Gartner Lee Associates Limited

Consulting Engineering Geologists and Hydrogeologists

Toronto-Buttonville Airport ■ Markham, Ontario ■ L3P 3J9 ■ 416-477-8400

HYDROGEOLOGY REVIEW

HYDE PARK LANDFILL

PREPARED BY:

GARTNER LEE ASSOCIATES LIMITED
E. GRANT ANDERSON, P.ENG.
CONSULTING HYDROGEOLOGIST

MARCH 11, 1982.

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March 11, 1982.

Ms. Toby Vigod,
Barrister and Solicitor,
Ms. Barbara Morrison,
Attorney at Law,
c/o Jay, Klaif and Morrison,
Attorneys and Counselors at Law,
1032 Ellicott Square,
Buffalo, New York,
14203.

Dear Ms. Vigod and Ms. Morrison:

Re: Hyde Park Landfill, Hydrogeological Review

Enclosed is my report prepared subsequent to thoroughly reviewing the most recent documents and other pertinent information on the above noted matter.

My review and analysis of all pertinent data indicates that the ground water model simulation recently submitted to the U.S. District Court for the Western District of New York by the United States Department of Justice is invalid and any conclusions arising from it are extremely misleading.

Nothing contained in the documents reviewed alters my position that the proposed settlement agreement is based on a fundamental misunderstanding, by the Parties, of the ground water flow system. Details of my findings are contained in the body of the report.

Yours very truly,

GARTNER LEE ASSOCIATES LIMITED

Grant Anderson

E.G. Anderson, P.Eng.,
Consulting Hydrogeologist.

EGA:jcm

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HYDROGEOLOGY REVIEW

1.0 BACKGROUND:

At Amici's request, I have reviewed a number of documents recently submitted to the U.S. District Court for the Western District of New York by the United States Department of Justice in the case of United States vs. Hooker Chemical and Plastics Corp. et al (Hyde Park).

The documents which I have specifically evaluated are as follows:

1. letter dated January 18, 1982 to the Hon. John T. Curtin from Michael Elder;

2. letter dated January 18, 1982 from Richard Johnston to Charles Morgan (with unsigned three page document entitled "Simplified Explanation of the Ground-Water Simulation Described in the Maslia/Johnston 1982 Report on Hyde Park Landfill");

3. Johnston, Richard H. and Morris L. Maslia, Simulation of Ground Water Flow in the Vicinity of Hyde Park Landfill, Niagara Falls, New York, U.S. Geological Survey, Open file Report 82-159;

4. West Coast Technical Service, Inc., Final Report (four water samples labelled B0677, B0777, B0780, and B0781, and five soil samples labelled B0749, B0750, B0776, B0778, and B0779); and

5. letter dated February 19, 1982 to the Hon. John T. Curtin from Michael Elder (with attachments A and B).

2.0 PURPOSE:

In my evaluation of these documents, I have addressed the following:

1. Does the Maslia/Johnston Model simulate actual field conditions in the area of the Hyde Park Landfill?

   a. Was the model selected an appropriate one for determination of flow direction and velocity?

   b. Were the substantive assumptions used in preparation of the model correct?

   c. Does the model input data correspond to factual data obtained by the Parties and/or Amici and presented in judicial proceedings held in autumn 1981 on the proposed Hyde Park Agreement?

2. Does the Model reveal any new information or resolve any issue in contention between Parties and Amici (as set forth in Amici's Post Hearing Submission, dated October 14, 1981)?

3. Does the model as prepared achieve its stated purpose of "determining probable directions and flow velocities of ground water moving away from the landfill to nearby areas of discharge" (Maslia/Johnston Report, at page 3)?

4. Does the Maslia/Johnston Report or any other documents listed above, in any way alter my conclusions with respect to the material inadequacy of the Hyde Park Agreement as it is proposed for judicial approval?

3.0 SUMMARY OF REVIEW:

In summary, subsequent to a thorough review of the documents referenced above, along with relevant data and transcripts from the 1981 proceedings in this matter, I have concluded that the model simulation prepared by the USGS is invalid and cannot be utilized to verify actual conditions present at Hyde Park. Details are as follows:

1. The Model does not simulate actual field conditions as previously measured by the parties at Hyde Park Landfill. Contrary to the Maslia/Johnston Report, the model was never calibrated.

2. While the modelling technique selected is theoretically capable of achieving the study's purposes, it has not been successful in doing so because of the material factual errors inputted to the system.

3. Many assumptions plugged into the model were in direct contradiction to the field data as measured and presented to the Court. The most significant erroneous assumptions and/or data plugged into the model which destroy its validity are:

   a. zero infiltration of precipitation occurs through the landfill's clay cap;

   b. extremely high horizontal hydraulic conductivity vs. very low vertical conductivity in the lower Lockport Dolomite and Rochester Shale which contributes to ground water flowing from the entire thickness of the Lockport Dolomite formation at the Gorge face;

   c. no ground water flows vertically downward through the bottom of the Rochester Shale.

   These three assumptions completely bias the model to depict ground water flow conditions that are predominantly lateral through the Lockport Dolomite. Yet Johnston, in his Report summary (appended to his letter dated January 18, 1982 to Charles Morgan) clearly disclaims the likelihood of predominantly lateral flow. (Item 3 at last paragraph).

4. The Model yields no new information beyond what was available to the Court at the conclusion of the hearing on October 16, 1981, and does not provide resolution of any issues in contention between the Parties and Amici.

   At bottom line, the three major assumptions used in the

Model contradict actual field conditions at Hyde Park Landfill. Therefore, the Model is useless and misleading. Likewise, all other calculations and conclusions derived from an interpretation of the modelling results are invalid.

4.0 THE GROUND WATER MODEL:

In order to understand the results of the Maslia/Johnston computer simulation, the assumptions utilized in the model which ultimately control its output must be analysed for accuracy and relevance. For the model to be accurate, the assumptions used must replicate actual field conditions. After the model is initially set up, it must be calibrated to reflect known field data. If the simulation is in conflict with real conditions, the model is useless.

This model merely calculates and plots the hydraulic head within a region defined by the specified boundaries. The model must be told the physical location of all boundaries, the boundary condition at each node on the boundary, and the material properties (horizontal and vertical permeability) of each finite element within the region.

The combination of material properties and boundary conditions determines the resulting hydraulic head distribution.

Thus by varying these parameters, we can change the hydraulic heads calculated by the model. The modeller varies these parameters until the hydraulic head distribution calculated by the model matches the observed heads in the real system as closely as possible. The modelling results will only be as accurate as the data inputted.

A hand calculation utilizes the factual field data to determine ground water flow direction, velocities, and quantities of flow, whereas the computer model first must replicate the field data and then, after it achieves replication of field data (calibration) it can calculate the velocity, direction, and quantity of flow. If the data simulated by the model does not replicate field measurements, that is, if calibration is not achieved, the model is rendered useless and any conclusions drawn from it are misleading and technically incorrect.

One of the key values of a model, beyond that of manual calculations, is that once it is properly calibrated a model can very quickly furnish information at numerous points within the modelled boundaries where no field measurements have been taken.

Whether a manual calculation is used to calculate flow direction or velocity at one particular point, or a model is used to simultaneously calculate flow direction or velocity within a region, the end result will only be as accurate as the data inputted into the model or used in the manual calculation.

The following three most serious erroneous assumptions used in the model which bias its results, either through failure to replicate or through direct contradiction of factual conditions, are discussed below:

4.1 INFILTRATION OF PRECIPITATION THROUGH THE CLAY CAP:

The model assumes zero infiltration of precipitation through the clay cap into the chemical waste.

It was acknowledged during the 1981 proceedings that the clay cap is in fact permeable and does allow infiltration into the chemical waste. In addition, actual water levels below the cap as measured in 1981, approximately three years subsequent to final placement of the clay cap, and operation of the toe drain, indicate six inches of infiltration through the cap into the 15 acre landfill. Also, the infiltration data shows the accumulation of a significant ground water mound in the chemical waste (cross examination of Frank Rovers.)

The ability and amount of precipitation to penetrate the clay cap are of fundamental importance in anticipating the extent of exfiltration of leachate through the bottom of the landfill. It was agreed in the proceedings, by experts for all Parties and Amici, that the vertical direction of ground water flow is downward. The existence and extent of precipitation infiltration materially determines the vertical gradient (direction) of ground water flow below the chemicals and affects flow direction throughout the remainder of the system.

The ground water mound within Hyde Park Landfill would not exist if there were no infiltration through the cap. In such a case, a valley or depression in the ground water table would occur, preventing vertical flow downwards.

The ground water mound which properly represents actual field conditions as measured by the Parties is properly depicted in Figure 2 of the Maslia/Johnston Report (at page 6). Yet, the model simulation, utilizing the theoretical and incorrect assumption of zero flow through the clay cap, shows the opposite configuration, a ground water valley below the chemical wastes as depicted in Figure 5 of the Maslia/Johnston Report (at page 15). That water table configuration of a valley resulted in the computer plotting ground water flowing laterally through the rock into the chemicals (Maslia/Johnston Figure 6, page 16), which is in complete contradiction to the actual field situation, as understood by the parties and the Court. Therefore, the model, because of the incorrect assumption regarding infiltration has biased the actual vertical flow system to an incorrect lateral flow system.

4.2 GROUND WATER FLOW AT THE GORGE FACE - HYDRAULIC CONDUCTIVITIES:

The Maslia/Johnston report depicts that ground water discharges from the entire thickness of the Lockport Dolomite at the face of the Gorge.

The ratio of horizontal to vertical hydraulic conductivity in the Lower Lockport Dolomite and Rochester Shale is set by Maslia/Johnston at 1000:1, (i.e. horizontal hydraulic conductivity is 1000 times greater than vertical hydraulic conductivity). This greatly inhibits downward flow through these layers, and incorrectly promotes lateral flow. This assumption contributes to the elevation of the water table within the glacial till near the Gorge face, and the resulting discharge of ground water from the entire thickness of the Lockport Dolomite. Also, the assumption of no flow through the bottom of the Rochester Shale further contributes to the above result.

If this water table configuration held any validity whatsoever, ground water would cascade down the Gorge face from the entire thickness of the Lockport Dolomite formation. Yet it is clear, by the testimony of experts for the Parties and for Amici that the Gorge face within the dolomite formation is, with rare exception, dry and devoid of seepage and/or springs (field data, as determined from visual observation at the Gorge face and corroborated by similar observations of Morgan, Davis, Mason, and Rovers). In addition, Johnston's 1964 report confirmed the same visual absence of discharge through the thickness of the Dolomite. Therefore, there was clearly no attempt to calibrate the model with actual field conditions at the Gorge.

In fact, the ground water table is located at or below the bottom of the Lockport Dolomite. The ground water which is not discharging at the Gorge face through the thickness of the Lockport Dolomite, is in fact flowing vertically downward through the Rochester Shale before reaching the Gorge face.

4.3 GROUND WATER FLOW THROUGH THE BOTTOM OF THE ROCHESTER SHALE:

Maslia/Johnston plugged into the model the assumption that no flow would occur through the bottom of the Rochester Shale. (FN-1). This assumption is clearly erroneous and contradicts field evidence of flow through the Shale.

As described in testimony, the bedrock layers below the Rochester shale are more permeable than the Rochester Shale. By placing a no-flow boundary at the base of the Rochester Shale, Maslia/Johnston have eliminated the possibility that flow may occur down through the Rochester Shale to the Queenston Shale to some 300 feet below the site. Any possible effect these more permeable rock layers below the Rochester Shale may have is not considered by this model. It is obvious from visual observation that there is no discharge of ground water from the Lockport Dolomite at the Gorge face and thus the ground water must be moving down through the Rochester Shale.

Since the Johnston/Maslia Report acknowledges that the Rochester Shale is of lower permeability than the Lockport

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FN-1 The Maslia/Johnston Report (last sentence of third paragraph at page 10) states that "a zero flux value was (also) assigned to boundaries CD, KB, and GH". Also see at page 11 - where Figure 4 reveals that boundary KB is the bottom of the Rochester Shale. When the computer allows no flow through the bottom of the Shale, the ground water has no choice but to move laterally.

Dolomite (overlying the Rochester Shale) but also is of lower permeability than the interlayered sandstone, limestone and shale underlying the Rochester Shale, all ground water moving into and through the Rochester Shale will continue to move vertically downward under field conditions and outside the constraint of the model's artificial no-flow boundary placed at the bottom of the Rochester Shale.

It is important not to be misled by the conclusion that 80% of ground water seepage will occur along the Gorge face through the Lockport Dolomite since the significance of that conclusion is in effect refuted by its authors in a specific disclaimer contained in Johnston's simplified explanation of his ground water simulation as appended to a letter to Charles Morgan dated January 18, 1982. (At Item 3, last paragraph).

There Johnston materially qualifies the 80% seepage figure with the statement that "the lack of seepage or springs at the Gorge face" (as mentioned in testimony) is due to vertical movement of water via "open" joints or within slope rubble. The computer simulation is therefore valid only to the point where the ground water intercepts either "open vertical joints or rubble".

It is clear that Johnston/Maslia are suggesting that if the Rochester Shale was not fractured and did not allow any flow vertically downwards, then 80% of the flow would discharge from the Lockport Dolomite above the Rochester Shale at the Gorge. The point here is that whatever volume of ground water could otherwise likely flow laterally and discharge at the Gorge face is at some point between the landfill and the Gorge face moving vertically downward into and below the Rochester Shale (as described in my testimony and as depicted on Figure 9, Position of Amici, Appendix 1 - Amici exhibit 97).

It is critical to note that the theoretical 80% discharge figure is arrived at, in part, because the computer was prohibited from allowing any water to flow vertically downward through the bottom of the Rochester Shale (Maslia/Johnston Report at page 10). Because in effect a large bottom plug is simulated under the Rochester Shale, a resultant bias of greater velocity and quantity of lateral ground water flow in the Lockport Dolomite occurs.

Therefore, the 80% figure results from inputting an assumption which directly contradicts actual hydrogeologic flow conditions. The chemical testing results presented by Amici during the 1981 proceedings indicate movement of contaminated ground water downward through and below the Shale into the whirlpool sandstone formation approximately 300 feet below ground surface. Dr. Hallet has concluded that nothing in the EPA's chemical testing at the Gorge face and control site provide any logical reason to alter the previous conclusion of Amici that chemicals from Hyde Park have already migrated vertically through the Rochester Shale, 300± feet down to the Niagara River. The recent EPA analysis only serves to corroborate the previous findings of Amici.

5.0 THE GROUND WATER MODEL CALIBRATION:

Maslia/Johnston did not use the fact that the water table does not intersect the face of the Gorge as evidenced by visual observations, to calibrate the water table in the model. In addition, water table data from observation well 9 clearly depicts a water table mound and not a water table valley below the chemicals as depicted (Figure 5, page 15) by the model.

Even more blatant in illustrating just how poorly this model is calibrated are the discrepancies between the actual field water level (head) measurements and the lines identified as equipotential lines on Figure 5, page 15 of Maslia/Johnston. ((FN-2). When the actual field water level measurements are plotted onto figure 5 (see Appendix 2) it is immediately obvious that calibration was never achieved.

FN-2 By definition, the water level in a piezometer which is drilled to an equipotential line as depicted on Figure 5 (Maslia/Johnston Report at page 15), will rise to an elevation equivalent to the potential marked on that equipotential line.

A determination of the adequacy of the model's calibration can be made by plotting the water level field data. The grossly inadequate calibration for this model can be exemplified by reference to OW-18 as shown on Figure 5. The piezometer tip is located at an elevation of 511 feet ASL at the bottom of the borehole. The water level is simulated at an elevation of about 582 feet ASL (located between equipotential lines 580 and 585, Figure 5). The field data, in fact, indicates a water level of 549± feet ASL. This is approximately 33 feet lower than the computer would indicate.

Similarly, if OW-30 is superimposed onto Figure 5, (as shown in Appendix 2) it is evident

FN-2 (continued)

that the piezometer tip is at an elevation of 539 feet above sea level and that the water level should be at 589 feet above sea level according to the computer simulation. In fact, the field data shows the water level to be at elevation 562 feet above sea level, approximately 27 feet lower than would be indicated by the computer.

These examples and others shown on Figure 5 in Appendix 2 reveal that the model is not calibrated. Perhaps the most serious consequence of this major error is the model's bias as an analysis of lateral flow. The vertical ground water gradient is very steep downward below the site, as evidenced at the hearing and by the actual field data, which is contrary to the pattern reflected by the computer simulation.

As an example, OW 18-80 is incorrectly simulated at elevation 582 feet when it should be at elevation 549 feet. OW 30-80 was ignored by Maslia/Johnston, yet could have been used along with OW 18-80 and others to achieve proper calibration of the model.

In addition to the failure of Maslia/Johnston to calibrate the model so as to achieve replication of field conditions as discussed above, the report itself disclaims even an attempt of calibration except within the till and the upper 15 feet of the Lockport Dolomite (page 14 of Maslia/Johnston, item 3).

There was no attempt to calibrate for flow below the top 15 feet of the Lockport Dolomite. Therefore, even if the model had any validity whatsoever, it would have only been applicable to the till and the upper 15 feet of the Lockport Dolomite.

However, the model result even for the till and upper dolomite has been materially biased by use of the assumption that there would be zero infiltration through the cap.

The purported calibration in those strata is only claimed to within a 5 foot accuracy of actual field measurements. The margin of error allowed is in fact 1/3 of the thickness of the unit claimed to have been calibrated, which is totally unacceptable as a replication of field conditions.

This so-called limited replication claimed for the upper layers (where ground water flow behaviour is not in contention between the Parties) is, in fact, no replication whatsoever and the 5 foot figure is in actuality a measure of the discrepancy between the model and field data within the upper 15 feet of the Lockport Dolomite. The erroneous plotting of a water table valley rather than the actual field condition of a water table mound within the landfill (as testified to by Frank Rovers and substantiated by the water elevation monitoring data) may account for this discrepancy in part.

The Maslia/Johnston report maintains that computer calibration was achieved by comparing the hydraulic heads, as plotted by the model, to a dramatically different water table configuration than actually exists at Hyde Park Landfill. They compare the simulated heads (located in this hypothetical and non-existent water table valley plotted below the chemical waste) to the actual hydraulic heads measured in the field (under the influence of the existing water table mound. Therefore, the claim that 'calibration' is within five feet in the upper layers is without merit.

6.0 CONCLUSIONS:

After review of the Maslia/Johnston documents, the chemical testing results, Hyde Park data and after discussion with my staff modeller, chemists, including Dr. Douglas Hallett, analytical environmental chemist at Environment Canada, and Richard Jackson, hydrogeologist with Environment Canada, I have reached the following conclusions:

1. The model as it was constructed is invalid and as such any conclusions drawn from it are misleading. Further it fails to simulate field conditions at the area of Hyde Park Landfill and as such is useless.

2. All material issues in contention were either assumed out of the model, or the results obtained were biased by the assumptions which were inputted.

3. The time for water to move from the landfill to the Gorge through the Lockport Dolomite reported by Maslia/Johnston of 5 to 7 years is biased by assuming 10 feet of clay exists below the chemicals on top of the Lockport Dolomite. In actual fact, chemicals are in direct contact with the dolomite and utilizing Maslia/Johnston's velocity ranges of 1.5 feet per day (548 feet/year) to 4.8 feet per day (1750 feet/year), dissolved chemicals would reach the Gorge in 1 to 3½ years. My original evidence was that chemicals could reach the Gorge face in less than a year. The material errors in the model previously discussed account for the discrepancy in travel time.

4. Johnston's evidence that the Rochester Shale is nearly impermeable on the basis of an inability to pump water from wells does not constitute a basis for assuming no ground water and thus chemicals will flow vertically through the Rochester Shale. Wells used for domestic

purposes require high hydraulic conductivity rock or soil in order to provide almost instant replenishment of water that is pumped from the well. In fact, ground water does replenish wells in shale but not fast enough for domestic water supply purposes. Although the quantities of water moving to a conventional 6 inch diameter well may be small and inadequate for domestic supply purposes, when one considers the more regional flow of water vertically down through the Rochester Shale over an area of 15 acres (area of chemical wastes) or more the quantity of flow is large and extremely significant to the migration of highly toxic chemicals.

I cannot emphasize strongly enough that chemicals will move through the Rochester Shale. The importance of this movement must not be undermined by using misleading information such as the assumption of impermeability based upon inability to pump significant quantities of water from a domestic well. As I previously testified, Gartner Lee Associates Limited has tested this Rochester Shale formation by carrying out in-situ hydraulic conductivity tests across the River from the Hyde Park Landfill near the Welland Canal approximately 4000 feet back from the top of the Niagara Escarpment. We are the only Company to have tested this shale to my knowledge. These tests clearly show that ground water movement does occur through the Rochester Shale.

As an analogous situation to illustrate vertical flow through fractured (otherwise virtually impermeable) material does occur, I have enclosed a paper in Appendix 3 which discusses this phenomenon. Although this paper deals with slowly permeable clays, the hydraulic conductivities of the clay are less than those assumed by Maslia/Johnston for the Rochester Shale. This document illustrates that,

a) vertical fractures will allow considerable flow of ground water,

b) technologies to test for ground water flow through these vertical fractures are readily available and have been applied to solve practical problems such as determining the amount of chemicals flowing through the Rochester Shale at Hyde Park. (Pages 36 to 39 in Appendix 3. See also, Anderson recommendations to the Court).

5. This hydrogeological review, when coupled with the chemical analysis which shows the presence of indicator chemicals from the Hyde Park Landfill, clearly substantiates that the Rochester Shale does not form a lower boundary for chemical movement.

6. The model fails in its purpose as it does not replicate or represent those field conditions not in contention by the Parties and Amici. As a result, nothing contained in the documents reviewed alters my position that the proposed settlement agreement is based on a fundamental misunderstanding of the ground water flow system by the Parties. As such, the studies required

in the agreement are incapable of pinpointing the migration pathways of chemicals escaping from beneath the Hyde Park Landfill. Consequently, the ill-conceived studies provided for in the agreement will preclude the formulation of a proper remedial strategy at this site. The agreement as it is constituted will continue to allow the contents of Hyde Park Landfill to slowly and continuously leak through the underlying layers of rock into the Niagara River.

Respectfully submitted,

GARTNER LEE ASSOCIATES LIMITED

Grant Anderson

E. Grant Anderson, P.Eng.
Consulting Hydrogeologist.

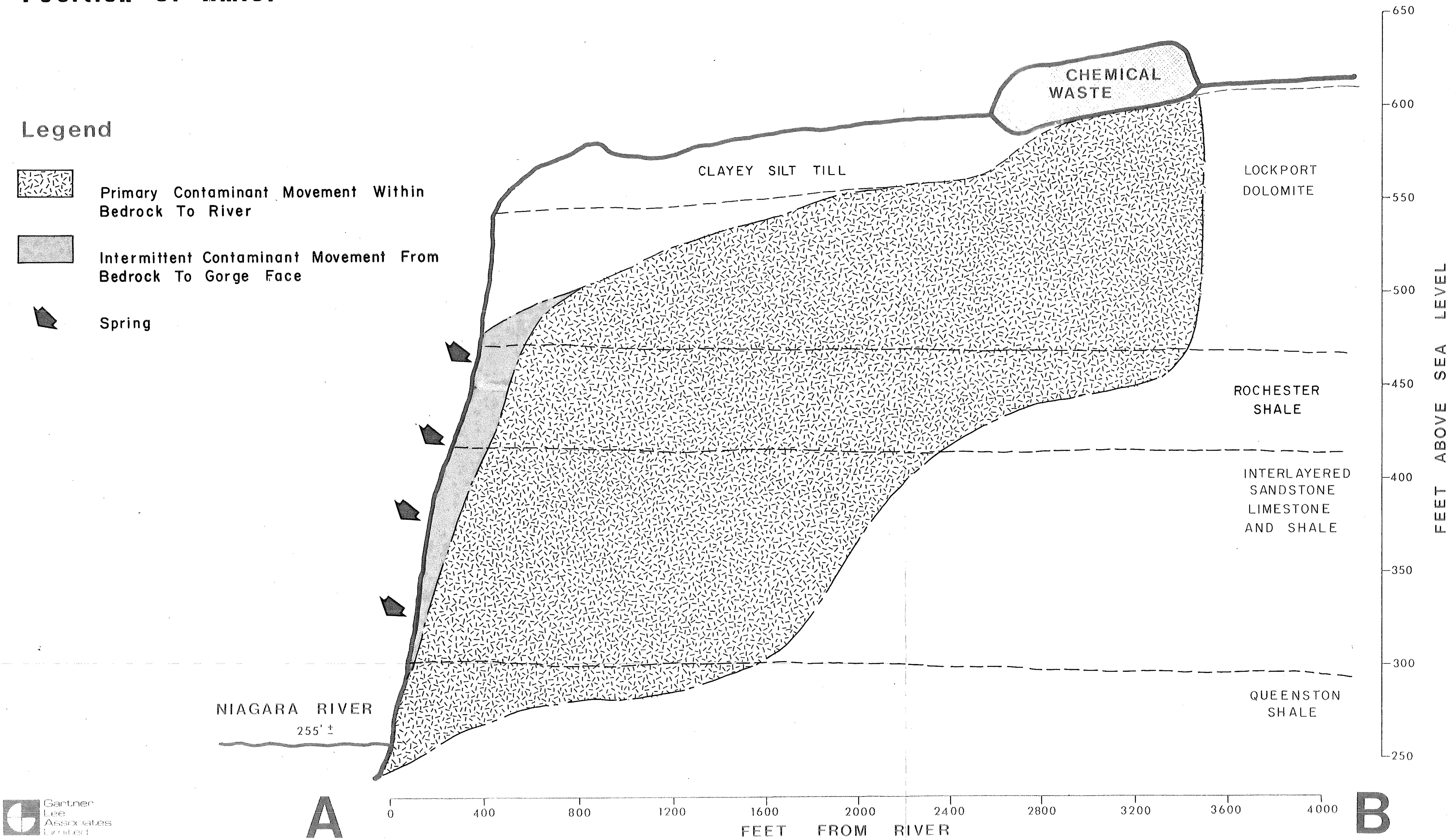
APPENDIX 1

Position of Amici, Figure 9

HYDE PARK ~ Bloody Run Area

FIGURE 9

Position Of Amici



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APPENDIX 2

FIGURE 5 - MASLIA/JOHNSTON MODIFIED TO SHOW DISCREPANCIES IN SIMULATED PIEZOMETRIC HEADS AND ACTUAL FIELD MEASUREMENTS

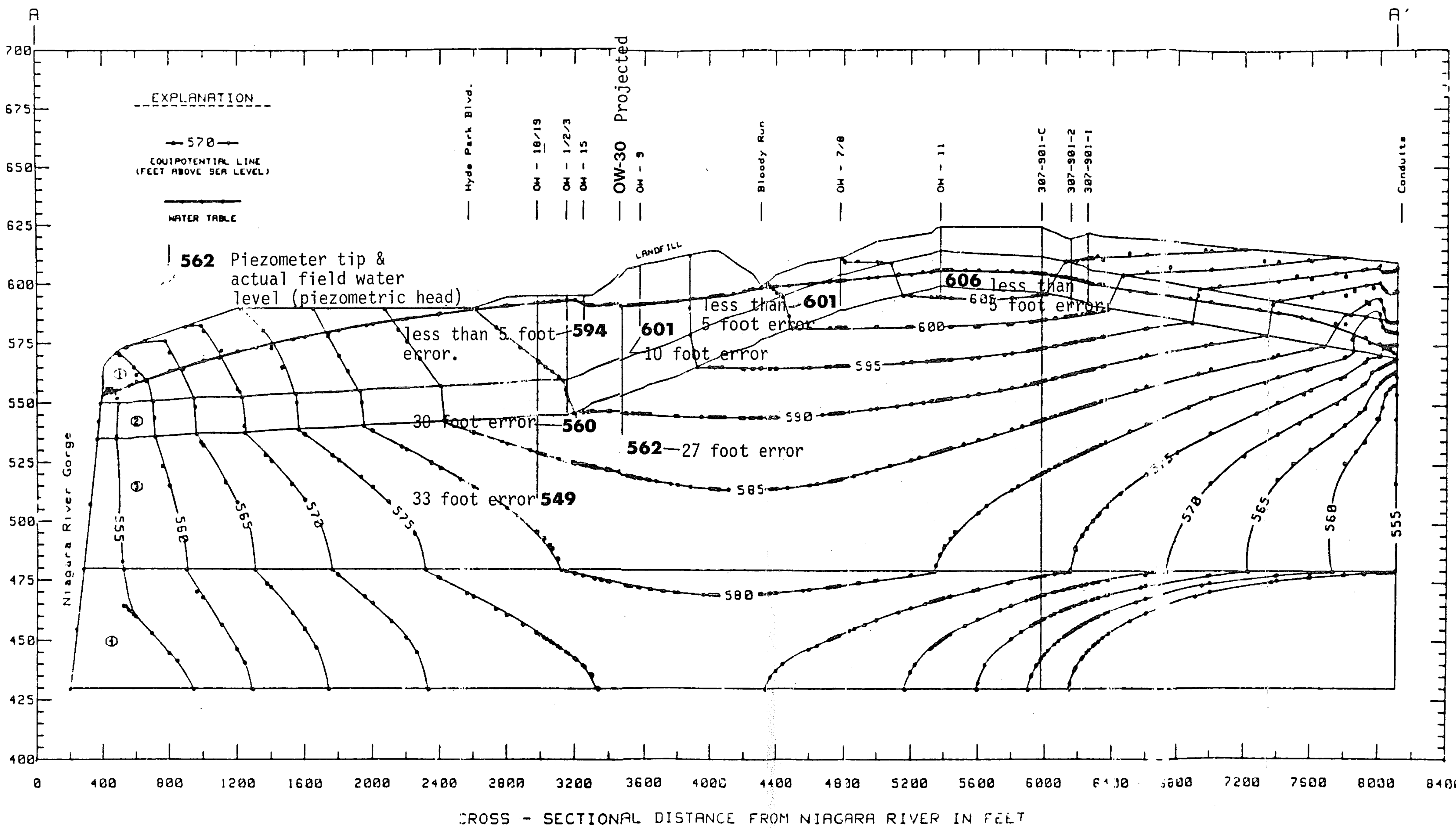

Figure 5. -- Simulated water table position and head distribution

Note: 1. Large black letters (i.e. **549**) indicate actual observation well water levels measured during August 1980.

2. See FN2 on page 13 of this report for an explanation of how to interpret this data.

APPENDIX 3

HYDROLOGIC CHARACTERISTICS AND RESPONSE OF FRACTURED TILL AND CLAY CONFINING A SHALLOW AQUIFER.

BY

G.E. GRISAK AND J.A. CHERRY

ILLUSTRATES:

- vertical flow through fractures
- techniques to assess vertical flow

Hydrologic Characteristics and Response of Fractured Till and Clay Confining a Shallow Aquifer

G. E. GRISAK[1] AND J. A. CHERRY

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Fractures in glacial till and glaciolacustrine clay were observed in excavations up to 20 ft (6.1 m) in depth and in drill cores at the Whiteshell Nuclear Research Establishment (WNRE) in southeastern Manitoba. The fractures are characteristically coated with carbonate and oxide precipitates, which indicate groundwater movement through the fractures. The fractures impart an effective bulk hydraulic conductivity to the clay–loam till and lacustrine clay, as evidenced by tritium tracer experiments and piezometer responses in the till and clay to pumping of an underlying sandy aquifer.

The intergranular hydraulic conductivity of clay–loam till and glaciolacustrine clay in the Interior Plains, as determined from laboratory consolidation test data, is in the range of 2×10^{-10} to 9×10^{-11} ft s^{-1} (6×10^{-9} to 2.7×10^{-9} cm s^{-1}). The bulk hydraulic conductivity of the fractured clay–loam till at WNRE, as determined from finite-element mathematical modeling, is about 6×10^{-9} ft s^{-1} (1.8×10^{-7} cm s^{-1}). The model value represents the effective hydraulic conductivity imparted to the till by the fractures.

Seven pumping tests, ranging in duration from 8.75 to 120 h were conducted on the sandy aquifer and drawdown data in the aquifer were analyzed to obtain the hydraulic conductivity and storativity of the aquifer.

A 32 day pumping test on the aquifer showed that many of the piezometers in the till and clay respond quickly and strongly to the aquifer drawdown, while others show no noticeable response. The responding piezometers intersect open fractures whereas the others do not. Analysis of the piezometer drawdowns during the long-term pumping test using the Neuman and Witherspoon 'ratio' method indicates that the rapid piezometer drawdowns in the confining layers can be accounted for by assigning specific storativity values in the range of 1×10^{-5} to 5×10^{-6} ft^{-1} (3×10^{-5} to 1.5×10^{-5} m^{-1}) to the clay–loam till and lacustrine clay. These values are typical of fractured rock. If intergranular specific storage values are used, the calculated piezometer drawdowns are very small or negligible.

Des fractures dans la moraine et l'argile glacio-lacustre ont été observées dans des excavations d'une profondeur maximum de 20 pi. et dans des échantillons prélevés en forages à la station de recherche nucléaire de Whiteshell (WNRE) dans le sud est du Manitoba. Les fractures, de façon caractéristique, sont enduites de dépôts d'oxydes et de carbonates, indicatifs de mouvements d'eau dans les fractures. Les fractures produisent une conductivité hydraulique globale effective dans la moraine argileuse et dans l'argile lacustre, mise en évidence par des essais avec des traceurs au tritium et par la réponse de piézomètres situés dans la moraine et l'argile, au pompage dans l'aquifère sableux sous-jacent.

La conductivité hydraulique intergranulaire de la moraine et de l'argile lacustre, déterminée par essais de consolidation au laboratoire, est de l'ordre de 2×10^{-10} à 9×10^{-11} pi./s (6×10^{-9} à 2.7×10^{-9} cm/s). La conductivité hydraulique totale de la moraine fracturée à la WNRE, déterminée à partir d'un modèle à éléments finis, est d'environ 6×10^{-9} pi./s (1.8×10^{-7} cm/s). Cette valeur représente la conductivité effective produite dans la moraine par les fractures.

Sept essais de pompage, d'une durée variant de 8.75 à 120 h ont été réalisés dans l'aquifère sableux et les mesures de rabattement de la nappe ont été analysées pour déterminer la conductivité hydraulique et la capacité d'emmagasinage de l'aquifère.

Un essai de pompage dans l'aquifère, d'une durée de 32 jours, a montré que de nombreux piézomètres dans la moraine et l'argile répondaient rapidement et de façon importante au rabattement de nappe dans l'aquifère, alors que d'autres n'avaient aucune réaction notable. Les piézomètres qui réagissent traversent des fractures ouvertes alors que les autres ne le font pas. L'analyse des variations piézomètriques durant l'essai à long terme à l'aide de la méthode des

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Can. Geotech. J., 12, 23 (1975)

"rapports" de Neuman et Witherspoon montre que les baisses rapides de niveaux piézométriques dans les couches étanches peut être expliquée en affectant des valeurs de l'ordre de 1×10^{-5} à 5×10^{-6} pi.$^{-1}$ (3×10^{-5} à 1.5×10^{-5} m^{-1}) à l'emmagasinage spécifique de la moraine et de l'argile. Ces valeurs sont typiques d'un roc fracturé. Si on utilise les valeurs d'emmagasinage spécifique intergranulaire, les variations piézométriques calculées sont très petites ou négligeables.

[Traduit par la Revue]

Introduction

The presence of ubiquitous fractures in glacial till and in glaciolacustrine clay in the Interior Plains Region of North America has long been recognized by geologists as one of the characteristic physical properties of these deposits in this region. More recently several investigators (Gilliland 1965; Freeze 1969; Render 1970; Vonhof 1970; Cherry *et al.* 1971; Sloan 1972) have suggested on the basis of indirect evidence that fractures have an important effect on the bulk hydraulic conductivity of till and clay.

The intergranular conductivity of clay–loam till and glaciolacustrine clay is very small. If there are open and interconnected fractures, the bulk hydraulic conductivity of these deposits will be much larger than the intergranular conductivity. Since glacial till and lacustrine clay are common types of Quaternary deposits in the Interior Plains Region, knowledge of the effects of fractures on the hydraulic properties of these materials is an essential prerequisite to understanding the behavior of hydrogeologic systems in the region.

The purpose of this paper is to show that the fractures in clay–loam till and lacustrine clay in an area in southeastern Manitoba are the dominant features controlling both the hydraulic conductivity and storativity of these deposits and that the fracture conductivity network is a hydraulic continuum across stratigraphic units. The till and clay are confining beds overlying a shallow sand aquifer of Pleistocene origin. They are lithologically and genetically similar to much of the till and lacustrine clay in the eastern part of the Interior Plains Region. The hydrologic character and response of the fractures were studied using borehole samples, observations in excavations, mapping of the natural groundwater flow patterns, mathematical flow-system simulations, short- and long-term pumping tests, and analysis of laboratory consolidation test data.

Field investigations were conducted as part of a hydrogeologic study of a shallow radioactive waste management site at the Whiteshell Nuclear Research Establishment (Cherry *et al.* 1973). The site is located on the western edge of the Canadian Precambrian Shield about 75 miles (120 km) northeast of Winnipeg (Fig. 1). The Precambrian granite bedrock of the area is overlain by 30–80 ft (10–26 m) of Pleistocene sediments, mainly lacustrine clay, clay–loam till, and sandy drift. The sandy drift comprises a relatively continuous confined aquifer that lies directly on top of the granite bedrock. The confining layers are principally clay–loam till and lacustrine clay. The water table is normally within a few feet of ground surface.

The climate is midcontinental, characterized by extreme temperature variations. Recorded temperature extremes over the study period were a high of 93 °F (34 °C), a low of −43 °F (−42 °C), with a mean of about 34 °F (1 °C). The annual precipitation varied from 16 to 26 in. (43 to 70 cm), with the largest portion occurring between May and September. During the winter the ground usually freezes to depths of 1–3 ft (0.3–1 m).

Geology

Field Methods

The gross physical character and stratigraphic relations of the Pleistocene deposits at WNRE were defined by 76 test holes drilled by vehicle-mounted hydraulic drills equipped with solid and hollow-stem augers. The locations of the test holes are shown in Fig. 1. The holes were drilled to depths at which further penetration was not possible. Auger flight samples were collected from all holes. Shelby-tube samples of the till and clay beds were obtained from some of the holes.

Eight additional holes penetrating to the bedrock surface were drilled using a cable-tool and split-spoon sampler. These holes provided more detailed stratigraphic and lithologic data from sandy beds below the till. These beds could not be adequately sampled using the auger drills. Seven of the cable-tool holes were

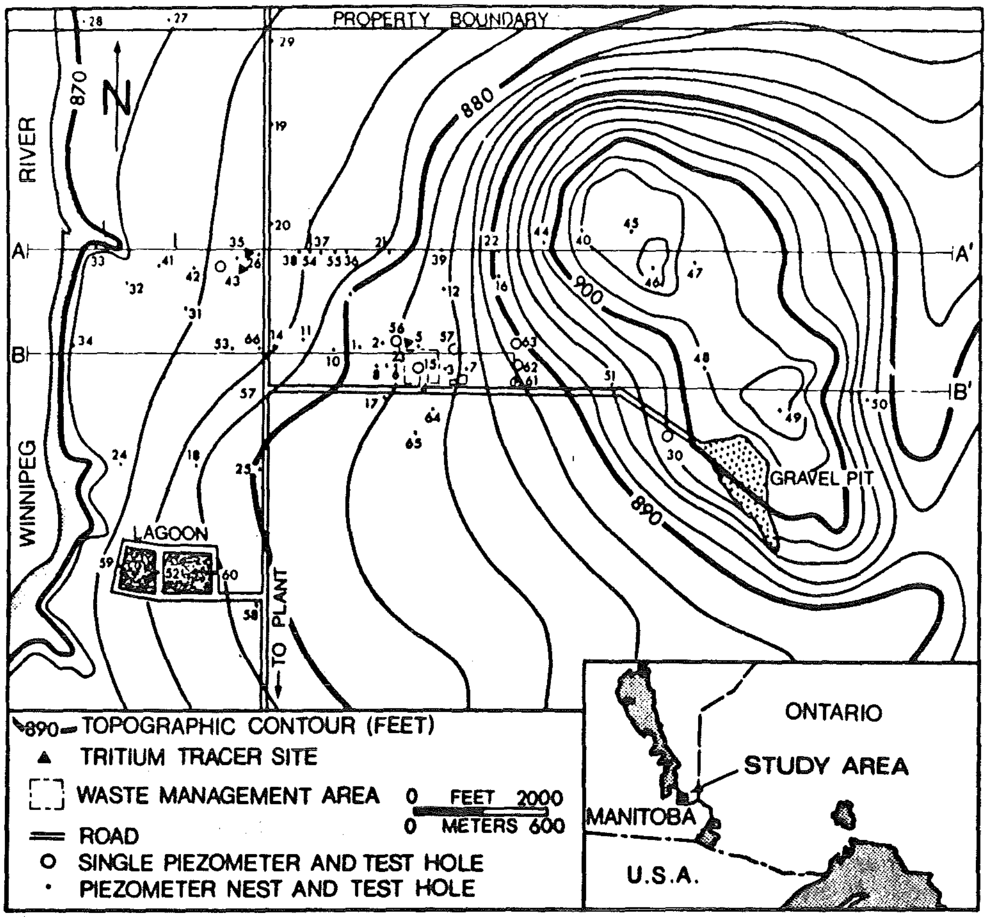

FIG. 1. Topography and instrumentation in the study area at the Whiteshell Nuclear Research Establishment.

drilled in an area referred to as the detailed study area (Fig. 2). The eighth hole was drilled 0.5 mile (0.8 km) west. The cable-tool holes terminated at refusal. A geophysical survey conducted by Chagarlamudi (1971) included seismic refraction and resistivity profiles along east–west transects through the area. From the consistency of the elevation of the refusal surface and from correlation with geophysical results, the refusal depth at nearly all of the drill sites appears to be at the bedrock surface.

Observations of the clay and till confining layers were made along the sidewalls of trenches excavated to 15 ft (4.5 m) below ground surface, and by examining relatively undisturbed material in Shelby-tube samples from boreholes.

Lithology and Stratigraphy

The geology of the WNRE study area is summarized in Fig. 3, which illustrates the stratigraphic relations between the silt, clay, till, and sandy deposits that overlie the bedrock.

The surficial silt unit in Fig. 3 includes thin, laminated beds of fine-grained sand, sandy silt, and clay. Where the silt unit is less than 2 ft (0.6 m) thick, it is somewhat obscured by the effects of soil profile development. The contact between the silt and the underlying lacustrine clay is gradational over a few feet (~1 m).

The lacustrine clay unit contains numerous carbonate pebbles and clasts of till. It is varved at some locations. A layer of clay-bound Precambrian cobbles and boulders occurs at the base of the till. Considerable difficulty was en-

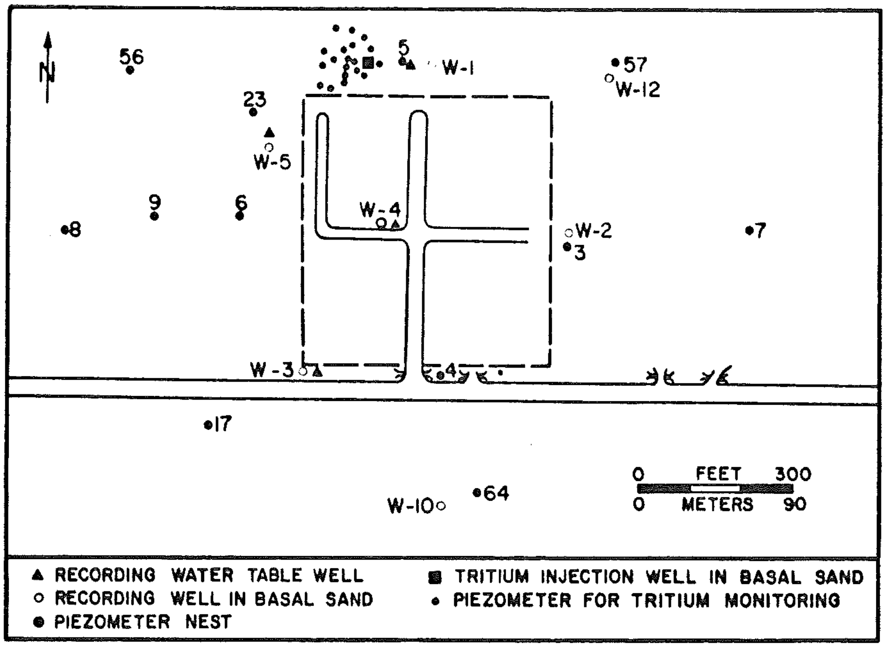

FIG. 2. Location of wells, piezometers, and tritium tracer site in the detailed study area.

countered penetrating this unit during both auger and cable-tool drilling.

Between the clay–loam till and the Precambrian bedrock there is a sandy deposit referred to in this paper as the basal sand. It is the only aquifer in the study area and appears at surface in the eastern portion of the study area as a medium-grained, well-sorted lacustrine sand. It grades laterally westward into an heterogenous sandy unit with numerous lenses of sandy silt, silt, and clay. In some areas it includes gravel which contains various amounts of sand and silt. Although the eastern extension of the basal sand outcrops on both sections A–A′ and B–B′, it is not vertically continuous in both sections. On section B–B′ the continuity is interrupted by the clay and till.

On the basis of X-ray diffraction and petrological studies Mills and Zwarich (1970) report that the lacustrine silt, clay, and clay–loam till have essentially the same mineralogy. The units consist of montmorillonites, kaolinite, chlorite, vermiculite, feldspars and quartz, and fine-grained calcite. The predominant clay minerals are of the montmorillonite group. The pebbles and cobbles in the clay–loam till are predominantly dolomite with the remainder being weathered Precambrian material. The basal sandy drift and lacustrine sand are composed of quartz and feldspar, with minor amounts of other silicate minerals, and calcite and dolomite.

History

The glacial and lacustrine sediments are a result of Wisconsinan glaciation and associated episodes of Glacial Lake Agassiz. Initial deglaciation occurred about 12 000 years ago and was followed by various episodes of Lake Agassiz which fluctuated between 1045 and 980 ft (318 and 298 m) above present-day sea level (Prest 1968). The present elevations in the study area range from 840 to 910 ft (256 to 277 m) above sea level.

McPherson (1968, 1970, 1971) conducted studies of the regional Pleistocene stratigraphy in the area to the immediate west of WNRE. He made use of field mapping, till pebble lithology, auger drilling, and water well log interpretation. The stratigraphic succession, described by McPherson (1971), from oldest to youngest, is (*a*) the Belair Drift consisting of till and associated outwash sediments, (*b*) the Libeau Drift which is primarily till, and (*c*)

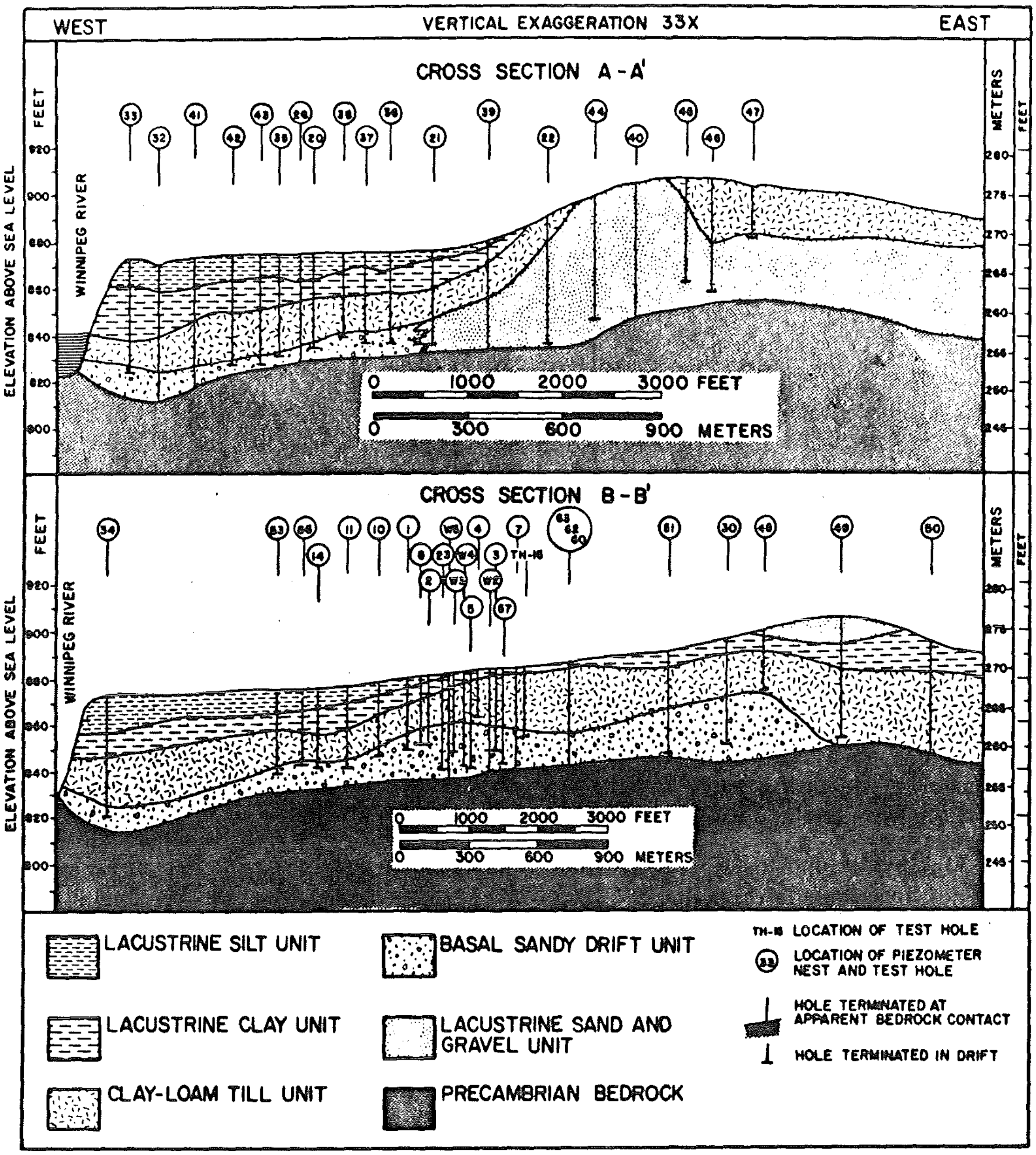

FIG. 3. Stratigraphy and locations of boreholes and piezometers along cross sections A–A' and B–B'.

Lake Agassiz deposits consisting of a lower clay unit, a middle mud unit, and an upper silt unit.

In the WNRE area, the lacustrine silt represents a late-phase shallow-water deposit of glacial Lake Agassiz. The underlying clay was deposited during an earlier high-water interval. The carbonate pebbles and clasts of till in the lacustrine clay were probably dropped from floating ice masses on Lake Agassiz during periods of sedimentation.

The clay–loam till correlates with the Libeau

FIG. 4. Fractured clay in an excavation in the detailed study area.

Drift described by McPherson (1970) in the area west of the Winnipeg River. This unit was deposited as a result of a minor glacial re-advance from the north–west.

The sandy drift underlying the clay–loam till correlates stratigraphically with the Belair Drift described by McPherson (1970). This unit is mainly glacial-fluvial material, probably deposited in close proximity to a glacier front. It is characterized by angular rock fragments of Precambrian shield lithologies set in a matrix which varies from sand to clayey silt.

Fractures

The Shelby-tube samples and observations in trenches and pits indicate that the clay and till are fractured through their entire thickness. The sidewalls of a newly-opened pit examined in 1972 had fractures over the entire 15 ft (4.5 m) depth of the excavation. The fractures are continuous across the contact between the till and the lacustrine clay. The average horizontal spacing of the fractures in the till as observed in the open excavation is about 4 cm. Variation of fracture spacing with depth is currently under investigation. Horizontal fractures were not evident in the excavation. Figure 4 is a photograph of the fractured clay as it occurred in an excavation in the detailed study area. The surface of many of the fractures in the excavation as well as those in cores from the area were coated with calcium carbonate (calcite) precipitates. Iron oxide coatings on fracture surfaces were evident in the upper few feet of the excavation.

In 1969 tritiated water was injected through a well screened between depths of 18 and 21 ft (5.4 and 6.3 m). The location of the injection site is shown in Fig. 1. The injection zone straddles the gradational contact between the lacustrine clay and the clay–loam till. Core samples at the site indicate no sand or silt interbeds in the injection zone. The flow path of the tritiated water was mapped by sampling piezometers several times annually and determining the tritium content in the laboratory using a liquid scintillation counter. By 1973 tritium was observed in piezometers at distances up to 40 ft (13 m) from the injection piezometer, indicating a west-northwest travel direction.

The flow rate of the tritiated water is in the order of 10 ft/year (~3 m/year), which under the existing hydraulic gradients and intergranular porosities would require an hydraulic conductivity of at least 10^{-5} ft s^{-1} (3×10^{-4} cm s^{-1}). The intergranular conductivity of the clay and till units is very low. For the clayey units Shawinigan Engineering (1960) obtained values of intergranular hydraulic conductivity of between 1.7×10^{-11} and 8.6×10^{-10} ft s^{-1} (5.1×10^{-10} and 2.6×10^{-8} cm s^{-1}). Additional intergranular conductivity data presented in the next section are in agreement with these values. The high flow rates of the tritiated water in units with such low intergranular conductivity is evidence that the fractures are open and interconnected and that they control the bulk hydraulic conductivity of the till and clay.

Origin of the Fractures

Fractures in the glacial till and clays of the Interior Plains may have formed (*a*) by regional extension of the earth's crust due to crustal rebound following glacial loading, (*b*) by a primarily vertical stress imposition and release in the Quaternary deposits following the readvance of glaciers during late Wisconsinan time, or (*c*) by volume changes due to desiccation during altithermal conditions in the Atlantic period of post-Wisconsinan time (7500–3000 years B.P.). Volume changes could also have resulted from osmotic or ion-exchange processes. Many fractures that occur in the present zone of soil profile development (upper 6 ft (2 m)) are attributed to desiccation and freeze–thaw processes. In this paper we are primarily concerned with the fractures that occur well below the soil zone, extending to depths of 50 ft (15 m) or more.

Vonhof (1970) has suggested that vertical fractures at several locations near Saskatoon and Esterhazy, Saskatchewan, formed either "(*a*) as fractures between horizontal to near-horizontal shear joints or (*b*) during loading or unloading of underlying till by advancing or receding continental ice sheets". Horizontal fractures in till at the same locations were considered to have formed during unloading that occurred during recessions of the glacial ice-sheet.

Although some horizontal fractures may be present at WNRE, none were observed in the till in the excavations. Because the vertical fractures extend through the lacustrine clay as well as through the till and because horizontal fractures are uncommon or absent in the till it is concluded that glacial unloading is probably not the dominant fracturing mechanism. The most likely cause of the fracturing of the till and clay is crustal extension that has occurred during post-glacial uplift. Fracturing has probably been aided by volume changes due to desiccation or geochemical processes.

Evidence for crustal rebound following glacial loading in the Lake Agassiz region was presented by Johnston (1946) who first observed that Lake Agassiz strandlines in Manitoba are tilted northward. In the WNRE area the amount of post-glacial uplift or rebound that has occurred is about 200 ft (61 m).

If the fractures in the tills and clays of the Interior Plains were formed by regional bending of the earth's crust, the local patterns in the till and clay would be controlled to some extent by the structural trends of the local bedrock. Because of the nearly continuous cover of Quaternary deposits, detailed structural mapping of the bedrock has not been done in the vicinity of WNRE. However, about 15 miles (23 km) north of the area the structural lineations and fold trends obtained by mapping outcrops of Precambrian rocks are principally east–west (Ermanovics 1971). As indicated above, the orientation of one set of fractures in the clay and till at WNRE is approximately east–west, possibly in response to the east–west structural trends in the bedrock. The other set of fractures in the clay and till strike approximately north–south and could be due to a north–south fracture system. Hydrologic evidence presented below suggests that a major fracture zone with this orientation is present.

Groundwater Flow System

Field Studies

Interpretations of the groundwater flow patterns along two cross sections through the study area are shown in Fig. 5. The distribution of equipotential lines is based on average water levels obtained from the piezometer network during the period May to July, 1971. An indication of the amount of piezometer control along the cross sections is given in Fig. 1. Each piezometer nest comprises three to five individual piezometers, usually set in separate stratigraphic units and in individual drill holes a few feet (~1 m) apart. The flow patterns are representative of the conditions that prevailed during the entire study period from 1968 to 1973. The direction of groundwater flow is generally parallel to cross section A–A′ but significant components of flow in the third dimension are common along cross section B–B′ (Cherry *et al.* 1973).

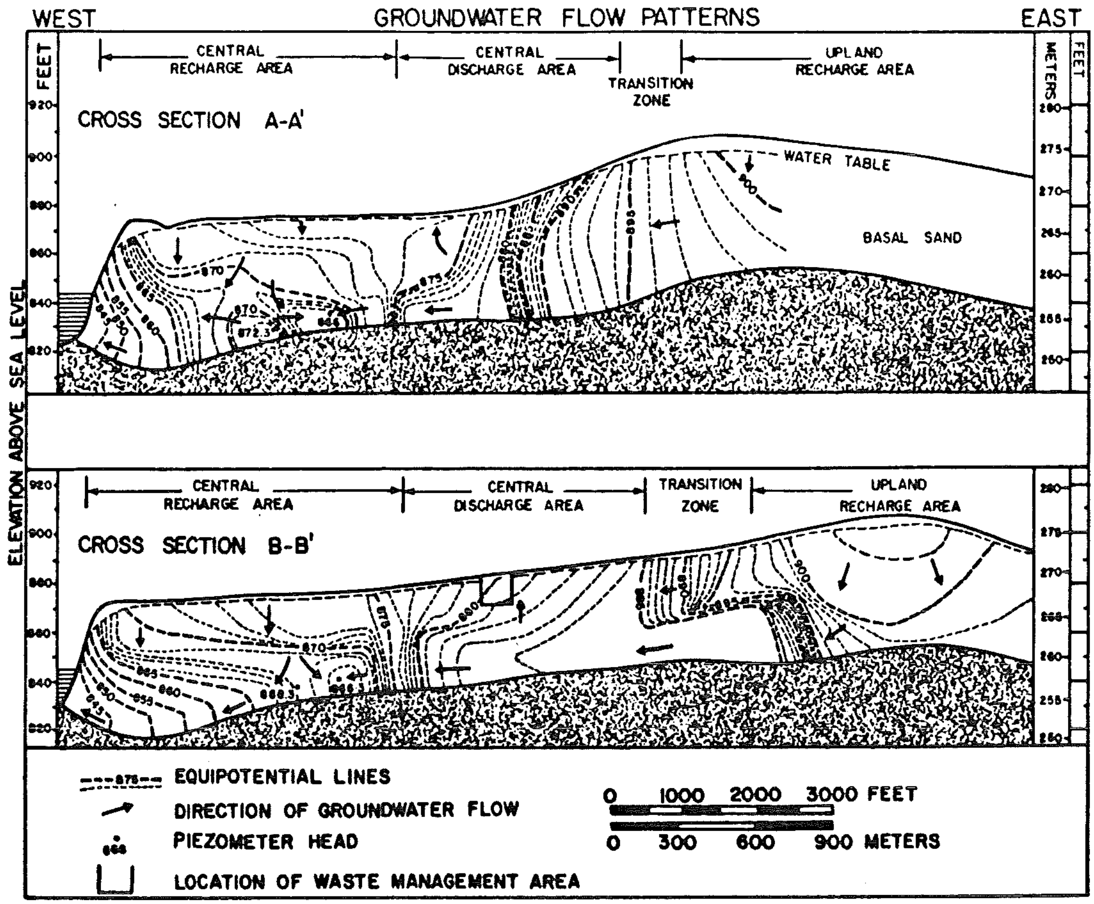

FIG. 5. Groundwater flow patterns along cross sections A–A′ and B–B′ interpreted from water-level data from the wells and piezometers.

The groundwater flow system has two distinct types of hydrologic environments; *recharge areas* where the water moves downward from the water table into the flow system, and *discharge areas* in which the water moves upward to the water table, where it is removed from the groundwater system by evapotranspiration and in some areas by surface runoff. The recharge and discharge areas are separated by *transition zones* in which the flow is mainly lateral.

Water enters the groundwater zone in the upland recharge area and moves downward and then laterally towards the Winnipeg River. As the water moves westward in the lacustrine sand and in the basal sand, upward hydraulic gradients in the overlying till and clay cause some of the water to flow upward into the central discharge area. Most of the water, however, continues to flow westward in the basal sand to a groundwater *sink* located about 0.5 miles (0.8 km) east of the Winnipeg River (Fig. 5). The *sink* is an area of anomalously low hydraulic head into which water flows from east and west and from above. The zone of anomalously low hydraulic head extends northward from cross section B–B′ to the northern boundary of the study area and possibly further. This low-head zone is probably caused by a large north–south trending fracture or fracture zone which has sufficient hydraulic conductivity to channel water out of the basal sand.

It is apparent from the hydraulic head distribution in the till and clay that these beds are part of a hydraulic continuum comprising all of the Quaternary deposits. Water is being effectively transmitted through the till and clay confining beds, primarily in the vertical direction. The problem now is to determine what bulk hydraulic conductivity and storativity are controlling the groundwater flow through these beds. Part of the answer to this problem (the bulk hydraulic conductivity) was obtained by mathematical simulation studies described below.

Simulation Studies

The groundwater flow pattern is continually varying in response to climatic fluctuations, however the piezometer and well data obtained during the 5 year observation period indicate that the system can be approximated reasonably well by the flow pattern shown in Fig. 5. To obtain estimates of the bulk hydraulic conductivities of the hydrostratigraphic units in the area, a steady-state digital simulation model was used. The model is based on the two-dimensional solution of Richards' equation describing steady-state flow of water in nonhomogeneous, anisotropic porous media, bounded by an irregular water table and no-flow barriers. Mathematical solutions were obtained using a computer program employing isoparametric finite elements developed by Pinder and Frind (1972). The program was adapted for use in the cross-sectional plane by Gillham and Farvolden (1974).

The flow pattern along cross section A–A′ was chosen for detailed digital simulation studies because the groundwater flow in the vicinity of this cross section is reasonably two-dimensional (Grisak 1974). The finite element grid used in the simulation is shown in Fig. 6. A fixed water table was used as the upper boundary of the model. The Precambrian bedrock, except in the apparent fracture zone, was assumed to be an impermeable barrier at the base of the flow domain. The fracture zone was simulated in the model as a groundwater sink using a fixed-head node. The eastern part of the flow area was bounded by a no-flow boundary representing a groundwater flow divide. Relative vertical and horizontal hydraulic conductivities were assigned to each of the finite elements.

Using these boundary and internal conditions the model was used to simulate the hydraulic head distribution in the flow domain. The simulated head distributions were then compared to the head values obtained from the piezometer and observation well network (Fig. 5). The relative hydraulic conductivities assigned to the elements were adjusted until the simulated head distribution was similar to the head distribution measured in the field.

The best simulation results were obtained using the relative hydraulic conductivities shown in Fig. 6 (bottom). Figure 7 compares the simulated and field hydraulic head distributions. The direction of the flow arrows on Fig. 7 are adjusted for vertical exaggeration after the method indicated in van Everdingen (1963). An anisotropy ratio (K_h/K_v) of 10 for hydraulic conductivity in the basal sand was found to produce the best simulation results. Since an interconnected fracture system governs the hydraulic characteristics of the confining layers and the fractures are considered to be relatively continuous, a K_h/K_v value of 1 was assigned to these layers. The model results were not significantly affected by changing the anisotropy in the confining layers to K_h/K_v = 10 or 0.1. They were affected much more significantly by changing the anisotropy in the basal sand.

Although we cannot prove rigorously that our 'best-fit' simulation result is a unique solution, uniqueness is suggested by the fact that (*a*) numerous hydraulic-head points from the field are available for comparison to the model results and (*b*) the simulated head pattern is relatively sensitive to changes in the hydraulic conductivity of the model elements and to changes in the anisotropy of the basal sand. For example, a change of anisotropy ratio in the basal sand to $K_h/K_v = 1.0$ causes a relatively large shift in the transition zones between the recharge and discharge areas. The transition zones are displaced about 200 ft (61 m) eastward toward the upland recharge area. An increase in anisotropy ratio to $K_h/K_v = 100$ shifts the boundaries a similar distance westward toward the Winnipeg River.

The sensitivity of the simulated flow pattern to changes in hydraulic conductivity is illustrated by observing the variation in the vertical component of the hydraulic gradient obtained in the central discharge and central recharge areas. An increase of one order of magnitude in the hydraulic conductivity of the till placed the 875 equipotential line (Fig. 7 (bottom)) above the contact between the till and the clay,

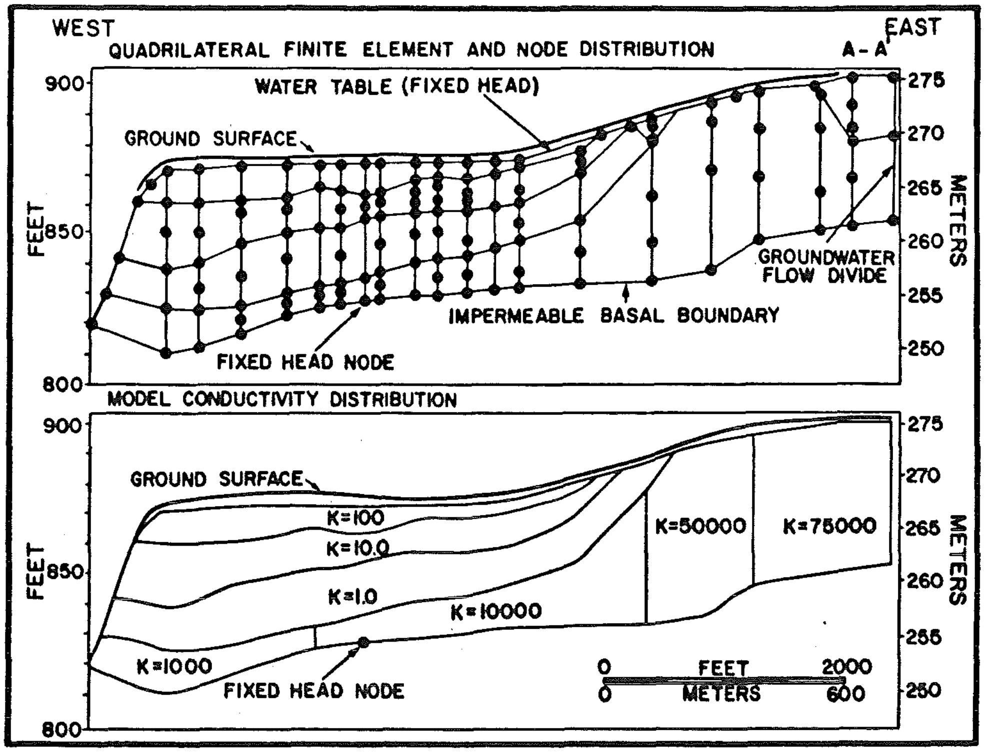

FIG. 6. Arrangement of elements, boundary conditions, and relative hydraulic conductivities used in the best-fit model simulation of cross section A–A'.

rather than near the till – basal sand contact. When the hydraulic conductivity of the till is decreased by an order of magnitude, the horizontal sections of the equipotential lines 878 to 875 are all within 5 ft (1.5 m) of the contact between the basal sand and the till. The positions of the equipotential lines in Fig. 7 (bottom) are quite sensitive to hydraulic conductivity changes as small as 1/3 an order of magnitude or less.

Use of a fixed-head node to represent the fracture zone produced a realistic hydraulic head pattern in the flow system beneath the central recharge area. The node simulates the removal at the node of water from the cross-sectional flow area. It is analogous to flow downward from the sand into the apparent fracture zone.

The most important conclusion from the simulation study is that the bulk hydraulic conductivity of the clay–loam till overlying the basal sand in the central part of the study area is approximately four orders of magnitude less than the bulk hydraulic conductivity of the basal sand in this area.

Analysis of Laboratory Data

Laboratory test data on the intergranular properties of the till and clay in the study area and on similar deposits located elsewhere in the Interior Plains Region of Canada were used to estimate representative values for the intergranular hydraulic conductivity and intergranular storativity of these deposits. The representative intergranular values were then compared to the results obtained from field pumping tests and from the simulation studies. The laboratory data, consisting of void ratios, compression indices, liquid limits, and moisture contents from borehole samples collected using the Shelby-tube method were provided from the files of various government agencies and private consulting firms.

Intergranular hydraulic conductivity values for the lacustrine clay in the study area were obtained by Shawinigan Engineering (1960)

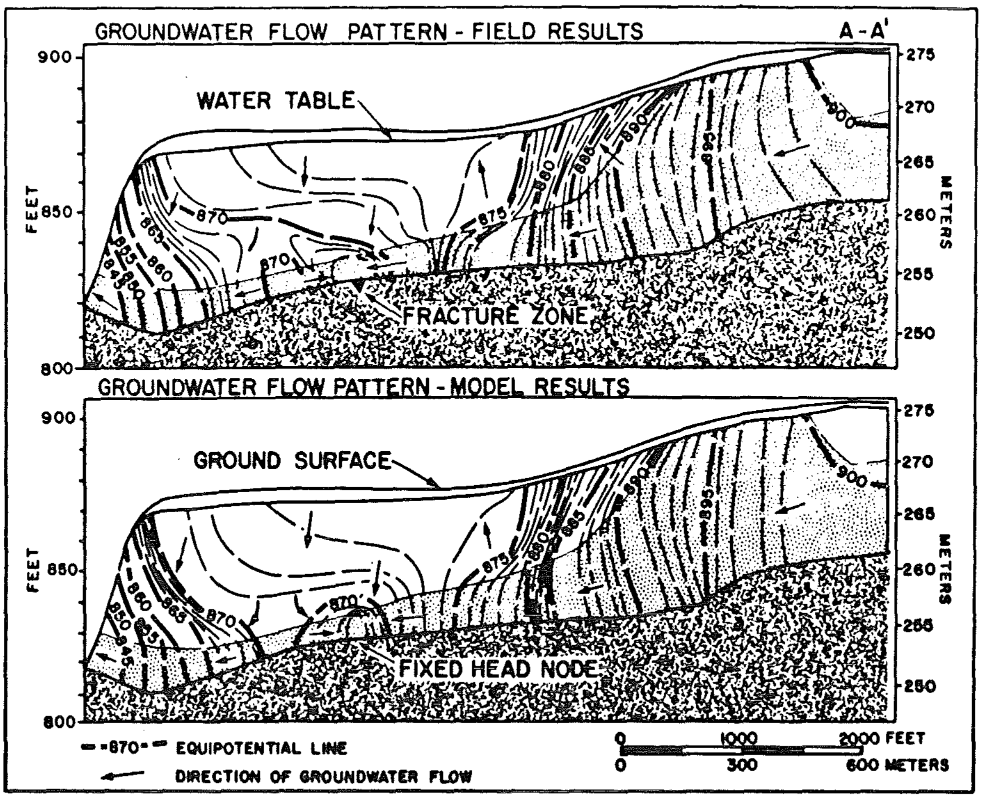

FIG. 7. Comparison of the simulated flow pattern with the pattern obtained from the field data.

using an empirical linear relationship (Taylor 1948, p. 116) between the void ratio plotted to natural scale against conductivity on a logarithmic scale. The conductivities were computed from consolidation test results using the calculation procedure outlined in Taylor (1948). This produced values for the intergranular hydraulic conductivity of the lacustrine clay between 1.7×10^{-11} and 8.6×10^{-10} ft s^{-1} (5.2×10^{-10} and 2.6×10^{-8} cm s^{-1}). This order of magnitude is typical of the intergranular hydraulic conductivity of clayey deposits (Hvorslev 1951).

Data from 171 time-consolidation tests of clay–loam till and glaciolacustrine clay samples from the Interior Plains Region were used to calculate hydraulic-conductivity values for these deposits. The theory on which the calculations are based is presented by Taylor (1948). The calculation results are given in Table 1 which indicates that representative intergranular hydraulic conductivities for clay–loam till and glaciolacustrine clay are in the range of 4×10^{-10} to 1×10^{-12} ft s^{-1} (1.2×10^{-8} to 3×10^{-11} cm s^{-1}). These results are close to the intergranular conductivity range obtained by Shawinigan Engineering (1960) for the lacustrine clay in the WNRE area.

The laboratory test results were also used to determine the intergranular storativity of the clay and till. Specific storage, S_s, as defined by Hantush (1964) is "the volume of water that a unit volume of aquifer releases from storage because of expansion of the water and compression of the grains under a unit decline in average head within the unit volume". Domenico (1972) derived an equation for the specific storage based on Jacob's (1940, 1950) original derivation of the differential equation governing the flow of water in an elastic aquifer.

TABLE 1. Intergranular hydraulic conductivity[a] and specific storage[b] of till and clay samples from the Interior Plains Region, calculated from time-consolidation tests

Location	No. of samples	K (mean, ft s^{-1})	K (standard deviation, ft s^{-1})	S_s (mean value, ft^{-1})	S_s (standard deviation, ft^{-1})
Manitoba till	34	2.01×10^{-10}	1.2×10^{-10}	3.02×10^{-3}	2.03×10^{-3}
Saskatchewan till	24	1.9×10^{-10}	2.36×10^{-10}	3.4×10^{-3}	9.2×10^{-4}
Alberta till	27	9.16×10^{-11}	1.35×10^{-10}	3.2×10^{-3}	2.6×10^{-3}
Interior Plains lacustrine clay	86	8×10^{-11}	7.6×10^{-11}	7.2×10^{-3}	3.2×10^{-2}

[a]K = hydraulic conductivity.
[b]S_s = specific storage.
NOTE: Consolidation test data was obtained from the following sources: Regina Office, Dep. Regional Econ. Expansion, Manitoba Dep. Highways, Sask. Dep. Highways, Manitoba Water Resources Branch.

TABLE 2. Engineering properties and specific storage of the lacustrine clay in the WNRE area

Boring No.	Depth (ft)	Initial void ratio (e)[a]	Compression index (a_v)[a]	Conductivity[b] (ft/s^{-1})	Specific storage[c] (S_s, ft^{-1})
1	6	1.445	0.37	1.7×10^{-11} to 3.8×10^{-10}	4.7×10^{-3}
2	11	1.619	0.49	5.8×10^{-11} to 8.6×10^{-10}	5.8×10^{-3}
3	10	1.507	0.62	9.4×10^{-11} to 1.0×10^{-10}	7.7×10^{-3}
4	8	1.591	0.51	—	6.1×10^{-3}

[a]Data from Shawinigan Engineering Company Limited (1960).
[b]Computed using the empirical relation given by Taylor (1948, p. 116).
[c]Computed using Eq. [3].

Using Domenico's notations the equation is

[1] $$S_s = \alpha \rho_\omega g + \eta \rho_\omega g \beta$$

where α is the vertical compressibility of the aquifer skeleton, ρ_ω is the density of water, g is the gravity acceleration constant, η is the porosity, and β is the compressibility of water.

The compressibility of water compared to the compressibilities of nonindurated geologic materials is negligible. The equation for specific storage therefore becomes

[2] $$S_s = \alpha \rho_\omega g$$

which Domenico and Mifflin (1965) have shown to be equivalent to

[3] $$S_s = \frac{a_v \rho_\omega g}{1 + e}$$

where a_v is the compression index and e is the initial void ratio.

The specific storage of the lacustrine clay at WNRE was obtained using Eq. [3] and the compression index and void ratio reported by Shawinigan Engineering (1960) and summarized in Table 2. The range of S_s values obtained for the lacustrine clay at WNRE is between 4.7×10^{-3} and 7.7×10^{-3} ft^{-1} (1.4×10^{-2} and 2.3×10^{-2} m^{-1}), which is in the upper range of S_s values typical of plastic clay indicated by Domenico and Mifflin (1965).

Calculation of specific storage for the clay–loam till was accomplished using a method somewhat less direct than that used in calculations for the lacustrine clay. Although the values of compression index and void ratio for the clay–loam till at WNRE were not obtained in the Shawinigan Engineering laboratory studies, natural water content and liquid limits were determined (Table 3).

Based on the definition of void ratio it can be shown that

[4] $$e = M_c G_s$$

where M_c is the water content as a fraction and G_s is the specific gravity of the solids. Terzaghi and Peck (1960, p. 66) indicate that an empirical relation exists between the liquid limit (L_w) of a remolded sample and the remolded compression index (a_v') of that sample

[5] $$a_v' = 0.007\,(L_w - 10\%)$$

On the basis of comparative studies of field samples and laboratory samples these authors suggest that a further relationship holds be-

TABLE 3. Specific storage of clay–loam till at WNRE calculated from laboratory test data[a]

	Lab test data			Calculated values		
				Compression index (a_v)		
	Moisture content (M_c, %)	Liquid limit (L_w, %)	Void ratio ($e = M_cG_s$)	Remolded[b] (a_v')	Undisturbed[c] ($a_v = 1.3\,a_v'$)	Specific storage[d] (S_s, ft^{-1})
Lower limit	14	22	0.38	0.084	0.109	2.5×10^{-3}
Average	18	29	0.49	0.133	0.173	3.6×10^{-3}
Upper limit	25	39	0.68	0.203	0.264	4.9×10^{-3}

[a]Number of samples tested 8; test data from Shawinigan Engineering Company Limited (1960).
[b]Calculated using Eq. [5].
[c]Calculated using Eq. [6].
[d]Calculated using Eq. [3].

tween the remolded compression index and the undisturbed compression index, given by

$$[6] \qquad a_v \sim 1.3\, a_v'$$

They report the reliability of these relationships to be about ±30%, which is satisfactory for determining intergranular specific storage for hydrologic purposes.

Using the laboratory data listed in Table 3, the void ratio e and the compression index a_v were calculated using Eqs. [4] and [6], respectively. Specific storages, S_s, were then calculated from Eq. [3]. Table 3 lists the calculated specific storage values for the clay–loam till and indicates that the average value is about a third of an order of magnitude less than the specific storage of the lacustrine clay. The lower value of specific storage coupled with the low water contents of the till (Table 3) suggest some degree of overconsolidation, possibly due to glacial loading.

Specific storage values for the till and clay samples from elsewhere in the Interior Plains were calculated using the compression indices determined in the laboratory and Eq. [3]. These storativity values, given in Table 1, agree favorably wtth those calculated for the WNRE samples.

Considerable care must be observed when calculating specific storage values using compression indices of shallow preconsolidated deposits. The compression index reported in soil mechanics studies is usually the value of the slope of the curve obtained by plotting e *versus* pressure *after* the preconsolidation pressure has been attained. The preconsolidation pressure is the highest confining pressure that a sample has been subjected to prior to testing. In deposits that are not overconsolidated the preconsolidation pressure is the confining pressure at the depth from which the sample was taken. However, in overconsolidated deposits, the preconsolidation pressure is higher than the confining pressure at the depth from which the sample was taken. In this case, the compression index of interest to hydrogeologists is not necessarily the same as the compression index routinely reported in soil mechanics studies.

To illustrate the problem, consider a point P in a preconsolidated clay at depth z below ground surface. If the hydraulic head, ϕ, is reduced at P by the amount $\Delta\phi$, the clay will consolidate as a function of the change in effective pressure, $\Delta\bar{\sigma}$. $\Delta\bar{\sigma}$ is produced by the reduction of the pore pressure, u. In order to use a_v reported in most soil mechanics studies in the specific storage calculation, $\Delta\bar{\sigma}$ produced by $\Delta\phi$ must occur over the range of pressures to which the reported a_v is applicable, that is at pressures greater than the preconsolidation pressure.

The e *versus* $\bar{\sigma}$ relation plotted on semilog graph paper has (*a*) an initial section with a small slope which occurs during pressure changes in the load range lower than the preconsolidation pressure, (*b*) a transition zone in which the preconsolidation pressure is located, and (*c*) a steeply sloping section in which pressure changes greater than the preconsolidation pressure occur. For hydrologic studies such as those conducted at WNRE, calculation of the total $\bar{\sigma}$ at P before $\Delta\phi$ occurs will indicate which portion of the e *versus* $\bar{\sigma}$ curve should be used to obtain the appropriate compression index. In overconsolidated deposits at relatively shallow depths, the compression index obtained from the shallow portion of the curve is likely to be the correct value.

Calculations of the total $\bar{\sigma}$ of the samples tested by Shawinigan Engineering (1960) indicate that the clay and till are only slightly, if at all, overconsolidated. Therefore, the compression indices used in the specific storage calculations are those from the portion of the curve at higher pressures than the preconsolidation pressure.

Response During Aquifer Pumping

To obtain field values for hydraulic conductivity and storativity of the aquifer and the till and clay confining beds, seven pumping tests were conducted in the detailed study area using wells screened in the basal sand.

Each test consisted of pumping one well in the aquifer and observing the response of the hydraulic head in other wells in the aquifer and in the water-table zone. The observation wells were equipped with water-level floats and Leopold-Stevens Type-F recorders. The wells used in the pumping tests are shown in Fig. 2. Wells RW1, RW3, RW5, and RW12 were used individually as pumping wells. Frequent manual water-level measurements were made in the observation wells to validate the chart records. Recorder charts with 24 h full-chart widths were used. Five of the tests were less than 1 day in pumping duration. The sixth test involved 5 days of pumping and the seventh test lasted for 32 days. During the 32 day test, manual water-level measurements were made in the piezometer nests located in and near the detailed study area. The purpose of the long pumping test was to observe the hydraulic head response in the confining beds overlying the aquifer. The shorter-period tests were primarily conducted to determine the hydraulic conductivity and storativity of the aquifer.

Aquifer Analysis

To determine the hydraulic conductivity and storativity of the aquifer, the drawdown and recovery data obtained from the wells in the basal sand during the short-term pumping tests were analyzed using (*a*) the Jacob semi-logarithmic graphical method (Cooper and Jacob 1946), (*b*) the Theis graphical superposition method (Wenzel 1942), and (*c*) the semi-logarithmic graphical method of recovery analysis derived from the Theis aquifer response model (Wenzel 1942).

The Theis, Jacob, and recovery methods are based on numerous assumptions, such as the aquifer is horizontal, isotropic, homogeneous, uniform in thickness, infinite in lateral extent, and confined by impermeable beds. Drilling results and the shapes of drawdown cones (Clister 1973; Grisak 1974) indicates that the aquifer is significantly heterogeneous. It is also evident that vertical leakage occurs through the till and clay confining beds as pumping continues for long time periods. In spite of these complications, however, the Theis, Jacob, and recovery methods of aquifer analysis were found suitable for obtaining useful estimates of the hydraulic conductivity and storativity of the aquifer. By restricting the analysis to suitable segments of the drawdown graphs the effects of deviations from the assumptions stated above were minimized.

Figure 8 shows representative drawdown graphs with straight-line graph segments at times of 1–3 days after the beginning of pumping. Linear drawdown segments such as these were used in the Jacob method of analysis. The apparent boundary condition encountered in the 'tailing-off' portion of the curves shown in Fig. 8 is caused by vertical leakage resulting in addition of water to the basal sand from the confining layers.

Table 4 summarizes the results of the Theis, Jacob, and recovery analyses of the short-term pumping test data. Transmissivities were converted to hydraulic conductivities using an average effective aquifer thickness of 10 ft (3.0 m). The thickness of the basal sandy unit in the detailed study area is generally about 20 to 25 ft (6.1 to 7.6 m), however, much of the unit is composed of beds or lenses of silty sand and silt. The aquifer thickness was approximated as 10 ft (3.0 m) in order to exclude these low-permeability segments of the aquifer. Table 4 indicates that the conductivities obtained from all of the tests are consistent within a relatively narrow range (Table 4). The average conductivity is 6×10^{-5} ft s^{-1} (1.8×10^{-3} cm s^{-1}), which is a reasonable value in relation to the texture of the aquifer material observed in borehole samples.

Single-well water-level response tests were

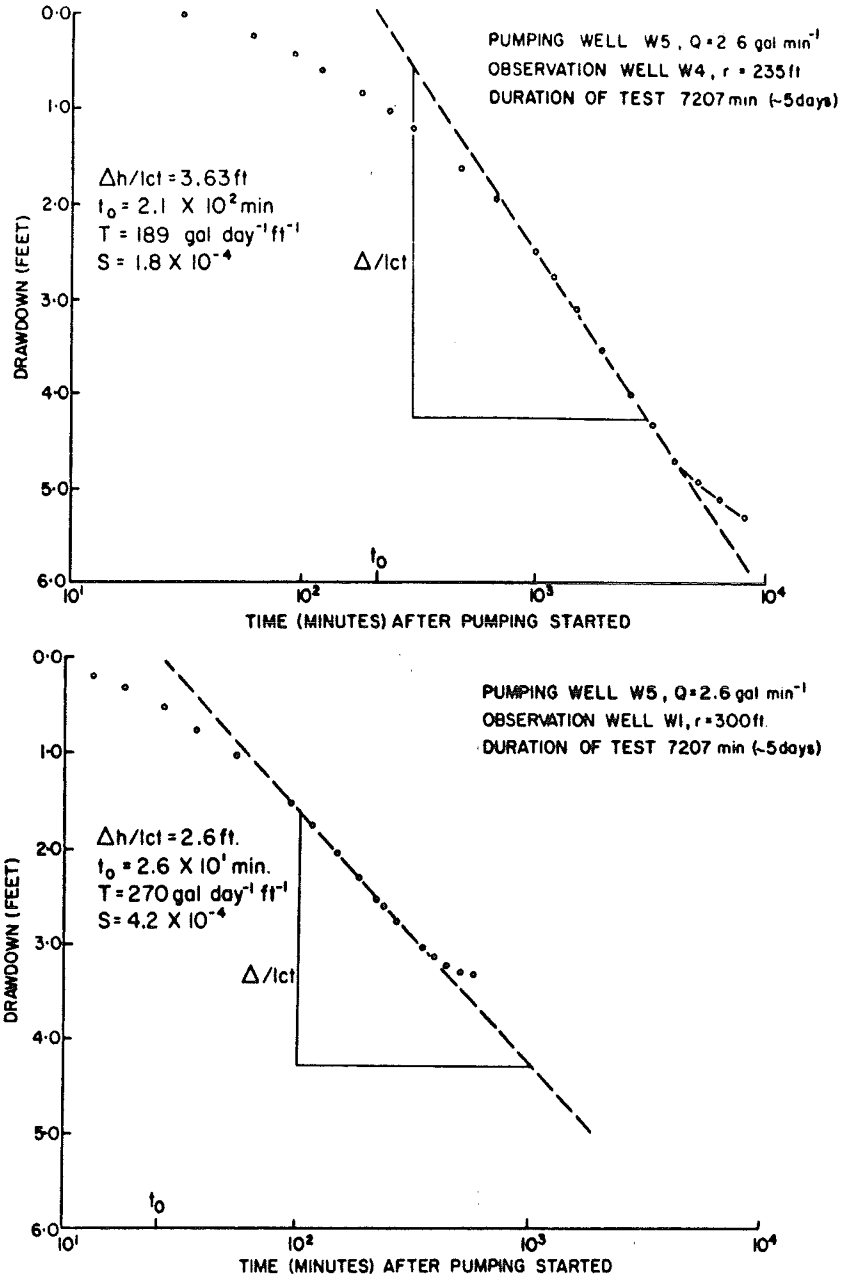

FIG. 8. Semilogarithmic graphs of drawdown *versus* time for two representative observation wells during an intermediate-term pumping test.

conducted in most of the wells in the basal sand following the methods of Cooper *et al.* (1967) and Vonhof (1975). The conductivity values from these tests do not differ significantly from the values listed in Table 4 (Grisak 1974).

The average storage coefficient is 2×10^{-4}, which is consistent with compressibilities of dense sand listed in Domenico (1972).

The digital simulation study of the groundwater flow pattern along cross section A–A′ indicated that the bulk hydraulic conductivity

TABLE 4. Summary of short-term pumping test results obtained using the Theis, Jacob, and recovery method of analysis

Pumping well	Pumping rate	Pumping duration	Observation well	Analysis[a] 1	2	3	4	Transmissivity	Conductivity	Coefficient of storage
RW3 24/8/71	9.0[b]	630	RW1		×			540[c]	1.0×10^{-4}[d]	1.3×10^{-4}
			RW2		×			450	8.6×10^{-5}	8.0×10^{-5}
			RW3				×	250	4.6×10^{-5}	
			RW4		×			630	1.2×10^{-4}	2.4×10^{-4}
RW1 26/8/71	1.0	1260	57-41		×			280	5.0×10^{-5}	1.7×10^{-4}
			1-5-29		×			220	4.0×10^{-5}	4.1×10^{-4}
RW3 18/4/72	8.7	525	RW1		×			540	1.0×10^{-4}	1.6×10^{-4}
			RW3				×	474	8.8×10^{-5}	
RW5 26/4/72	2.6	7207	RW1	×				164	3.0×10^{-5}	5.4×10^{-4}
			RW1			×		205	3.8×10^{-5}	4.9×10^{-4}
			RW1		×			219	4.0×10^{-5}	3.8×10^{-4}
			RW3		×			181	3.2×10^{-5}	1.5×10^{-4}
			RW3	×				152	2.8×10^{-5}	1.9×10^{-4}
			RW4	×				184	3.4×10^{-5}	1.6×10^{-4}
			RW4		×			189	3.4×10^{-5}	1.8×10^{-4}
			RW5				×	289	5.2×10^{-5}	
RW3 1/6/72	7.6	1492	RW1		×			254	4.6×10^{-5}	1.3×10^{-4}
			RW1	×				152	2.8×10^{-5}	1.8×10^{-4}
			RW4		×			284	5.2×10^{-5}	3.5×10^{-4}
			RW4	×				219	4.0×10^{-5}	1.9×10^{-4}
			RW5	×				207	3.8×10^{-5}	3.7×10^{-4}
			RW5		×			287	5.2×10^{-5}	2.5×10^{-4}
RW12 4/9/72	3.2	750	RW1	×				159	2.8×10^{-5}	2.1×10^{-4}
			RW1		×			256	4.6×10^{-5}	1.6×10^{-4}
			RW4	×				367	6.8×10^{-5}	2.4×10^{-4}
			RW5	×				378	7.0×10^{-5}	1.4×10^{-4}
Average								290	6×10^{-5}	2×10^{-4}

[a]1. Drawdown in observation well, Theis type curve superposition method (Todd 1959).
2. Drawdown in observation well, Jacob semilogarithmic method (Todd 1959).
3. Recovery in observation well, semilogarithmic recovery method (Davis and Deweist 1966).
4. Recovery in pumping well, semilogarithmic recovery method (Davis and Deweist 1966).
[b]1 imperial gal $min^{-1} = 4.55 \times 10^{-3}$ m^3 min^{-1}.
1 imperial gal min^{-1} = 1.2 U.S. gal min^{-1}.
[c]1 imperial gal day^{-1} $ft^{-1} = 1.242 \times 10^{-2}$ m^3 day^{-1} m^{-1}.
1 imperial gal day^{-1} ft^{-1} = 1.2 U.S. gal day^{-1} ft^{-1}.
[d]ft s^{-1}, calculated by dividing transmissivity by aquifer thickness of 10 ft.

of the clay–loam till confining bed is approximately four orders of magnitude less than the bulk conductivity of the basal sand in the central part of the study area. Taking the average value of 6×10^{-5} ft s^{-1} (1.8×10^{-3} cm s^{-1}) obtained from the pumping test analysis to be representative of the basal sand in the central part of the study area, a bulk hydraulic conductivity of 6×10^{-9} ft s^{-1} (1.8×10^{-7} cm s^{-1}) for the overlying till is obtained. This value is nearly two orders of magnitude larger than the intergranular hydraulic conductivities of clay–loam till listed in Table 1. The larger field conductivity of the till is attributed to the fractures in the till and indicates that the fractures are sufficiently interconnected to allow considerable flow of groundwater through the entire deposit.

Confining Layer Response

The 32 day pumping test was conducted by pumping well W-3 in June and July, 1972. The response was observed at the recording wells and by daily manual measurements in piezometer nests located near the pumping well. Biweekly measurements were made on the remainder of the piezometer network. Due to electrical failures there were three short pumping interruptions during the test. During the remainder of the test a relatively constant pumping rate of 4.15 gal min^{-1} (19 l min^{-1}) was maintained.

Figure 9 shows the water-level response in piezometer nests 3 and 17. Generalized borehole logs of the piezometer sites and the W-3 pumping rate are also shown on these figures. The locations of the pumping well and piezometer nests are indicated in Figs. 2 and 3. The water-level graphs in Fig. 9 indicate that before the pumping test began, the hydraulic head in the aquifer was considerably above the levels in the overlying till and clay. This prepumping condition of groundwater discharge is extensive in the area which was affected by the drawdown cone caused by pumping well W-3 (Cherry *et al.* 1973).

Figure 9 shows that the water levels in some of the piezometers in the till and clay overlying the aquifer went into rapid decline soon after pumping began. The water levels in these piezometers began rising after pumping was terminated. Some of the piezometers in the other piezometer nests in the detailed study also displayed this type of water-level response.

Figure 9 also shows that some of the piezometers in the till and clay were not significantly affected by the pumping. For example the 17 ft (5.2 m) piezometer at nest 17 and the 5 ft (1.5 m) piezometer at nest 3 had water-level declines during the pumping period but did not rise after the pumping was terminated. The slow decline is attributed to the fact that the pumping period coincided with a relatively dry period. Considerable rain fell before the test was started. If the decline in these piezometers was caused by pumping, they would have responded after pumping stopped.

We conclude that the two types of piezometer response displayed during the pumping period can be accounted for using the concept of fracture conductivity in the till and clay. The piezometers with rapid and marked response to pumping must be located in or near fracture zones, whereas the other piezometers either do not intersect fractures or do not respond because the fracture connectivity was destroyed during drilling. Some of the relatively shallow piezometers such as the 13 ft (4.0 m) piezometer in nest 17 (Fig. 9) and some of the water-table wells (Grisak 1974) responded to the pumping. This indicates that the effective hydraulic connectivity of some of the fractures extends from the aquifer through to the water-table zone.

Quantitative Analysis

Hantush (1960, 1964) developed a mathematical model that describes the water-level response in confined aquifers during aquifer pumping. In this model the confining beds have significant hydraulic conductivity and storativity. The model is based on analytical solutions of the groundwater flow equations and includes the following major assumptions: the aquifer and confining beds are horizontal, homogeneous, isotropic, and areally symmetrical about the pumping well, they possess storativity linearly related to the hydraulic pressure changes, the storage volume of water in the pumping well is negligible and the pumping well is fully screened through the aquifer. Hantush produced aquifer drawdown solutions that he showed to be valid for relatively early time periods and late time periods.

Neuman and Witherspoon (1969*a,b*; 1971; 1972) extended the Hantush model using both analytical and finite-element methods. They concluded that the time limitations that Hantush placed on his solutions were overly conservative. They also presented solutions for the drawdown response in the confining beds. We have used these solutions to calculate theoretical piezometer drawdown curves for the 13 ft (4.0 m) piezometer in nest 17 and the 19 ft (5.8 m) piezometer in nest 3 (Fig. 10). Calculated drawdown curves for other piezometers in the confining beds at these nests and other nests are given by Grisak (1974). Figure 10 also shows the actual aquifer drawdown for comparison. The drawdown graphs extend only for 17 days, in order to avoid the complications introduced by the two pumping interruptions that occurred after this period. The calculated drawdown curves shown in Fig. 10 are based on bulk values of hydraulic conductivity for the aquifer [K] and the confining bed [K'] and bulk specific storativity values for the aquifer [S_s] and the confining bed [S_s']. The values for K and S_s were obtained from the aquifer analysis using the Jacob, Theis, and recovery methods mentioned above. K' was obtained from the finite-element flow pattern analysis which indicated that $K' \approx K \times 10^{-4}$.

The purpose of calculating the theoretical piezometer drawdown curves was to obtain estimates of S_s' for the confining layers. Drawdown curves were therefore obtained using

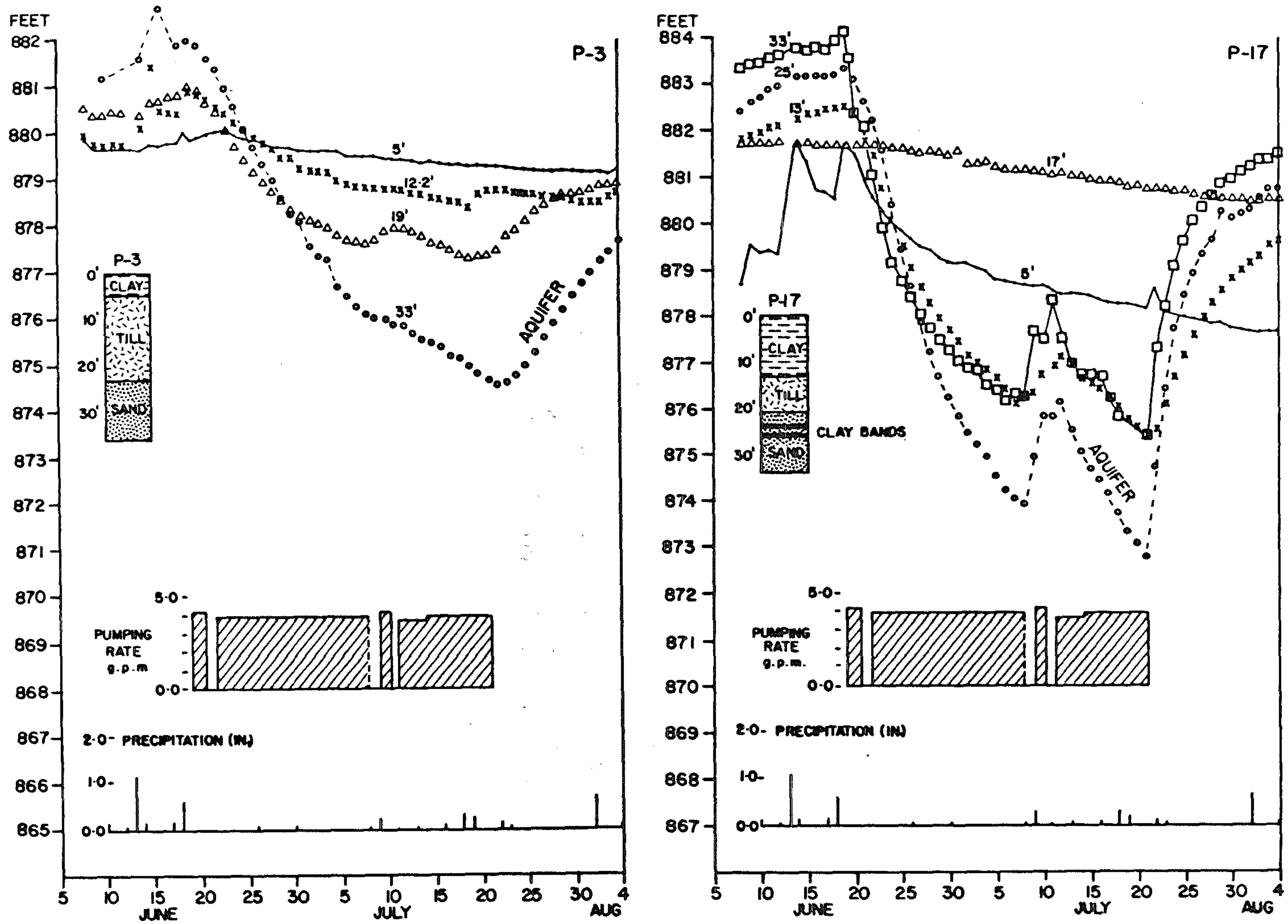

FIG. 9. Water-level response in piezometers at nests 3 and 17 during the 32 day pumping test.

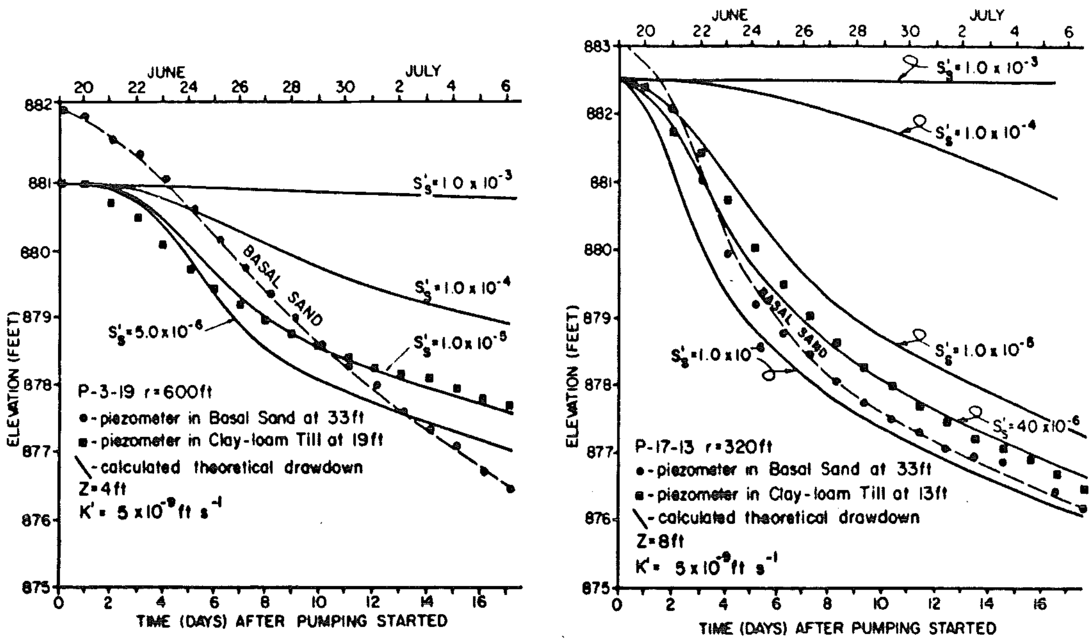

FIG. 10. Comparison of calculated and field drawdown graphs for two piezometers in the clay–loam till and for two piezometers in the basal sand aquifer. Different drawdown curves resulting from different choices of S_s' are shown. Z is the vertical distance from the top of the aquifer to the piezometer intake zone. r is the horizontal distance from the piezometer to the pumping well.

various chosen values of S_s' in order to determine which value produced drawdown curves most closely approximating the field drawdown data. Figure 10 shows the best-fit curve for the aquifer drawdowns, the best-fit curves for the piezometer drawdowns, and other theoretical piezometer drawdown curves that do not fit the field data. It is apparent from Fig. 10 that the theoretical curves approximate confining layer drawdowns only when S_s' values in the order of 1×10^{-5} to 5×10^{-6} ft^{-1} (3.0×10^{-5} to 1.5×10^{-5} m^{-1}) are used. Analysis of other piezometer drawdowns using this approach also produced S_s' values in this range (Grisak 1974).

These specific storativity values are typical of the range of specific storage given by Domenico and Mifflin (1965) for fractured or jointed rock. As discussed previously, Table 1 indicates that the representative intergranular specific storage for till and lacustrine clay in the Interior Plains Region is much larger, with values generally in the order of 10^{-3} ft^{-1} (3×10^{-3} m^{-1}). The theoretical curves in Fig. 10 indicate that if the bulk specific storativity [S_s'] of the confining layers were in the intergranular range, the till and clay would have shown little or no piezometer response during the pumping test. This indication was shown to be correct by applying the same type of analysis shown in Fig. 10 for piezometers in the till which showed little or no response (Grisak 1974).

Summary of Conclusions

1. Ubiquitous fractures in the clay–loam till and lacustrine clay at WNRE were observed in fresh excavations to depths up to 20 ft (6.1 m) and in Shelby-tube samples from depths up to 30 ft (9.1 m) below ground surface. The fractures are generally coated with calcite or iron oxide.

2. The continuity of the pattern of hydraulic head distribution in the till and clay and in the underlying aquifer indicates that the till and clay are an integral part of the active groundwater flow system in the area.

3. An injection of tritiated water as a tracer in the clay and till indicated that the fractures rather than the intergranular pore network

control the effective hydraulic conductivity of these deposits.

4. The bulk hydraulic conductivity of the clay–loam till determined by modeling the natural groundwater flow pattern using the finite-element method is four orders of magnitude less than the bulk hydraulic conductivity of the underlying sandy aquifer. Since the average aquifer value is $\sim 6 \times 10^{-5}$ ft s^{-1} ($\sim 2 \times 10^{-3}$ cm s^{-1}), the conductivity of the clay–loam till is therefore $\sim 6 \times 10^{-9}$ ft s^{-1} ($\sim 2 \times 10^{-7}$ cm s^{-1}), which is one to two orders of magnitude larger than the intergranular conductivities typical of these types of deposits.

5. The long-term aquifer pumping test showed that many of the piezometers in the till and clay respond quickly and strongly to the aquifer drawdown. It also showed that some of the piezometers do not undergo response. The fracture concept adequately accounts for this behavior. The responding piezometers intersect open fractures, whereas the others do not.

6. The analysis of the piezometer drawdowns during the long-term pumping test using the Neuman and Witherspoon method indicates that the rapid piezometer drawdowns can be accounted for by assigning specific storativity values in the range of 1×10^{-5} to 5×10^{-6} ft^{-1} (3×10^{-5} to 1.5×10^{-5} m^{-1}) to clay–loam till and clay. These values are typical of fractured rock. If intergranular specific storage values are used, the calculated piezometer drawdowns are very small or negligible.

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