	125.24	2.18456	.0654	8.73E-2	.364	1.3E-8	7.5E-9
7	34.86	62.27	2.18772	.0623	8.31E-2	.370		
8	8.79	15.70	2.19654	.0535	7.14E-2	.388		
9	2.19	3.91	2.20572	.0443	5.91E-2	.406		

Load #	Sq. Rt.of Time Method			Log. of Time Method		
	t_{90}	c_v	m_v	t_{50}	c_v	m_v
1	3.42	.0343	2.55E-3	2.11	.0130	2.55E-3
2	4.80	.0242	1.87E-3	2.38	.0113	1.87E-3
3	4.62	.0248	1.70E-3	2.20	.0121	1.70E-3
4	4.00	.0277	8.06E-4	2.29	.0113	8.06E-4
5	4.00	.0258	6.20E-4	1.98	.0126	6.20E-4
6	4.75	.0215	3.96E-4	1.93	.0124	3.95E-4

CONSOLIDATION TEST * #5 *

Project Field Site 2 Boring No. 2-6 Sample Depth 19'

Soil Description Gray silty-clay

Ring Diameter 2.800 in. Ring Height .7495 in Ring Weight 92.25 gm

	Before Test	After Test
Wt Wet Soil and Ring	260.70 gm	259.31 gm
Wt Dry Soil and Ring	232.26 gm	232.26 gm
Wt Water	28.44 gm	27.05 gm
Wt Dry Soil	140.01 gm	140.01 gm
Water Content	20.31 %	19.32 %

	Initial Conditions
Volume of Ring, V	4.615 in.^{3} = 75.64 cm^3
Volume of Solids, Vs	3.106 in.^{3} = 50.90 cm^3
Volume of Voids, Vv	1.509 in.^{3} = 24.73 cm^3
Initial Void Ratio, e_o	.486
Initial Saturation, S	112 %

Water Content Trimmings	
Tin and Wet Sample	37.80 gm
Tin and Dry Sample	34.28 gm
Wt. Water	3.52 gm
Tin Wt.	15.67 gm
Dry Sample Wt.	18.61 gm
Water Content	18.9 %

CONSOLIDATION TEST- Calculation Sheet * #5 *

Sample Diameter 2.800 in. Sample Area 6.158 in.^{2}

Initial Sample Height .7495 in. Initial void ratio .486

Initial dial reading 2.2000 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	2.20	3.93	2.18941	.0106	1.41E-2	.465	1.7E-7	6.3E-8
2	4.41	7.88	2.18467	.0153	2.04E-2	.456	6.4E-8	2.6E-8
3	8.82	15.76	2.17580	.0242	3.23E-2	.438	4.7E-8	2.4E-8
4	17.66	31.54	2.16423	.0358	4.77E-2	.415	4.4E-8	1.7E-8
5	35.23	62.93	2.14912	.0509	6.79E-2	.385	2.0E-8	1.1E-8
6	70.53	125.99	2.12952	.0705	9.40E-2	.346	1.3E-8	8.5E-9
7	35.23	62.93	2.13268	.0673	8.98E-2	.352		
8	8.82	15.76	2.14211	.0579	7.73E-2	.371		
9	2.20	3.93	2.15209	.0489	6.52E-2	.389		

Load #	Sq. Rt. of Time Method			Log. of Time Method		
	t_{90}	c_v	m_v	t_{50}	c_v	m_v
1	3.72	.0315	3.59E-3	2.40	.0115	3.59E-3
2	4.37	.0264	1.59E-3	2.55	.0105	1.59E-3
3	5.57	.0203	1.51E-3	2.56	.0103	1.51E-3
4	3.72	.0296	9.76E-4	2.22	.0116	9.76E-4
5	5.20	.0204	6.44E-4	2.20	.0111	6.44E-4
6	5.06	.0199	4.14E-4	1.75	.0135	4.14E-4

CONSOLIDATION TEST * #6 *

Project Field Site 1 Boring No. 1-1 Sample Depth 9'

Soil Description Gray silty-clay

Ring Diameter 2.800 in. Ring Height .7495 in. Ring Weight 87.05 gm

	Before Test	After Test
Wt Wet Soil and Ring	257.16 gm	257.06 gm
Wt Dry Soil and Ring	229.18 gm	229.18 gm
Wt Water	27.98 gm	27.88 gm
Wt Dry Soil	142.13 gm	142.13 gm
Water Content	19.69 %	19.62 %

	Initial Conditions
Volume of Ring, V	4.615 in.^{3} = 75.64 cm^3
Volume of Solids, Vs	3.188 in.^{3} = 52.26 cm^3
Volume of Voids, Vv	1.427 in.^{3} = 23.38 cm^3
Initial Void Ratio, e_o	.448
Initial Saturation, S	119 %

Water Content Trimmings	
Tin and Wet Sample	39.70 gm
Tin and Dry Sample	35.82 gm
Wt. Water	3.88 gm
Tin Wt.	15.79 gm
Dry Sample Wt.	20.03 gm
Water Content	19.37%

CONSOLIDATION TEST- Calculation Sheet * #6 *

Sample Diameter 2.800 in. Sample Area 6.158 in.^{2}

Initial Sample Height .7495 in. Initial void ratio .448

Initial dial reading 2.2000 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	2.19	3.91	2.19178	.00822	1.10E-2	.432	2.7E-7	1.5E-7
2	4.39	7.84	2.18666	.01334	1.78E-2	.422	1.0E-7	7.2E-8
3	8.80	15.72	2.17845	.02155	2.88E-2	.406	1.2E-7	6.7E-8
4	17.38	31.05	2.15791	.03209	4.28E-2	.386	7.9E-8	4.0E-8
5	34.90	62.34	2.15392	.04608	6.15E-2	.359	3.8E-8	2.0E-8
6	70.26	125.50	2.13638	.06362	8.49E-2	.325	2.1E-8	1.5E-8
7	34.90	62.34	2.14002	.05998	8.00E-2			
8	8.80	15.72	2.14973	.05027	6.71E-2			
9	2.19	3.91	2.15878	.04122	5.50E-2			

Load #	Sq. Rt.of Time Method			Log. of Time Method		
	t_{90}	c_v	m_v	t_{50}	c_v	m_v
1	1.90	.0623	2.81E-3	.790	.0349	2.81E-3
2	3.03	.0883	1.73E-3	.990	.0273	1.73E-3
3	2.04	.0559	1.40E-3	.850	.0313	1.40E-3
4	1.96	.0568	9.13E-4	.900	.0289	9.13E-4
5	2.59	.0415	5.98E-4	1.15	.0218	5.98E-4
6	2.72	.0377	3.70E-4	.900	.0267	3.70E-4

CONSOLIDATION TEST * #7 *

Project Field Site 1 Boring No. 1-3 Sample Depth 12'

Soil Description Gray silty-clay

Ring Diameter 2.800 in. Ring Height .7495 in. Ring Weight 92.23 gm

	Before Test	After Test
Wt Wet Soil and Ring	249.31 gm	247.28 gm
Wt Dry Soil and Ring	216.12 gm	216.12 gm
Wt Water	33.19 gm	31.16 gm
Wt Dry Soil	123.89 gm	123.89 gm
Water Content	26.79 %	25.15 %

	Initial Conditions
Volume of Ring, V	4.615in.^{3} = 75.64 cm^3
Volume of Solids, Vs	2.779in.^{3} = 45.54 cm^3
Volume of Voids, Vv	1.836in.^{3} = 30.10 cm^3
Initial Void Ratio, e_o	.661
Initial Saturation, S	106%

Water Content Trimmings	
Tin and Wet Sample	47.13 gm
Tin and Dry Sample	40.61 gm
Wt. Water	6.52 gm
Tin Wt.	15.60 gm
Dry Sample Wt.	25.01 gm
Water Content	26.07%

CONSOLIDATION TEST- Calculation Sheet * #7 *

Sample Diameter 2.800 in. Sample Area 6.158 in.^{2}

Initial Sample Height .7495 in. Initial void ratio .661

Initial dial reading 2.2000 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	2.21	3.95	2.17812	.02188	2.92E-2	.612	5.3E-7	1.1E-7
2	4.41	7.88	2.17100	.0290	3.87E-2	.596	8.5E-8	2.7E-8
3	8.82	15.76	2.15931	.0407	5.43E-2	.571	4.2E-8	1.9E-8
4	17.69	31.60	2.14448	.0555	7.41E-2	.538	1.9E-8	8.2E-9
5	35.33	63.11	2.12672	.0733	9.78E-2	.498	1.4E-8	8.1E-9
6	70.56	126.04	2.10528	.0947	1.26E-1	.451	7.1E-9	5.3E-9

Load #	Sq. Rt.of Time Method t_{90}	c_v	m_v	Log. of Time Method t_{50}	c_v	m_v
1	2.50	.0473	7.39E-3	2.82	.0097	7.39E-3
2	4.84	.0230	2.42E-3	3.60	.0072	2.42E-3
3	7.73	.0140	1.98E-3	4.00	.0063	1.98E-3
4	10.56	.0099	1.25E-3	5.65	.0043	1.25E-3
5	8.17	.0122	7.29E-4	3.22	.0072	7.29E-4
6	9.24	.0104	4.48E-4	2.95	.0077	4.48E-4

CONSOLIDATION TEST * #8 *

Project Field Site 1 Boring No. 1-4 Sample Depth 14'

Soil Description Gray silty-clay

Ring Diameter 2.800 in. Ring Height .7495 in Ring Weight 87.39 gm

	Before Test	After Test
Wt Wet Soil and Ring	246.26 gm	246.15 gm
Wt Dry Soil and Ring	212.95 gm	212.95 gm
Wt Water	33.31 gm	33.20 gm
Wt Dry Soil	125.56 gm	125.56 gm
Water Content	26.53 %	26.44 %

	Initial Conditions
Volume of Ring, V	4.615 $in.^3$ = 75.64 cm^3
Volume of Solids, Vs	2.816 $in.^3$ = 46.17 cm^3
Volume of Voids, Vv	1.799 $in.^3$ = 29.47 cm^3
Initial Void Ratio, e_o	.639
Initial Saturation, S	113%

Water Content Trimmings	
Tin and Wet Sample	52.05 gm
Tin and Dry Sample	44.60 gm
Wt. Water	7.45 gm
Tin Wt.	15.46 gm
Dry Sample Wt.	29.14 gm
Water Content	25.57%

CONSOLIDATION TEST- Calculation Sheet * #8 *

Sample Diameter 2.800 in. Sample Area 6.158 in.^{2}

Initial Sample Height .7495 in. Initial void ratio .639

Initial dial reading 2.2000 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	2.20	3.93	2.18509	.0149	1.99E-2	.606	6.6E-8	1.8E-7
2	4.41	7.88	2.17848	.0215	2.87E-2	.592	1.8E-7	4.9E-8
3	8.82	15.76	2.16929	.0307	4.10E-2	.572	5.6E-8	3.5E-8
4	17.55	31.35	2.15788	.0421	5.62E-2	.547	2.5E-8	1.9E-8
5	35.23	62.93	2.14338	.0566	7.55E-2	.515	1.7E-8	1.2E-8
6	70.53	125.98	2.12392	.0761	1.02E-1	.473	9.9E-9	7.9E-9
7	35.23	62.93	2.12862	.0714	9.52E-2			
8	8.82	15.76	2.14218	.0578	7.71E-2			
9	2.20	3.93	2.15531	.0447	5.96E-2			

Load #	Sq. Rt.of Time Method			Log. of Time Method		
	t_{90}	c_v	m_v	t_{50}	c_v	m_v
1	1.39	.085	5.06E-3	1.17	.0235	5.06E-3
2	2.19	.0519	2.23E-3	1.85	.0143	2.23E-3
3	4.75	.0235	1.56E-3	1.77	.0146	1.56E-3
4	6.40	.0169	9.75E-4	1.99	.0127	9.75E-4
5	5.66	.0184	6.13E-4	1.83	.0133	6.13E-4
6	6.40	.0154	4.20E-4	1.88	.0123	4.20E-4

CONSOLIDATION TEST * #9 *

Project Field Site 2 Boring No. 2-6 Sample Depth 17'

Soil Description Gray silty-clay

Ring Diameter 2.500 in. Ring Height 1.00 in Ring Weight 112.79 gm

	Before Test	After Test
Wt Wet Soil and Ring	287.50 gm	286.66 gm
Wt Dry Soil and Ring	259.22 gm	259.22 gm
Wt Water	28.28 gm	27.44 gm
Wt Dry Soil	146.43 gm	146.43 gm
Water Content	19.31 %	19.31 %

	Initial Conditions
Volume of Ring, V	4.909 in.^{3} = 80.44 cm^3
Volume of Solids, Vs	3.249 in.^{3} = 53.24 cm^3
Volume of Voids, Vv	1.660 in.^{3} = 27.20 cm^3
Initial Void Ratio, e_o	.511
Initial Saturation, S	102 %

Water Content Trimmings	
Tin and Wet Sample	57.55 gm
Tin and Dry Sample	50.50 gm
Wt. Water	7.05 gm
Tin Wt.	15.59 gm
Dry Sample Wt.	34.91 gm
Water Content	20.19%

CONSOLIDATION TEST- Calculation Sheet * #9 *

Sample Diameter 2.500 in. Sample Area 4.909 in.^{2}

Initial Sample Height 1.00 in. Initial void ratio .511

Initial dial reading 0.0000 *H = R_o - R

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	1.54	3.45	0.0090	.009	9.00E-3	.497	2.9E-7	1.3E-7
2	3.07	6.88	0.01435	.0144	1.44E-2	.489	1.7E-7	6.5E-8
3	6.14	13.76	0.02285	.0229	2.99E-2	.476	8.7E-8	4.8E-8
4	12.34	27.65	0.03431	.0343	3.43E-2	.459	5.9E-8	3.4E-8
5	24.77	55.50	0.05005	.0501	5.01E-2	.435	4.4E-8	2.1E-8
6	49.46	110.83	0.07321	.0732	7.32E-2	.400	2.3E-8	1.2E-8
7	24.77	55.50	0.07085	.0709	7.09E-2			
8	6.14	13.76	0.06188	.0619	6.19E-2			
9	1.54	3.45	0.05255	.0526	5.26E-2			

Load #	Sq. Rt. of Time Method			Log. of Time Method		
	t_{90}	c_v	m_v	t_{50}	c_v	m_v
1	2.92	.0723	2.61E-3	1.46	.0336	2.61E-3
2	2.96	.0701	1.56E-3	1.78	.0271	1.56E-3
3	4.49	.0456	1.24E-3	1.90	.0250	1.24E-3
4	4.32	.0465	8.25E-4	1.72	.0272	8.25E-4
5	3.88	.0504	5.67E-4	1.87	.0243	5.67E-4
6	5.20	.0360	4.18E-4	2.31	.0190	4.18E-4

CONSOLIDATION TEST * #10 *

Project Field Site 2 Boring No. 2-6 Sample Depth 19'

Soil Description Gray silty-clay

Ring Diameter 2.50 in. Ring Height 1.00 in. Ring Weight 113.59 gm

	Before Test	After Test
Wt Wet Soil and Ring	285.41 gm	284.95 gm
Wt Dry Soil and Ring	256.95 gm	256.95 gm
Wt Water	28.46 gm	28.00 gm
Wt Dry Soil	143.36 gm	143.36 gm
Water Content	19.85 %	19.53 %

	Initial Conditions
Volume of Ring, V	4.909 in.^{3} = 80.44 cm^3
Volume of Solids, Vs	3.181 in.^{3} = 52.13 cm^3
Volume of Voids, Vv	1.728 in.^{3} = 28.31 cm^3
Initial Void Ratio, e_o	.543
Initial Saturation, S	100%

Water Content Trimmings	
Tin and Wet Sample	64.06 gm
Tin and Dry Sample	56.38 gm
Wt. Water	7.68 gm
Tin Wt.	15.49 gm
Dry Sample Wt.	40.89 gm
Water Content	18.78 %

CONSOLIDATION TEST- Calculation Sheet * #10 *

Sample Diameter 2.50 in. Sample Area 4.909 $in.^2$

Initial Sample Height 1.00 in. Initial void ratio .543

Initial dial reading 0.0000 $*H = R_o - R$

Load #	Applied Load	Stress	Final Dial	* H	Strain	Void Ratio	k_S	k_L
1	1.54	3.45	.00882	.00882	8.82E-3	.529	4.8E-7	1.2E-7
2	3.07	6.08	.01588	.01588	1.59E-2	.518	2.3E-7	5.0E-8
3	6.14	13.76	.02777	.02777	2.78E-2	.500	1.9E-7	5.5E-8
4	12.31	27.58	.04397	.04397	4.40E-2	.475	9.3E-8	3.3E-8
5	24.73	55.41	.06558	.06558	6.56E-2	.442	6.8E-8	2.0E-8
6	49.58	111.10	.09225	.09225	9.23E-2	.401	1.5E-8	9.6E-9
7	24.73	55.41	.08908	.08908	8.91E-2			
8	6.14	13.76	.07832	.07832	7.83E-2			
9	1.54	3.45	.06742	.06742	6.74E-2			

Load #	Sq. Rt.of Time Method			Log. of Time Method		
	t_{90}	c_v	m_v	t_{50}	c_v	m_v
1	1.74	.121	2.56E-3	1.67	.0294	2.56E-3
2	2.85	.0727	2.06E-3	3.01	.0160	2.06E-3
3	2.82	.0722	1.73E-3	2.28	.0208	1.73E-3
4	3.80	.0522	1.17E-3	2.48	.0186	1.17E-3
5	3.35	.0571	7.76E-4	2.70	.0165	7.76E-4
6	8.82	.0205	4.79E-4	3.22	.0131	4.79E-4

APPENDIX V

TRIAXIAL CELL DATA USED TO DETERMINE THE LABORATORY HYDRAULIC CONDUCTIVITY OF SOIL SAMPLES FROM SITES 1 AND 2

Triaxial Cell Hydraulic Conductivity Data

Sample Identification: Boring 1-1, 9 feet
Sample Length (dl): 2.75 inches
Pore Water Pressure Difference: 3.0 psi.
Sample Diameter: 1.375 inches
Hydraulic Gradient (dh/dl): 30.2
Overburden Pressure (σ_o): 5.0 psi.
Confining Pressure (σ_3): 10.0 psi.

Elapsed Time (min.)	Burrette Reading (ml.)
0	5.60
535	5.71
1465	6.04
1980	6.21
2895	6.46

Calculated Hydraulic Conductivity: 1.8×10^{-8} cm/sec

Sample Identification: Boring 1-3, 12 feet
(fracture present in hand sample)
Sample Length (dl): 2.25 inches
Pore Water Pressure Difference: 4.0 psi.
Sample Diameter: 1.375 inches
Hydraulic Gradient (dh/dl): 49. psi.
Overburden Pressure (σ_o): 6.5 psi
Confining Pressure (σ_3): 7.00 psi.

Elapsed Time (min.)	Burrette Reading (ml.)
0	1.85
895	2.62
1495	3.11
2720	4.12
3690	4.95
4210	5.39
4965	6.04

Calculated Hydraulic Conductivity = 3.0×10^{-8} cm/sec

Sample Identification: Boring 1-4, 14 feet
Sample Length (dl): 2.75 inches
Pore Water Pressure Difference: 4.0 psi.
Sample Diameter: 1.375 inches
Hydraulic Gradient (dh/dl): 40. psi
Overburden Pressure (σ_o): 7.4 psi.
Confining Pressure (σ_3): 7.0 psi.

Elapsed Time (min.)	Burrette Reading (ml.)
0	1.80
505	2.00
1985	2.45
2850	2.70
4815	3.30
6440	3.81
7635	4.17
10155	4.86
11565	5.16

Calculated Hydraulic Conductivity = 1.3×10^{-8} cm/sec

Sample Identification: Boring 2-6, 16 feet
Sample Length (dl): 2.75 inches
Pore Water Pressure Difference: 4.00 psi.
Sample Diameter: 1.375 inches
Hydraulic Gradient (dh/dl): 40.
Overburden Pressure (σ_o): 8.0 psi.
Confining Pressure (σ_3): 8.0 psi.

Elapsed Time (min.)	Burrette Reading (ml.)
0	2.50
390	2.68
1830	3.44
2760	3.88
3165	4.08
4335	4.68

Calculated Hydraulic Conductivity = 2.2×10^{-8} cm/sec

Sample Identification: Boring 2-6, 19 feet
Sample Length (dl): 2.25 inches
Pore Water Pressure Difference: 4.00 psi.
Sample Diameter: 1.375 inches
Hydraulic Gradient (dh/dl): 49.
Overburden Pressure (σ_0): 9.5 psi.
Confining Pressure (σ_3): 9.5 psi.

Elapsed Time (min.)	Burrette Reading (ml.)
0	2.40
1380	3.05
2975	3.77
3710	4.34
4720	5.12

Calculated Hydraulic Conductivity = 2.8 x 10^{-8} cm/sec

Sample Identification: Boring 2-6, 21 feet
Sample Length (dl): 2.75 inches
Pore Water Pressure Difference: 4.00 psi.
Sample Diameter: 1.375 inches
Hydraulic Gradient (dh/dl): 40.
Overburden Pressure (σ_0): 10.6 psi.
Confining Pressure (σ_3): 10.5 psi.

Elapsed Time (min.)	Burrette Reading (ml.)
0	2.35
1115	2.86
2910	3.60
4100	4.20
7600	6.17

Calculated Hydraulic Conductivity = 2.0 x 10^{-8} cm/sec

APPENDIX VI

WELL ELEVATION AND SCREEN MIDPOINT DATA

Well Elevation Data

Site 1:

Well No.	Rod Interval (ft)	Screen Midpoint* (ft)
1-1	4.33	14.92
1-2	3.97	13.64
1-3	3.86	12.94
1-4	4.00	13.92
1-5	3.97	13.89
1-6	4.10	14.02

Site 2:

Well No.	Rod Interval (ft)	Screen Midpoint* (ft)
2-1	6.05	18.25
2-2	5.70	18.08
2-3	5.34	17.39
2-4	5.00	17.05
2-5	4.81	16.41
2-6	4.00	15.70
2-7	5.30	17.00
2-8	5.05	17.00
2-9	4.79	16.74
2-10	4.50	16.20
2-11	4.26	16.06
2-12	4.02	15.77
2-13	4.60	17.35
2-14	4.51	17.26

* Screen midpoints are relative to the ground elevation of well 1-1 or 2-1 as datum.

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