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REQUISITE REMEDIAL TECHNOLOGY STUDY
Overburden & Bedrock
Hyde Park Remedial Program

RRT
Overburden and
Bedrock
OB modelling / pump tests

April, 1984
Ref. No. 1069

CONESTOGA-ROVERS & ASSOCIATES LIMITED

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MB	MB.	

April 9, 1984

Norman H. Nosenchuck, P.E.
Director, Division of Solid Waste
NYS Dept. of Environmental Conservation
50 Wolf Road
Albany, NY 12233

copy to file

Joseph Spatola, Ph.D.
Hyde Park Program Coordinator
United States EPA, Region II
26 Federal Plaza
New York, NY 10278

Subj: Hyde Park Remedial Program - Study of Requisite Technologies For Addressing Chemical Plumes

Dear Sirs:

Occidental Chemical Corporation (OCC) previously completed an assessment which indicated that chemicals were migrating from the Hyde Park Landfill. The basis for this assessment and all underlying data were submitted to EPA/State on October 12, 1983 and December 7, 1983.

Pursuant to the Settlement Agreement, OCC initiated a study of technologies to appropriately address the chemical migration and this study is now being submitted pursuant to the requirements of Paragraph C(8) of the agreement. The study includes OCC's plan for the Requisite Remedial Technology (RRT) necessary to address the chemical migration.

This submission constitutes OCC's response to the following EPA/State letters:

February 27, 1984 -- Review Comments on Hyde Park Pump Tests
February 29, 1984 -- Pump Test Protocols - An Outline
March 23, 1984 -- Review Comments on Meeting Held March 8, 1984.

A supplementary study of the benefits to the public health and environment of the RRT is being finalized by Arthur D. Little, Inc., (ADL) of Cambridge, Massachusetts.

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Special Environmental Programs
Hooker Chemical Center 360 Rainbow Boulevard South Box 728, Niagara Falls, New York 14302 716 286-3000

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The calculations and studies of ADL leading to the major conclusions included in the enclosed report of Conestoga-Rovers have been completed. However, the final ADL supplementary study does not yet contain all of the references and supporting documentation which, as a result of our discussions regarding the Bloody Run Risk Assessment, the parties recognized as integral to any comprehensive report. We expect this effort to be completed and submitted to the EPA/State no later than May 1, 1984.

If you have any questions on this submission, please call me at (716) 286-3609.

Respectfully,

John R. Nichter

John R. Nichter
Hyde Park Coordinator

2002eJRNac

Enc. 1. Requisite Remedial Technology Study, Overburden and Bedrock, Conestoga-Rovers, Ltd., April, 1984

2. Lockport Formation Jointing at the Hyde Park Landfill Site, Woodward-Clyde Consultants, March 29, 1983

cc: J. Slack - 5 copies
J. Schwartz - 5 copies
P. Buechi - 3 copies

MEMORANDUM

TO: Joseph Spatola, Ph.D.

FROM: Julia Schwartz

DATE: April 17, 1984

SUBJECT: Distribution of Hyde Park Correspondence: Letter of April 9, 1984 from John Nichter (OCC) to Norman Nosenchuck (NYSDEC) and Joseph Spatola (USEPA) regarding: HYDE PARK REMEDIAL PROGRAM - STUDY OF REQUISITE TECHNOLOGIES FOR ADDRESSING CHEMICAL PLUMES; Attachments: Requisite Remedial Technology Study Overburden and Bedrock-Hyde Park Remedial Program, April 1984; and Lockport Formation Jointing at the Hyde Park Landfill Site, March 29, 1983.

Five (5) copies of the subject correspondence were received by Ecology and Environment, Inc., Buffalo, New York, on April 9, 1984. Additional copies have been made and distributed to the following persons:

Copy 1	*Mr. George Pavlou	U.S. EPA Region II Air and Waste Management Division, New York, NY
Copy 2	*Mr. Mark Haulenbeek	U.S. EPA Surveillance and Analysis Division, Edison, NJ
Copy 3	*Mr. George Shanahan, Esq.	U.S. EPA Region II Water Enforcement, New York, NY
Copy 4	*Mr. William Walsh, Esq.	U.S. EPA Enforcement Counsel-Waste, Washington, DC
Copy 5	*Mr. Bruce Berger	U.S. Department of Justice, Washington, DC
Copy 6	*Ms. Norma Lewis	U.S. EPA MERL, Cincinnati, OH
Copy 7	*Mr. Jerry Thornhill	U.S. EPA, R. Kerr Laboratory, Ada, OK
Copy 8	*Mr. James Mercer	GEOTRANS, Herndon, VA
Copy 9	*Mr. Marcel Bergeron	U.S. Geological Survey, Ithaca, NY
Copy 10	*Mr. Herb Buxton	U.S. Geological Survey, Syosset, NY

Copy 11	Mr. Peter Crabtree	Ontario Ministry of the Environment, Hamilton, Ontario
Copy 12	Mr. Richard A. Findlay	Environment Canada, Toronto, Ontario
Copy 13	Mr. Matt Forcucci	U.S. EPA Public Information Office, Niagara Falls, NY

Please advise me of any additional distribution needed.

JS/lc
Attachments

cc: P. Crabtree, R. Findlay, G. Pavlou, M. Forcucci, B. Possin, NU-434
Files G-6; E & E corres; H.P. Corres.

cc w/o attachments: C. Morgan, R. Holmes, M. Cavanaugh

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*Document transmittal sheets sent

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1.0 INTRODUCTION

On April 30, 1982 the Stipulation and Judgment Approving Settlement Agreement (Settlement Agreement) for the Hyde Park Landfill Site was approved. Figure 1 presents a general site plan of the study area.

In accordance with Paragraph C of Addendum I of the Settlement Agreement, Occidental Chemical Corporation (OCC) undertook a program of surveys, tests and studies to determine the extent of chemical migration from the Hyde Park Landfill Site. This program was completed in December 1983. The results of the surveys, tests and studies were submitted to the government in reports entitled:

1) "Hyde Park - Bloody Run Aquifer Survey & Testing Program", Submission date - October 12, 1983.
2) "Pump Well Installation and Pump Test Results", Submission date - December 12, 1983.

Subsequently the government comments under letters received February 7, and February 27, 1984 were addressed by the following reports/meetings:

1) March 12, 1984 - Report to address EPA/STATE comments regarding the Aquifer Survey.

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APPENDIX B PRESSURE GROUT FILLERS & ADMIXTURES

APPENDIX C CHEMICAL GROUTS

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APPENDIX E ENHANCED OIL RECOVERY PROCESSES

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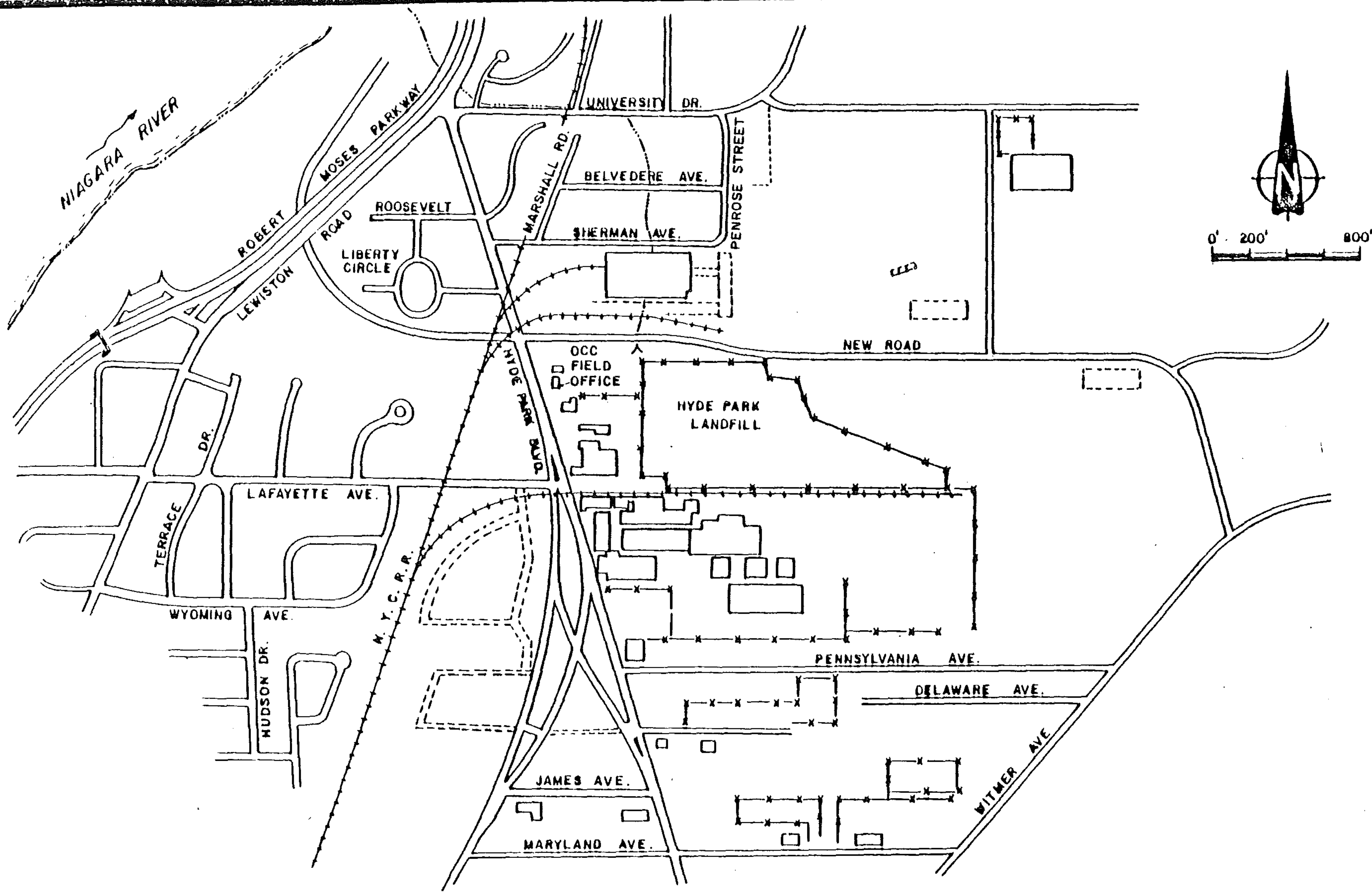

figure 1

HYDE PARK - BLOODY RUN AREA
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

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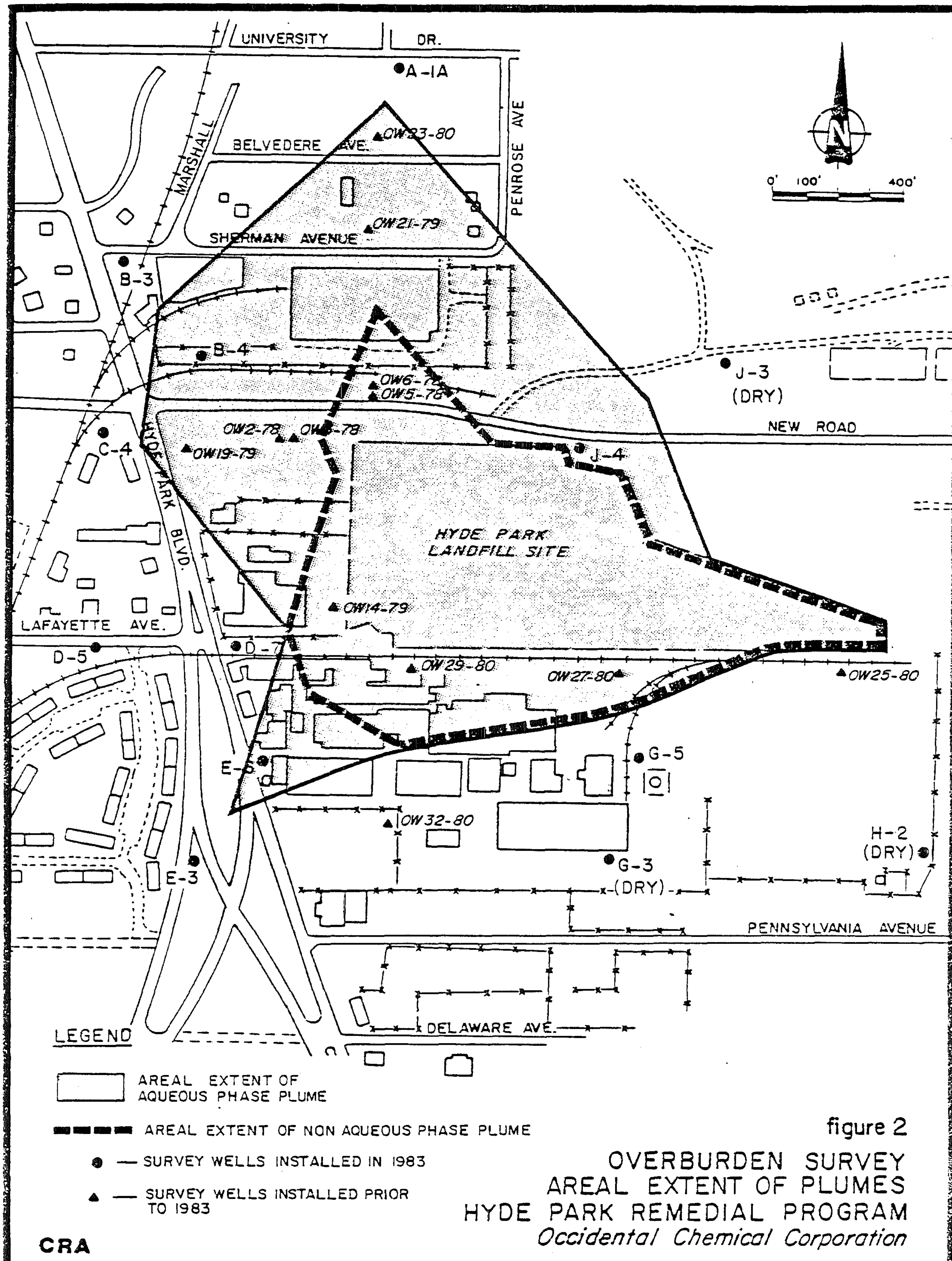


figure 2

OVERBURDEN SURVEY
AREAL EXTENT OF PLUMES
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

1069-21/03/84

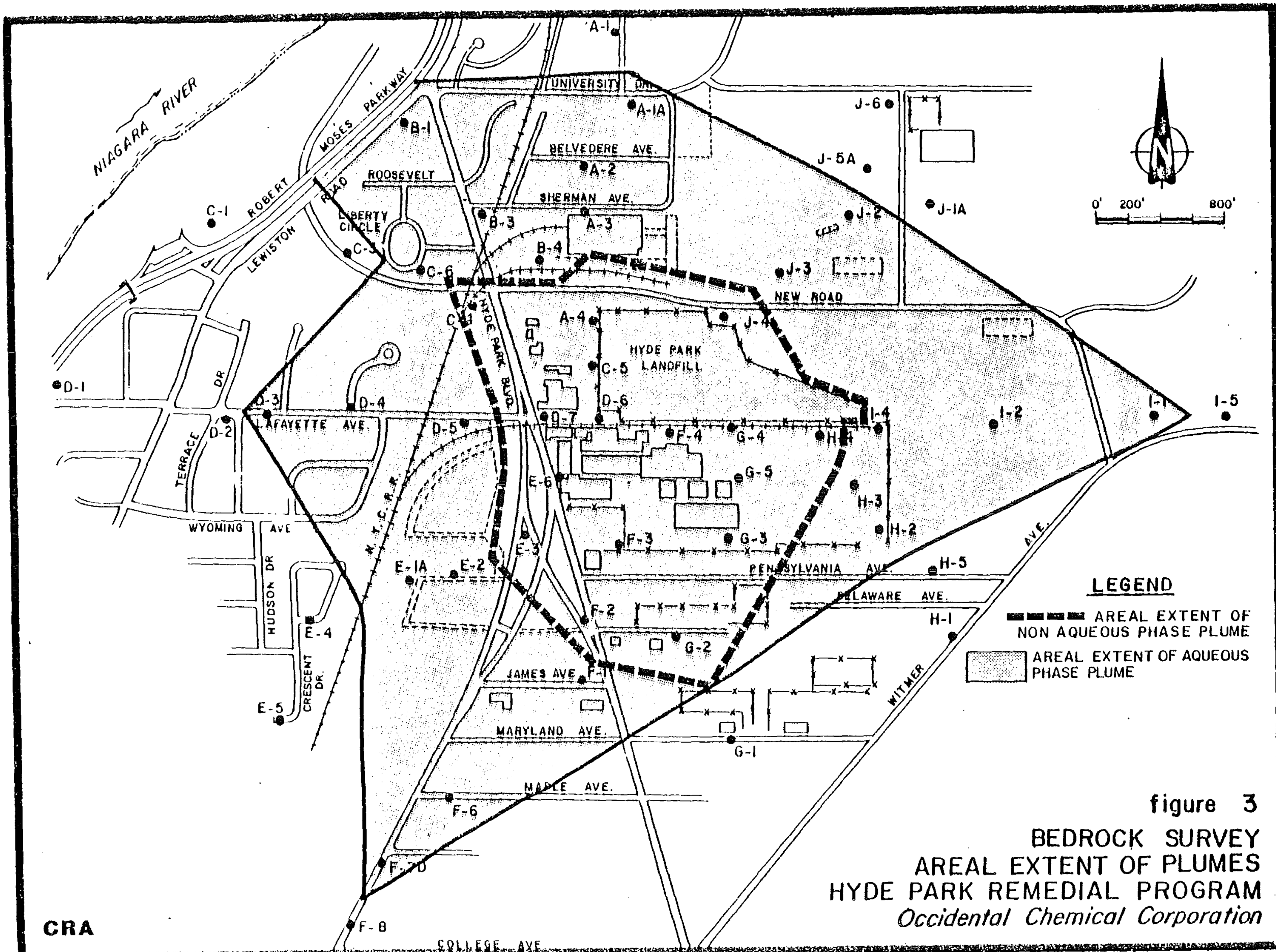


figure 3

BEDROCK SURVEY
AREAL EXTENT OF PLUMES
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

2) March 8, 1984 - Meeting to address EPA/STATE comments regarding the Pump Tests.

Based on the results of the surveys, tests and studies, the areal extent of the plumes of aqueous and non-aqueous phase chemicals in the overburden and bedrock were defined. The plume limits are presented in Figures 2 and 3 respectively.

Subparagraph C(7)(b) of Addendum I of the Settlement Agreement defines the study that shall be initiated "to determine what Requisite Remedial Technology, if any, is required to address the plume composed of chemicals which have migrated from the Landfill Site".

This report is the Requisite Remedial Technology Study conducted to address the plumes from the Hyde Park Landfill. The report concludes by recommending the appropriate remedial technology.

2.0 SUMMARY

The purpose of this Requisite Remedial Technology (RRT) Study is to determine what, if any, action is required to address the identified chemical plumes from the Hyde Park Landfill. Through the studies conducted to date, the extent of the plumes have been identified and some aquifer ~~characterization~~ has been determined in the overburden and the ~~bedrock.~~

2.1 OVERBURDEN

The proposed RRT plan for the overburden regime involves the installation of an overburden barrier-collection system to hydraulically control the migration of chemicals. The system is essentially the same as the Overburden Barrier Collection System (OBCS) described in the Settlement Agreement. It is to be noted that the design of the proposed overburden collection system will take account of the results of future surveys and testing programs for the swales and underground utilities in the area.

2.2 BEDROCK

Data collected to date define the extent of the aqueous phase liquid (APL) and non-aqueous phase liquid (NAPL) plumes from the landfill in the Lockport Formation. More than 99 percent of the chemical loading in the bedrock is contained in the NAPL plume.

Taking account of a benefits analysis and the cost of remedial action to address the APL plume, it is concluded that such technology is not "requisite" and is not required to satisfy the goal of the Settlement Agreement.

An extensive survey of technologies to address the NAPL plume shows that the only promising technologies which may satisfy the standards of the Settlement Agreement are hydraulic containment (purge wells) and physical containment (bedrock grouting). The performance of either of these technologies could further be enhanced by use of supplemental technologies such as bedrock fracturing and recirculation wells.

In developing the final design of a technology to address the NAPL plume in the bedrock, it is apparent that although the data base of hydrogeologic information collected during the site investigation is extensive, supplementation of the data base is required. For example, the data base should be augmented by further knowledge regarding the flow characteristics and behavior of NAPL in the bedrock. This need was evidenced by the pump tests which demonstrated that NAPL has a significant impact on groundwater flow through the prevailing bedrock environment. However, further data collection should not be done by additional short term testing which is considered to be too dependent on the immediate testing area to be of significant additional value in the assessment of the overall performance of a proposed RRT containment system. Furthermore, such short term testing would delay the implementation of a remedial plan. - suit [illegible]

In order to initiate NAPL plume containment and supply valuable data for finalizing the design of an effective remedial plan, it is recommended that the NAPL in bedrock be addressed as follows:

1) install a remedial program initially consisting of prototype purge wells and a test section of bedrock grouting,

2) evaluate the performance of the above installations to develop design criteria,

3) iteratively add to the prototype wells and/or grout until an effective remedial program has been achieved.

Modelling of the above installations suggests that during pumping of the prototype purge wells, an effective hydraulic containment of the NAPL plume can be achieved.

The iterative installation and evaluation of remedial technologies will allow the development of design and monitoring criteria to ultimately ensure that an effective remedial program has been implemented.

The following points should also be noted:

1) Installation of a remedial program to address the entire depth of the Lockport Formation will render redundant the Bedrock Barrier Collection System (BBCS) defined in the Settlement Agreement which addresses only the upper 15 feet of the bedrock. Therefore, the BBCS should not be installed.

2) Implementation of any remedial program at the landfill is dependent upon a permanent leachate storage and handling facility, an on-site aqueous liquid treatment facility and off-site non-aqueous liquid disposal. These activities require local and EPA permits which OCC have not yet received.

3.0 DATA BASE

The hydrogeologic conditions of the area surrounding the Hyde Park Landfill Site have been investigated through a number of studies. The first major study was conducted in conjunction with the construction program for the Robert Moses Generating Station. This study involved the mapping and definition of the geologic stratigraphy and groundwater flow regimes along the alignment of the waterway conduits constructed to provide water from the Upper Niagara River to the power project. This report was prepared in 1964 and is entitled "Groundwater in the Niagara Falls Area, New York" by Richard H. Johnston (USGS).

Several site specific investigations were conducted by OCC in the late 1970's and early 1980's to collect information regarding chemical migration and hydrogeologic conditions local to the site. The collected information provided a preliminary site specific data base.

The Settlement Agreement specified additional surveys, tests and studies to be performed to further define and characterize hydrogeologic conditions and the extent of chemical plumes in the overburden and bedrock regimes. The information collected during these surveys, tests and studies at Hyde Park are presented in the following reports:

1) "Hyde Park-Bloody Run Aquifer Survey and Testing Program", October 3, 1983 by Conestoga-Rovers & Associates Limited.

2) "Pump Well Installation and Pump Test Results, Hyde Park Remedial Program", December 1983 by Conestoga-Rovers & Associates Limited.

The above two reports provide the major data base used for the Requisite Remedial Technology Study to develop the proposed remedial program.

Another significant study of the bedrock formation involved the review of aerial photographs, bedrock cores and a field survey to evaluate the bedrock's structural control features. The results of this study, including suspected lineament features, were presented in the report entitled, "Lockport Formation Jointing at the Hyde Park Landfill Site - March 29, 1983" by Woodward-Clyde Consultants.

An assessment has been undertaken by Arthur D. Little to determine the "anticipated benefits to the public health and the environment of any action or inaction recommended in the (RRT) study". The calculations and studies of Arthur D. Little leading to the major conclusions included in the enclosed report have been completed. However, the final report does not yet contain all of the reference and supporting documentation which, as a result of discussions regarding the Bloody Run Risk Assessment are recognized to be an integral part of any comprehensive report. Once this has been added, the report will be submitted to EPA/STATE. The title of the report is "Hyde Park Remedial Program - Benefits of Proposed Requisite Remedial Technology - Overburden and Bedrock" (Benefits Report).

4.0 ALTERNATE REMEDIAL PROGRAM AND TECHNOLOGY REVIEW

The assessment of risk to public health and the environment at the Hyde Park Site identify the following chemical sources requiring remediation:

1) Landfill
2) NAPL and APL in the overburden
3) NAPL and associated APL within the NAPL plume in the bedrock.

As detailed in the Benefits Report, it is estimated that over 99 percent of the chemical loading in the bedrock from the landfill is contained in the NAPL plume. The Benefits Report concludes that the presence and continued migration of the APL outside the NAPL plume in the bedrock does not pose a significant risk to the public health and the environment. The report also concludes that implementation of remedial technology to address the APL plume could adversely impact the environment and public health. In light of these factors and the cost of implementing remedial technology to address the APL plume as specified in Paragraph 4 of the Settlement Agreement, it is concluded that remedial technology is not "requisite" and is unnecessary to satisfy the goal of the Settlement Agreement.

Table 1 identifies remedial programs evaluated in the RRT study to address each significant chemical source. Table 2 identifies the remedial technologies evaluated for each remedial program and shows the remedial technologies deemed to be unsuitable. Table 3 presents the reasons particular technologies were deemed to be unsuitable. Those technologies which required further evaluation in order to determine their suitability are also identified in Table 3.

It is to be noted that those remedial technologies deemed to be suitable by inclusion in the Settlement Agreement are not discussed further in this report. These technologies include the following:

(1) existing barrier collection system
(2) capping (clay, concrete, asphalt, synthetic membrane)
(3) surface drainage control (ditches, swales, grading, pipes, culverts)
(4) treatment of collected liquids
(5) partial excavation to accomodate capping

TABLE 1

ALTERNATE REMEDIAL PROGRAMS

Chemical Source	Remedial Program
Landfill	- hydraulic containment - physical containment - relocation - treatment
NAPL and APL in Overburden	- hydraulic containment - physical containment - relocation - treatment
NAPL and APL in Bedrock within NAPL plume	- hydraulic containment - physical containment - relocation - treatment

TABLE 2

ALTERNATE REMEDIAL TECHNOLOGIES

Remedial Program	Remedial Technologies Considered, Found Suitable and Evaluated	Remedial Technologies Considered and Found Unsuitable
Landfill		
a) Hydraulic containment	- existing barrier collection system - capping (clay, concrete, asphalt, synthetic membrane) - NAPL and APL containment in bedrock (see remedial technologies for NAPL and APL in bedrock) - overburden barrier collection system (tile)	- extraction wells - enhanced recovery wells (recirculation, chemical injection and heat injection) - augment existing barrier collection system
b) Physical containment	- capping (clay, concrete, asphalt, synthetic membrane) - surface drainage control (ditches, swales, grading, pipes, culverts)	- grout base why - continuous barrier wall around landfill in overburden (bentonite, clay, cement, asphalt, designed backfill) - pressure grouting in bedrock around landfill - fixation/stabilization in place
c) Relocation		- excavation
d) Extraction & Treatment	- biological treatment of liquids - chemical treatment of liquids - physical treatment of liquids (separation, incineration, activated carbon, filtration) - fixation/stabilization of collected liquids with landfilling	- landfilling of excavated site - biological treatment of liquids in situ - chemical treatment of liquids in situ

TABLE 2 (cont'd)

ALTERNATE REMEDIAL TECHNOLOGIES

Remedial Program	Remedial Technologies Considered, Found Suitable and Evaluated	Remedial Technologies Considered and Found Unsuitable
NAPL & APL in Bedrock		
a) Hydraulic containment	- extraction wells - enhanced recovery wells (recirculation wells) - bedrock fracturing (hydrofracturing and drilling)	- enhanced recovery wells (chemical injection and heat injection) - proposed bedrock barrier collection system - bedrock fracturing (blasting) - tile collection system
b) Physical containment	- pressure grouting (grout curtain wall, grouting through controlled fractures and defined fracture grouting using singularly or combination of bentonite, cement, fly ash, sodium silicates and chemicals)	- fixation/stabilization in place - continuous barrier wall (bentonite, clay, cement, asphalt, designed backfill)
c) Relocation		- excavation of all bedrock within NAPL plume
d) Extraction & Treatment	- biological treatment of liquids - chemical treatment of liquids - physical treatment of liquids (separation, incineration, activated carbon, filtration) - fixation/stabilization of collected liquids with landfilling	- landfilling of bedrock - biological treatment of liquids in situ - chemical treatment of liquids in situ

TABLE 2 (cont'd)

ALTERNATE REMEDIAL TECHNOLOGIES

Remedial Program	Remedial Technologies Considered, Found Suitable and Evaluated	Remedial Technologies Considered and Found Unsuitable
NAPL & APL in Overburden		
a) Hydraulic containment - surface & subsurface	- overburden barrier collection system (tile) - existing barrier collection system - capping	- extraction wells - enhanced recovery wells (recirculation, chemical injection and heat injection) - augment existing barrier collection system
b) Physical containment 1) Surface	- capping (clay, concrete, asphalt, synthetic membrane) - surface drainage control (ditches, swales, grading, pipes, culverts)	- fixation/stabilization in place
2) Subsurface		- surface capping (clay, concrete, asphalt, synthetic membrane) - continuous barrier wall (bentonite, clay, cement, asphalt, designed backfill) - fixation/stabilization in place
c) Relocation 1) Surface	- partial excavation to accommodate capping	- complete excavation
2) Subsurface		- excavation
d) Extraction & Treatment - surface & subsurface	- biological treatment of liquids - chemical treatment of liquids - physical treatment of liquids (separation, incineration, activated carbon, filtration) - fi[illegible]on/s[illegible]ili[illegible]on [illegible] [illegible]oll[illegible]d liquids with landfilling	- landfilling of excavated site - biological treatment of liquids in situ - chemical treatment of liquids in situ

TABLE 3

UNSUITABLE REMEDIAL TECHNOLOGIES

Technology	Discussion
* Further discussion of these alternatives are presented in the appropriate category of Section 8.0 of this report.	
Landfill	
- extraction wells*	- unsuitable hydraulic characteristics of landfill - redundant since it addresses only a portion of chemical source
- enhanced recovery wells	- unsuitable hydraulic characteristics of landfill - environmental risk due to unknown reactions - redundant since it addresses only a portion of chemical source
- augmented existing barrier collection system	- existing system not amenable to augmentation to address chemicals at depth - health and environmental risk of excavation into landfill - redundant since it addresses only a portion of chemical source
- grout base	- redundant since it addresses only a portion of chemical source - high cost - health and environmental risk due to extensive drilling through landfill
- continuous barrier wall around landfill in overburden	- made redundant by required installation of Overburden Barrier Collection System
- pressure grouting in bedrock around landfill	- redundant since it addresses only a portion of chemical source
- fixation/stabiliation in place	- unsuitable hydraulic characteristics of landfill - technology not available

TABLE 3 (cont'd)

UNSUITABLE REMEDIAL TECHNOLOGIES

Technology	Discussion
Landfill (cont'd)	
- excavation/landfilling*	- redundant since it addresses only a portion of chemical source - high cost - health and environmental risk - secure landfill availability
- biological treatment of liquids in situ	- technology not available
- chemical treatment of liquids in situ	- technology not available
NAPL & APL in Overburden	
- extraction wells	- unsuitable hydraulic characteristics
- enhanced recovery wells	- unsuitable hydraulic characteristics
- augment existing barrier collection system	- existing system not amenable to address chemicals at depth - made redundant by required installation of Overburden Barrier Collection System
- continuous barrier wall (surface & subsurface)	- made redundant by required installation of Overburden Barrier Collection System
- fixation/stabilization in place (surface & subsurface)	- unsuitable hydraulic characteristics of overburden - technology not available - made redundant by required installation of Overburden Barrier Collection System

TABLE 3 (cont'd)

UNSUITABLE REMEDIAL TECHNOLOGIES

Technology	Discussion
NAPL & APL in Overburden (cont'd)	
- capping (subsurface)	- existing clean surface soil provides suitable cap
- complete excavation (surface)	- high cost - redundant since it addresses only a portion of chemical source - made redundant by required installation of Overburden Barrier Collection System
- excavation/landfilling* (subsurface)	- high cost - made redundant by required installation of Overburden Barrier Collection System - health and environmental risk - secure landfill availability
- biological treatment of liquids in situ	- technology not available
- chemical treatment of liquids in situ	- technology not available
NAPL & APL in Bedrock	
- enhanced recovery wells* (chemical, heat)	- unknown NAPL rheological characteristics - health and environmental risk due to unknown reactions - technology not available (chemical injection) - high cost (heat injection)
- proposed Bedrock Barrier Collection System*	- redundant since it addresses only a small portion of chemical source in bedrock

TABLE 3 (cont'd)

UNSUITABLE REMEDIAL TECHNOLOGIES

Technology	Discussion
NAPL & APL in Bedrock (cont'd)	
- bedrock fracturing* (blasting)	- health and environmental risk due to chemical releases
- tile collection system*	- high cost - other technologies of equal effect and lower cost available - health and environmental risk due to construction requirements (blasting, dewatering, work environment)
- fixation/stabilization in place	- technology not available
- continuous barrier wall*	- high cost - other technologies of equal effect and lower cost available
- excavation/landfilling*	- high cost - health and environmental risk - secure landfill availability
- biological treatment in situ	- technology not available
- chemical treatment in situ	- technology not available

5.0 OVERBURDEN PLUME CONTAINMENT SYSTEM

In accordance with the Settlement Agreement, an evaluation of the data including an assessment of the overburden groundwater characteristics, the presence of APL and NAPL in the overburden regime and appropriate remedial alternatives to address the identified plumes has been undertaken.

5.1 OVERBURDEN HYDROGEOLOGIC CONDITIONS

The overburden in the Hyde Park area was observed to be interbedded glaciolacustrine deposits of predominantly silty clay and clayey silt ranging in thickness from 1.4 feet to in excess of 34 feet. Occasional thin beds of silt and fine sand were observed. A red-brown silty fine sand till was typically encountered immediately overlying the bedrock.

As a result of the horizontally stratified fine grained overburden, significant groundwater movement is not expected in the horizontal direction over the majority of the overburden depth. The possible exception to this could be within the till immediately overlying the bedrock. Due to the horizontal stratification of the overburden, significantly less movement is anticipated in the vertical direction.

Slow groundwater movement in the overburden was verified by the field test data. Most of the installed wells in the overburden are unable to provide more than one well volume of groundwater at a time without allowing considerable time for recovery. Further evidence of the poor groundwater transmissivity characteristics of the overburden regime are the number of wells in the survey which were unable to provide enough water for sample collection. These wells included G-3, H-2 and J-3. Recovery tests of wells installed in the overburden in 1978, indicated permeabilities ranging from 8.7×10^{-5} cm/sec to 1.6×10^{-7} cm/sec. The results of these recovery tests performed in November 1978 are presented in Table 4.

Using groundwater measurements collected during the regular monitoring program at Hyde Park, groundwater contour maps of the overburden and overburden/bedrock interface have been generated. Figures 4 and 5 respectively present the groundwater contours as measured on December 21, 1983. Both Figures 4 and 5 show that overburden groundwater flow is to the west and is influenced to some degree by groundwater mounding in the site.

TABLE 4

CALCULATED OVERBURDEN PERMEABILITIES

HYDE PARK LANDFILL

Well No.	Average K (Permeability Factor) (cm/sec)	Strata	Equation Used
OW2-78	7.0×10^{-7}	Overburden	1
OW3-78	4.9×10^{-7}	Interface	2
OW5-78	7.3×10^{-5}	Interface	2
OW6-78	9.4×10^{-5}	Overburden	1
OW8-78	9.8×10^{-6}	Interface	2
OW9-78	8.9×10^{-7}	Overburden	1
OW10-78	6.0×10^{-6}	Interface	2

Note: Permeability, K, calculated using one of the following variable head equations:

1) $$K = \frac{R^2{}_2}{2L\ (t_2 - t_1)} \ln\left(\frac{L}{R_2}\right) \ln\left(\frac{H_1}{H_2}\right)$$

2) $$K = \frac{t\ {}^{R}{}_1}{4\ (t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right) \left(\frac{A_2}{A_1}\right)$$

TABLE 4 (Cont'd)

CALCULATED OVERBURDEN PERMEABILITIES

HYDE PARK LANDFILL

Where R_1 = radius of augered hole (cm)

Where R_2 = radius of pipe (cm)

Where A_1 = area of augered hole (cm^2)

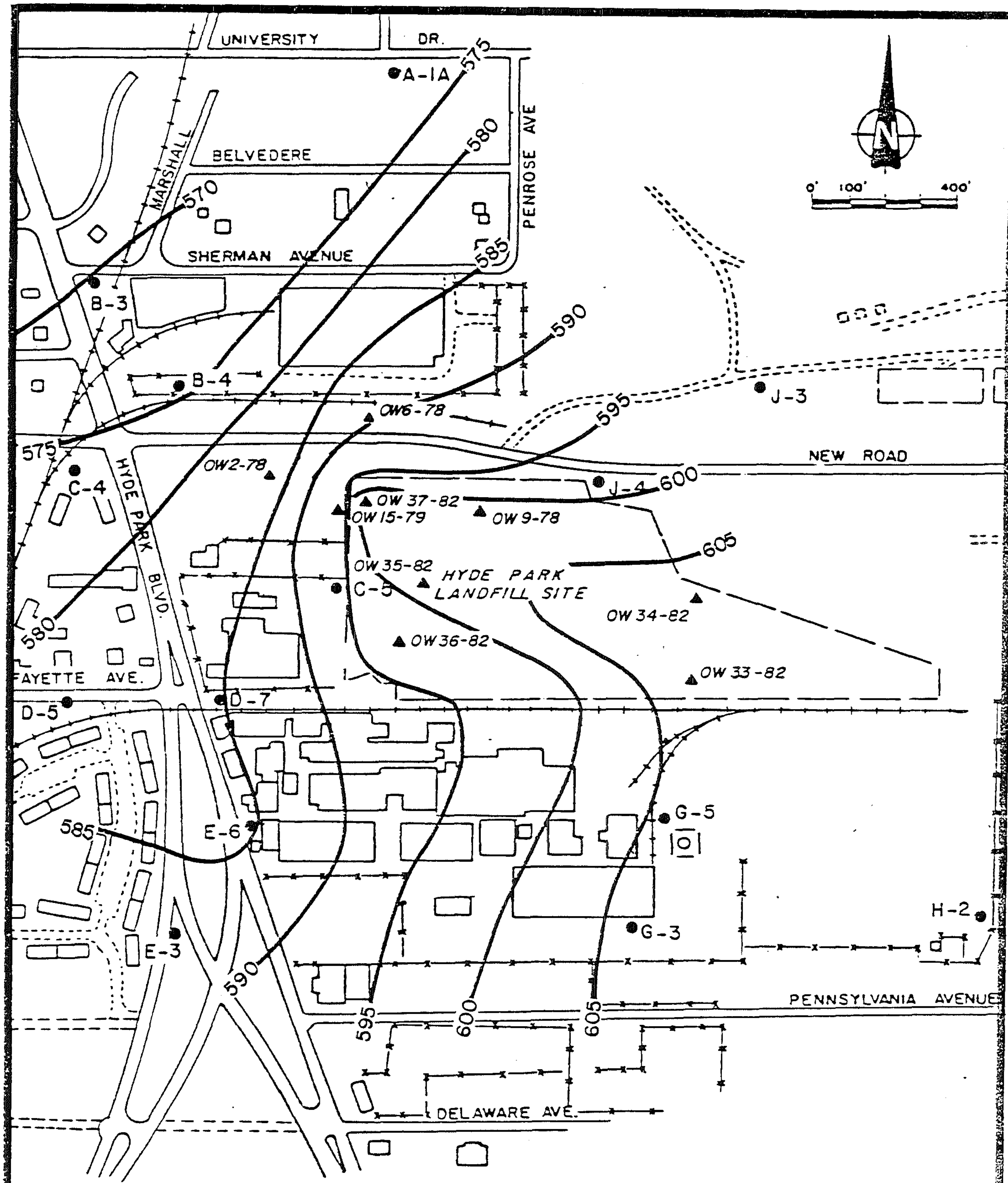
Where A_2 = area of pipe (cm^2)

Where L = sand pack length

Where H_1 = depth below static water level at time t_1

Where H_2 = depth below static water level at time t_2

REFERENCE: Design Manual - Soil Mechanics, Foundations and Earth Structures
NAVFAC DM-7 March 1971



LEGEND

▲ SURVEY WELLS INSTALLED PRIOR TO 1983

● SURVEY WELLS INSTALLED IN 1983

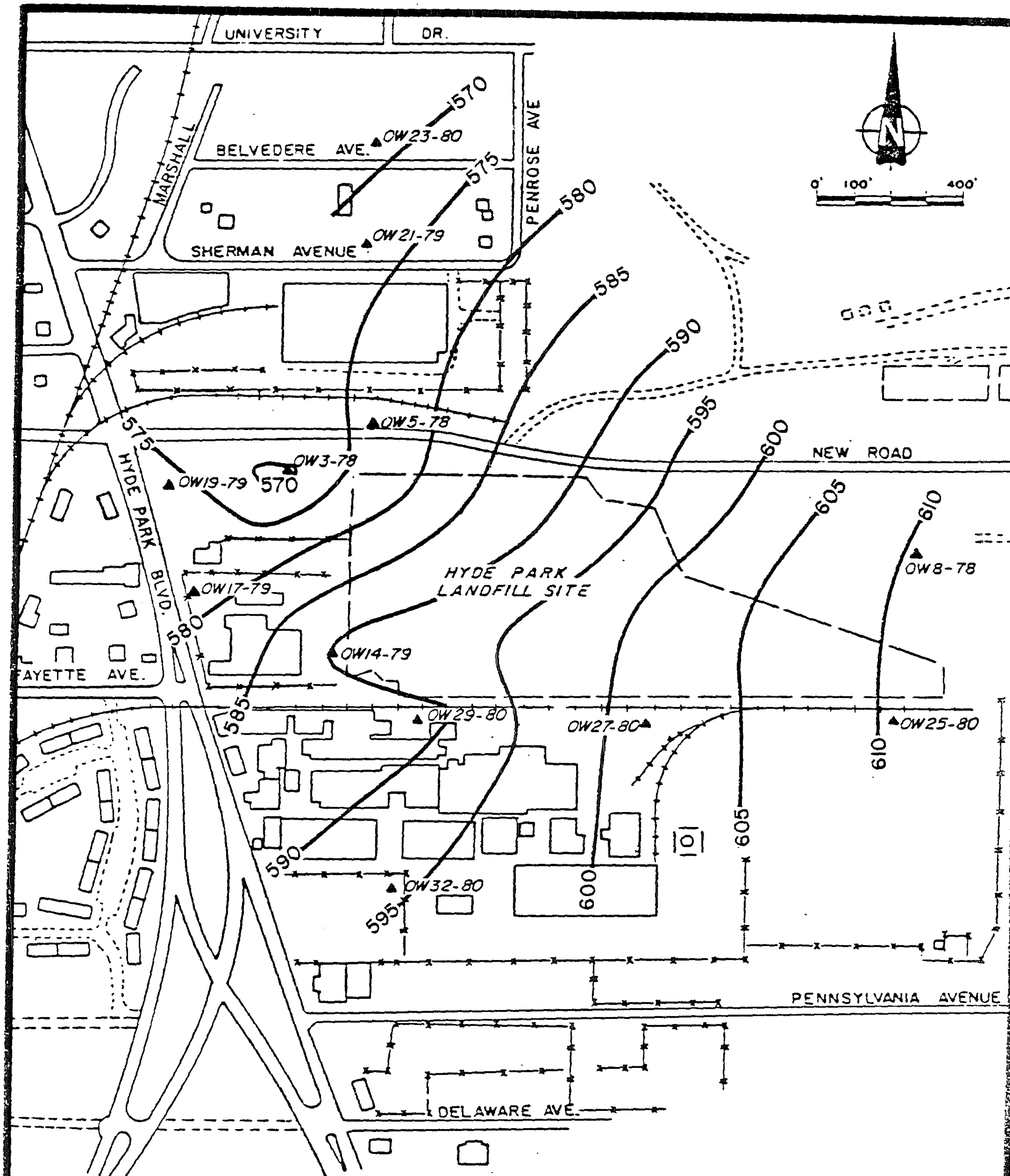
—— GROUNDWATER CONTOURS (USGS DATUM)

CRA

figure 4

OVERBURDEN GROUNDWATER CONTOURS AS MEASURED DECEMBER 21, 1983
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

1069-21/03/84



LEGEND

▲ SURVEY WELLS

GROUNDWATER CONTOURS (USGS DATUM)

CRA

figure 5

OVERBURDEN / BEDROCK
INTERFACE GROUNDWATER CONTOURS
AS MEASURED DECEMBER 21, 1983
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

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5.2 OVERBURDEN PLUME DEFINITION

Chemical plumes in the overburden were defined by the data obtained during the Hyde Park off-site overburden aquifer survey. These plumes are identified in Figure 2.

Due to the interconnection between the groundwater in the bedrock and till, it would be difficult to establish whether any chemicals identified in the overburden actually migrated through the overburden or are the result of the overburden aquifer's connection to the plume in the bedrock immediately beneath the till. In those areas where chemicals are found in the overburden at some distance from the landfill, cross-contamination from the bedrock to the overburden well during development and sampling probably occurred. The overburden materials as previously discussed are not conducive to extensive horizontal migration. The mapping of the plumes in the overburden does not take account of such a possibility.

The analytical results of the overburden wells indicate that the overburden APL plume is defined by wells OW 3-78, OW 5-78, OW 6-78, OW 14-79, OW 19-79, OW 21-79, OW 23-80, OW27-80, OW 29-80 B-4, E-6 and J-4. It is to be noted that at each of the wells where the only parameter(s) in excess of the plume definition limits was either TOH and/or phenol (OW19-79, OW21-79, OW23-80, B-4 and E-6), the top 15 foot interval of the bedrock near the overburden well also exhibited elevated TOH and phenol concentrations. This relationship is presented in Table 5. It is also to be noted that the well screen of three of the five wells (OW 19-79, OW21-79 and B-4) are set in the till overlying the bedrock. One of the wells (E-6) is within 0.2 feet of the observed till strata and the fifth well (OW 23-80) is in contact with the bedrock. As discussed previously, induced flow from the bedrock during sampling of the overburden well is concluded to be the cause of measured levels of chemicals in the overburden at these outer wells.

The plume of NAPL was identified from a review of the soil logs and groundwater quality information collected during the aquifer survey and previous drilling programs in the area. Generally the presence of NAPL in the overburden is restricted to the area immediately surrounding the landfill site. This is as expected because of the generally fine grained nature of the overburden soils which would restrict NAPL migration. The presence of NAPL was observed in only one of the groundwater samples obtained from an overburden survey well, namely OW6-78, which was

TABLE 5

OVERBURDEN/BEDROCK GROUNDWATER QUALITY COMPARISON

Overburden Well			Adjacent Bedrock Well Top 15' Interval		
Well No.	TOH (mg/l)	Phenol (mg/l)	Well No.	TOH (mg/l)	Phenol (mg/l)
OW 19-79	50	ND	C-4	9.73	9.8
			C-5	31.76	30.0
OW 21-79	1.63	0.03	A-3	3.28	0.02
OW 23-80	0.7	ND	A-2	6.15	ND
B-4	11.88	0.68	B-4	20.	56.
E-6	65.15	ND	D-6	50.	4200
			E-3	10.65	0.5

installed to a depth of 15.2 feet below the ground surface. During the drilling programs at Hyde Park, NAPL was observed in the overburden soil at the following borehole and well locations: OW 4-78, OW 13-79, BH 24-79 (8' south of OW 15-79), A-4, C-5, F-4 and G-5.

5.3 OVERBURDEN REMEDIAL PLAN

The remedial plan to contain chemicals in the overburden plumes will be based on the construction of an Overburden Barrier-Collection System (OBCS) and bedrock containment system (see Section 9.0).

The proposed OBCS is designed to address by an inward gradient, the NAPL plume and will take account of future surveys and testing programs for the swales and underground utilities in the area.

The APL plume in the overburden will be addressed by the induced horizontal gradient to the OBCS and the downward gradient induced by the bedrock containment system.

If it is determined through monitoring (see Section 5.4) in the overburden that the proposed OBCS combined with the bedrock containment system are not effective in hydraulically containing the overburden plumes, the need for further remedial action will be evaluated for those specific areas.

The alignment and grade of the proposed OBCS will remain generally the same as originally conceptualized in the Settlement Agreement except for the following:

1) The north section of the collection system will follow parallel to the north edge of the high voltage electric transmission lines so as to reduce interference with power lines during construction.

2) The northwest corner has been extended outward to address the NAPL.

The proposed alignment and profile of the OBCS are presented in Figures 6 through 9. As has been noted, the data base from the future survey of swales and underground utilities will be considered in the final design. Any modification is not expected to entail more than an extension or relocation of the proposed system.

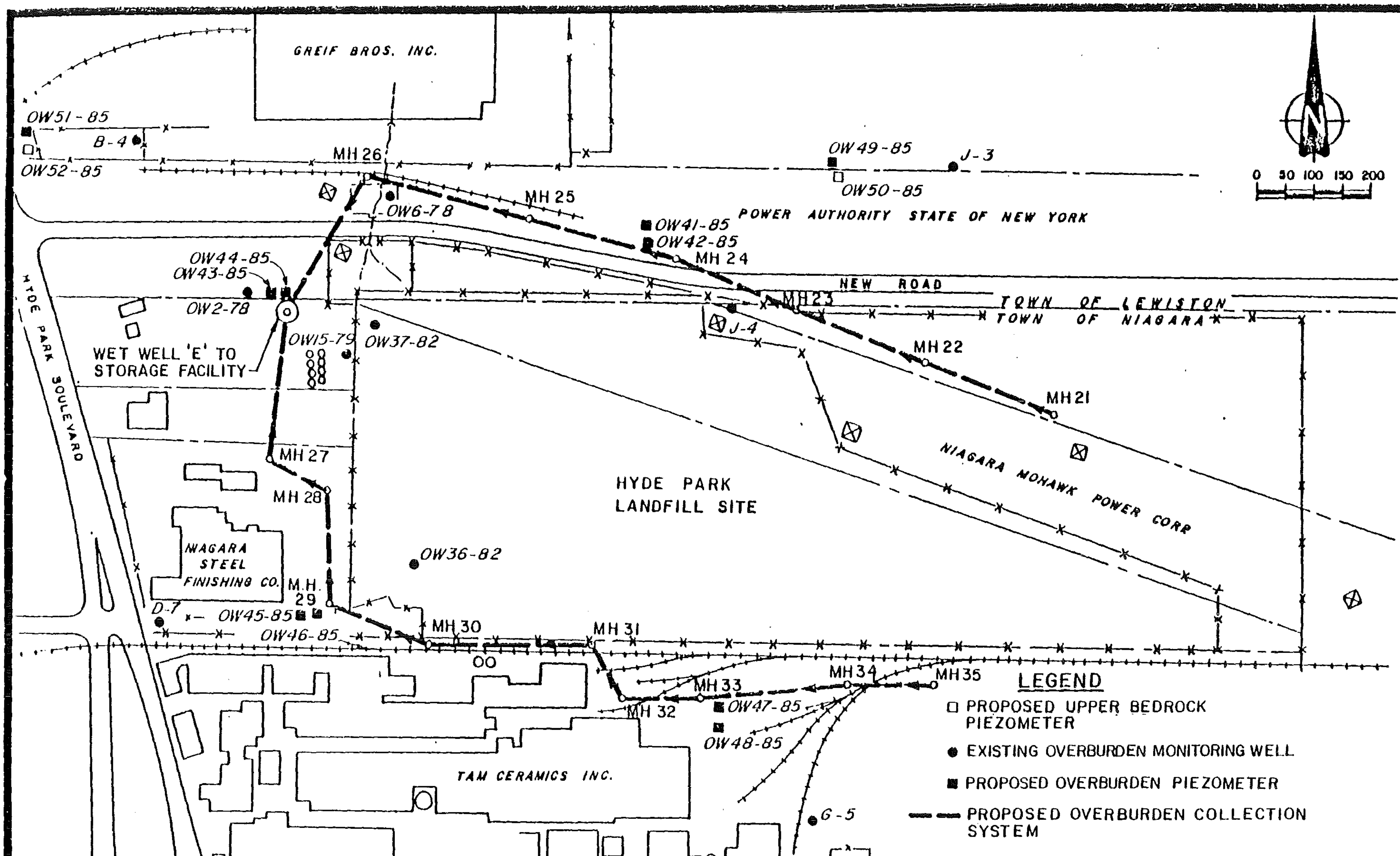

figure 6

OVERBURDEN BARRIER-COLLECTION SYSTEM
PLAN VIEW
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

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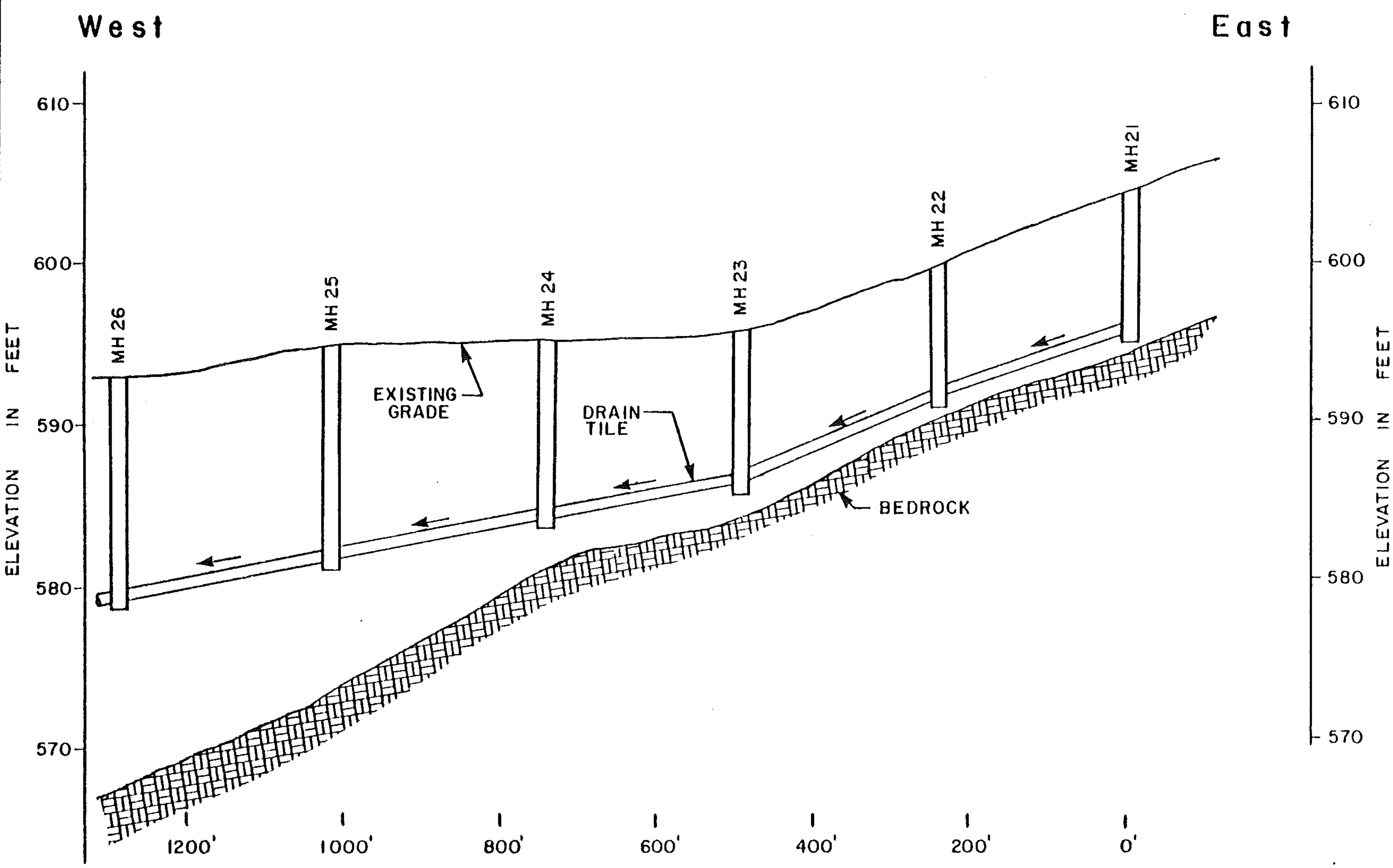

figure 7

OVERBURDEN BARRIER - COLLECTION SYSTEM
NORTH PROFILE
HYDE PARK REMEDIAL PROGRAM
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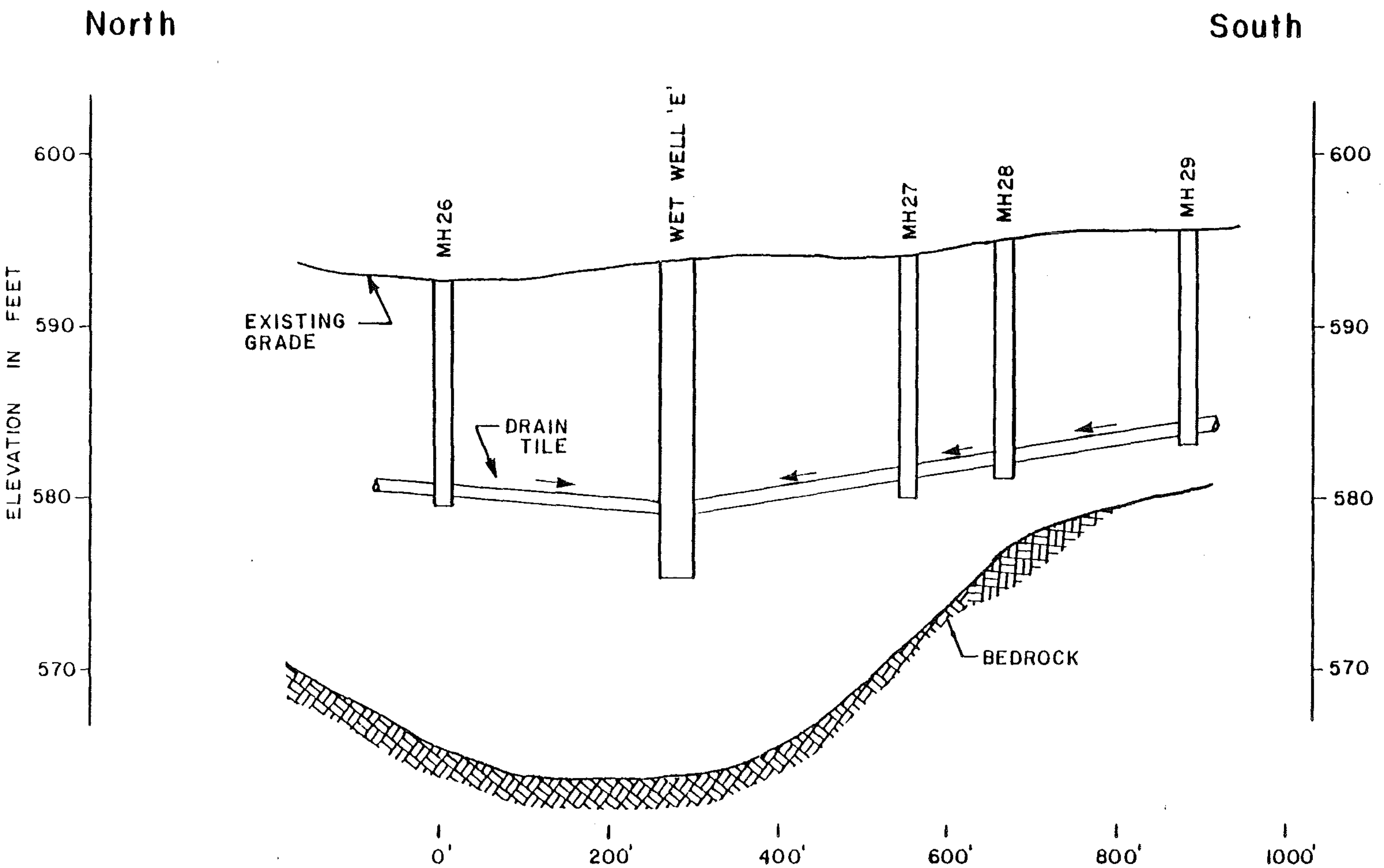

figure 8

OVERBURDEN BARRIER - COLLECTION SYSTEM
WEST PROFILE
HYDE PARK REMEDIAL PROGRAM
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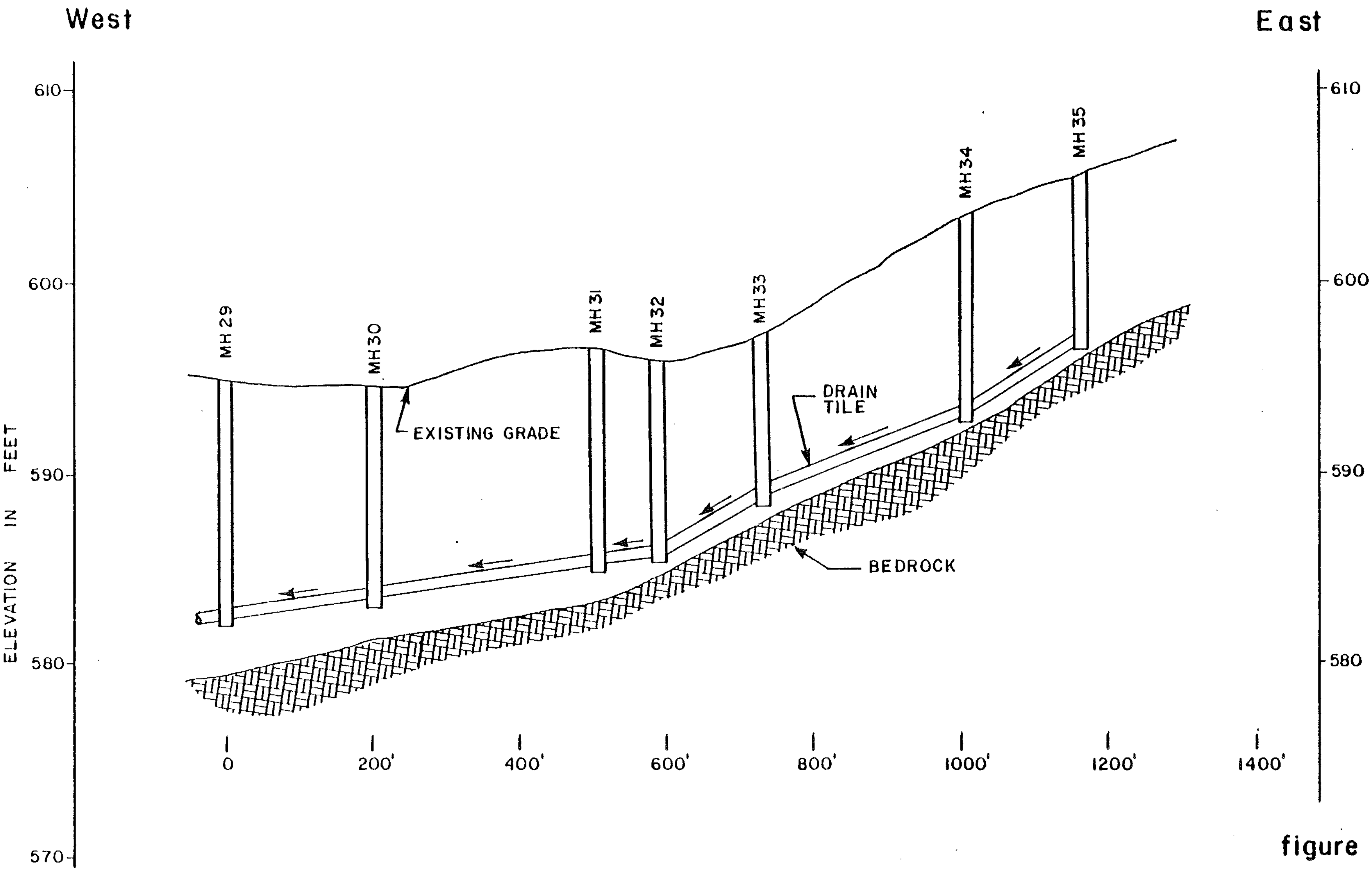

figure 9

OVERBURDEN BARRIER - COLLECTION SYSTEM
SOUTH PROFILE
HYDE PARK REMEDIAL PROGRAM
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ecology and environment

The estimated cost of the OBCS is presented in Appendix A.

5.4 MONITORING PROGRAM

The determination of the effectiveness of the overburden plume containment plan is based on the ability to show that the hydraulic gradient within the plumes is toward the OBCS or bedrock. In order to determine the hydraulic gradient impact of the OBCS or bedrock containment system, eight (8) piezometers will be installed in pairs adjacent to the proposed OBCS and six (6) additional piezometers will be installed in pairs on the perimeter of the APL plume. In each pair of piezometers on the APL plume perimeter, one piezometer will be installed in the overburden and one piezometer will be installed into the upper bedrock. The proposed piezometer locations are identified on Figure 6.

Groundwater measurements will be taken from the newly installed piezometers and selected existing wells as shown on Figure 6. The frequency of measurements shall be as follows:

a) monthly during the first year, and

b) quarterly thereafter

The hydraulic data will be evaluated to determine the effectiveness of the OBCS and bedrock containment system with time.

Each overburden piezometer will be constructed as follows:

a) Collect continuous split spoon samples in advance of the augering operation. The installed depth will be equivalent to the depth of the OBCS adjacent to the piezometer.
b) The piezometer will be inserted into the open borehole and backfilled with a measured quartzite sandpack to within 4 feet of the ground surface. The piezometer will consist of a 2 inch diameter (5 foot long) stainless steel well screen coupled to 2 inch diameter black steel riser pipe.
c) A one foot bentonite plug will be placed over the sandpack and the remaining annular space will be backfilled to within 6 inches of the ground surface with cement grout. The final 6 inches of backfill material will be native soil.
d) The piezometer will be secured with a lockable cap.

A typical overburden piezometer installation is presented in Figure 10.

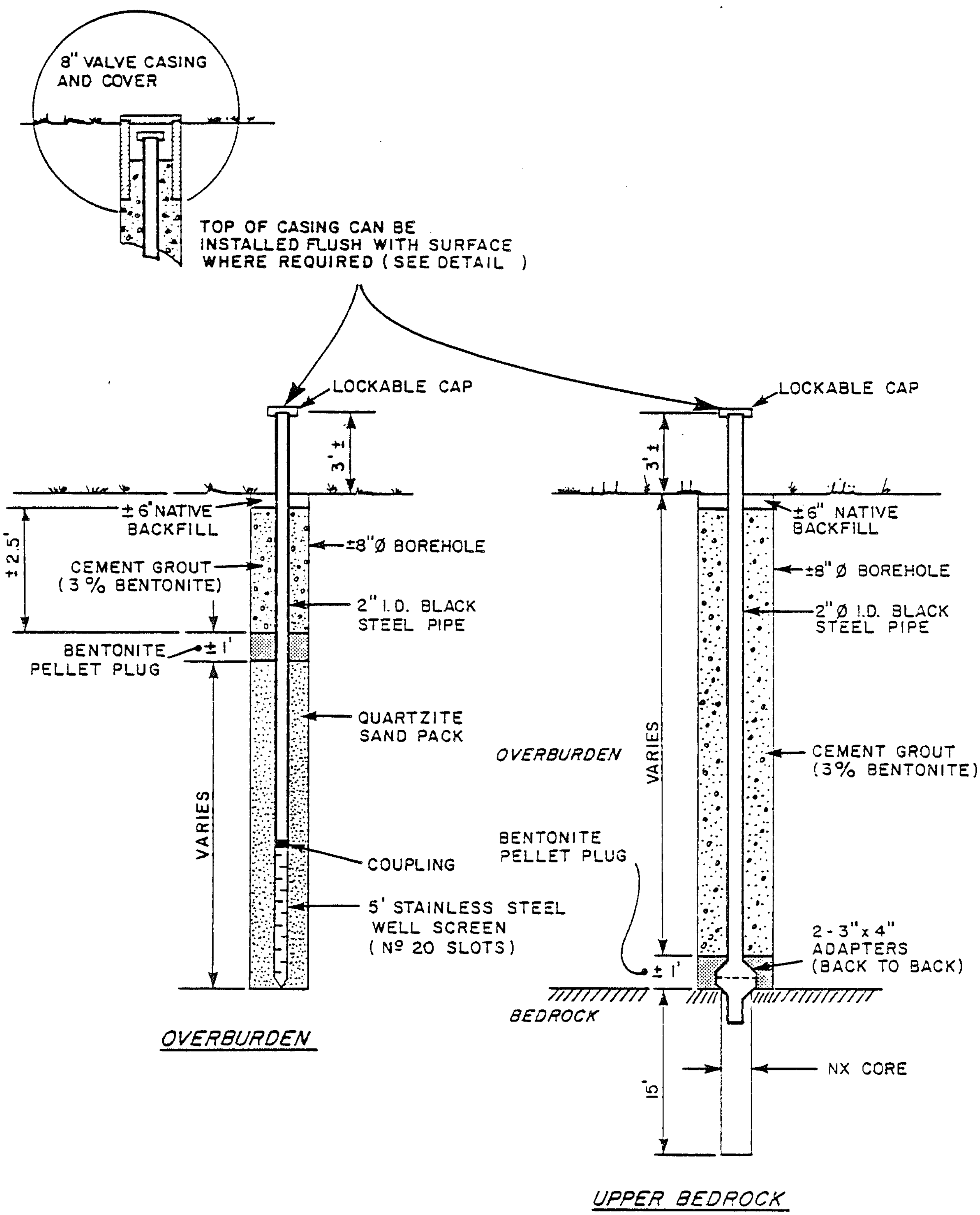

figure 10

TYPICAL
PIEZOMETER INSTALLATION
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

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Each bedrock piezometer will be constructed as follows:

a) Auger through overburden to top of bedrock.

b) Remove the auger and install a permanent steel casing into the open borehole.

c) Seal the annular space between the borehole wall and casing with cement grout.

d) Core with an NX diamond drill bit to a depth of 15 feet into the bedrock.

A typical bedrock piezometer installation is presented in Figure 10.

6.0 BEDROCK AQUIFER CHARACTERISTICS

6.1 DATA BASE

The hydrogeology of the Hyde Park area has been documented in previous reports, i.e. Johnston (1964), Maslia and Johnston (1982) and Hyde Park - Bloody Run Aquifer Survey and Testing Program prepared for Occidental Chemical Corporation by CRA. As previously discussed, the overburden stratigraphy consists of silty-clay to clayey silt overlying the Lockport Formation. The Lockport Dolomite is composed of five members and is approximately 100 feet thick in the Hyde Park area. The Lockport Dolomite is underlain by the Rochester Shale Formation.

In November 1983, three pump tests were conducted in the Lockport Formation to further evaluate the bedrock aquifer characteristics in the Hyde Park area. The results of these tests were presented in the report entitled "Pump Well Installation and Pump Test Results" - December 1983 by CRA.

Postulated cones of influence for pump tests C-5A, D-6 and F-4A are presented on Figures 11, 12 and 13 respectively. The cones of influence were necessarily postulated because 1) cones of influence were unsymmetrical and 2) the effect of NAPL on the aqueous flow regime i.e. blockage of connections in the fractured aquifer, is unknown.

Significant NAPL interference on the flow characteristics was postulated from the pump test data. For example, during the pump testing of well C-5A, water elevations at wells F-4 and F-4-1 were initially measured to be different by 20 feet. Observed pump test drawdowns in wells F-4 and F-4-1 were 3 and 16 feet respectively. Final water elevations after cessation of the pump test were approximately the same. It is therefore postulated that a localized plug of NAPL in the area of D-6-1 had initially maintained the water table elevation in this area at an elevated level. During the pump test, a sufficient gradient across the plug was created causing the plug to move and allowing the water table in the area of F-4-1 to equilibrate with F-4. Thus, the drawdown at F-4 was felt to be more representative of the aqueous system and was employed in drawing the contours. Similarily, during pump test C-5A, wells D-6-1 and OW28-80 had drawdowns of 7.5 and -0.2 feet respectively. Again it was postulated that a plug of NAPL was producing an artificially elevated water table near OW28-80. Additional data supporting the above hypothesis were observed during pump test D-6.

Initial elevations in D-6-1 and OW28-80 before the start of pump test D-6 were 559.9 and 578.1 feet respectively. The separation distance between the two wells is on the order of 140 feet with D-6-1 closer to D-6. D-6-1 and OW28-80 had drawdowns of 2.2 and 7.0 feet respectively which is inconsistent with the expected results. Final elevations were still 13 feet different. It is postulated that a plug of NAPL was causing aquifer response interference. Therefore, it was concluded that the results of D-6-1 are more representative of the aqueous system. Thus, drawdowns of 7.5, 2.2 and 2.1 feet at D-6-1 were employed for pump tests C-5A, D-6 and F-4A respectively. It is interesting to note that D-6-1 appears to have better hydraulic connections with C-5A (both of which contained NAPL) which is 517 feet distant than with D-6 (NAPL not observed) which is only 51 feet from D-6-1.

6.2 ANALYSES OF PUMP TESTS

6.2.1 Methodology

To obtain preliminary estimates of transmissibility and storage coefficients for input to the flow model, the pump test data was analyzed by the

Cooper-Jacob method. This method is a modification of the Jacob non-equilibrium method to account for variable pumping rates. The Jacob method is itself a modification of the Theis method and has insignificant error when

$$u = 1.87\ r^2\ S/\ Tt < 0.05 \qquad (1)$$

where: r = distance to observation well (ft)

S = storage coefficient

T = transmissibility coefficient (gpd/ft)

t = time since pumping started (days)

For the Jacob method, the coefficient of transmissibility can be estimated from

$$T = \frac{264\ Q}{Ds} \qquad (2)$$

where: Q = pumping rate (gpm)

Ds = difference in drawdown per log cycle of time

When drawdown(s) versus t is plotted on semi-log graphs, a straight line drawn through the points, neglecting the ones that violate the constraint imposed by Equation (1), is used to determine Ds.

When T has been estimated using Equation (2), S can be estimated by

$$S = \frac{0.3\ Tt_o}{r^2} \qquad (3)$$

where: t_o = intercept at zero drawdown (days).

It is to be noted that the Theis formula is based upon the following assumptions (Johnson Division, 1975):

1) The water bearing formation is uniform in character and permeability in both horizontal and vertical directions

2) The formation has uniform thickness

3) The formation has infinite areal extent

4) The formation receives no recharge from any source

5) The pumped well penetrates and receives water from the full thickness of the water bearing formation

6) The water removed from storage is discharged instantaneously with lowering of the head.

To account for variable pumping rates, the Cooper-Jacob method plots specific discharge (s/Q) versus a weighted time ($\bar{t}$). Equations (2) and (3) are then modified to

$$T = \frac{264}{D(s/Q)} \qquad (4)$$

$$S = \frac{0.3\ T\ \bar{t}_o}{r^2} \qquad (5)$$

where: D(s/Q) = specific drawdown per log cycle of time (ft/gpm)

For a more complete description of the method, the reader is referred to Johnson Division (1975) and Kruseman and de Ridder (1970).

To estimate the coefficient of T from the recovery data, two forms of the Theis recovery method were employed. It is to be noted that the time-recovery data from a pumped well is more accurate than the corresponding time-drawdown curve since the effects of momentary pumping rate fluctuations and pump vibration do not affect the water level measurements in the pumped well. The first method consists of plotting s' versus t/t', using $Q' = Q_i t_i / t_i$ as the weighted mean of the discharge rate (Kruseman and de Ridder, 1970).

Thus Equation (2) becomes

$$T = \frac{264\ Q'}{Ds'} \tag{6}$$

where: Ds' = residual drawdown (ft) per log cycle of time

t' = time since pumping stopped (days)

Estimates of S are not possible with this method.

The second method consists of plotting (s-s') versus t' employing the same weighted mean discharge as the previous method. Thus equation (2) becomes:

$$T = \frac{264Q'}{D(s-s')} \tag{7}$$

Where: s = calculated drawdown by extending the drawdown curve (ft)

D(s-s') = calculated recovery per log cycle of time (ft)

Estimates of S are possible with this method and no changes occur in Equation (3).

The first recovery method is preferred as it provides an independent check on the estimated transmissibility coefficients. It is not dependent on the time-drawdown curve while the second method requires the extension of this curve. Thus any error in the time-drawdown curve is propogated into the recovery estimates of transmissivity and storativity.

6.2.2 Description of the Pump Tests

The pump tests performed in the field are described in the report "Pump Well Installation and Pump Test Results - Hyde Park Remedial Program, December, 1983" prepared by CRA for Occidental Chemical Corporation. The following discusses the assumptions made when analyzing the pump test results to estimate the coefficients of T and S.

For pump test C-5A, 8:35 a.m. on November 1, 1983 was chosen as the start of the pump test. Pumping was performed previous to this time but the system was pumping NAPL and experienced many stoppages due to pump failures and electrical problems. Although the pump was off from 8:03 a.m. to 9:11 a.m. on November 2, 1983, the aquifer system did not recover to any significant degree. Therefore, the entire record from 8:35 a.m. on November 1 to 3:30 p.m. on November 3, 1983 was treated as one pump test.

For pump test D-6, the entire pumping record from 12:15 p.m. on November 7 to 12:29 p.m. on November 12, 1983 was utilized. On two occasions the pump was shut down briefly but the system did not recover to any significant degree. Therefore, it was treated as one pumping test.

Pump test F-4A was treated as two tests, one with pumping from 12:57 a.m. to 6:00 p.m. on November 15, 1983 and the other from 10:28 p.m. on November 15 to 6:11 a.m. on November 17, 1983. During the prolonged shut down, the system did recover significantly. Prior to 12:57 a.m. on November 15, some pumping occurred but it was intermittent due to pump failures and electrical problems.

6.2.3 Discussion of Pump Test Results

The results of the pump tests are presented on the figures indicated in Table 6. The results are summarized in Table 7.

The results of pump test well D-6 show an increase in slope at approximately 230 minutes. This could indicate that the cone of depression has encountered a less permeable boundary. Another interesting point illustrated on Figure 23 is that, after a pump shut down, the steep portion of the plot repeats itself but is displaced to the right

TABLE 6

PUMP TEST INFORMATION

Date of Test	Pumped Well	Observation Well	Figure No.
Nov. 1, 1983 to Nov. 3, 1983	C-5A	C-5A	14 & 15
		C-5-1	16,17,18
		D-6-1	19
		OW18-79	20,21,22
Nov. 7, 1983 to Nov. 12, 1983	D-6	D-6	23 & 24
		C-5A	25
		C-5-1	26
		D-6-1	27
		OW13-79	28
		OW18-79	29
		OW28-80	30
Nov. 15, 1983 to Nov. 17, 1983	F-4A	F-4A	31
		F-4	32
		F-4-1	33
		C-5A	34
		C-5-1	35
		D-6-1	36
		H-3	37
		OW18-79	38

TABLE 7

SUMMARY OF PUMP TEST RESULTS

Pump Test	Well	T (gpd/ft) Pumping	Recovery	S	Orientation	Distance (from pumping well)
C-5A	C-5A	630	1404	--	--	--
	C-5-1	1955	1404	5.94×10^{-3}	E	49'
			1649	1.80×10^{-3}		
	D-6-1	2256	--	2.23×10^{-6}	S20°E	517'
	OW18-79	1955	1404	9.20×10^{-5}	W25°N	319'
			1649	3.24×10^{-5}		
D-6	D-6	5500	355 (1010)	--	--	--
	C-5A	2296	--	5.57×10^{-4}	N20°W	477'
	C-5-1	1123	--	9.57×10^{-4}	N15°W	477'
	D-6-1	2983	--	3.73×10^{-2}	S45°E	51'
	OW13-79	3180	Little Recovery	4.83×10^{-5}	N	120'
	OW18-79	2983	--	2.09×10^{-4}	N90°W	741'
	OW28-80	2000	Little Recovery	2.67×10^{-5}	S75°E	188'
F-4A	F-4A(1)*	825	--	--	--	--
	F-4A(2)	325	--	--	--	--
	F-4(1)	1553	--	5.4×10^{-4}	N80°W	45'
	F-4(2)	2200	--	1.18×10^{-3}	N80°W	45'
	F-4-1(1)	3385	--	2.36×10^{-2}	S40°E	47'
	F-4-1(2)	4400	--	3.86×10^{-2}	S40°E	47'

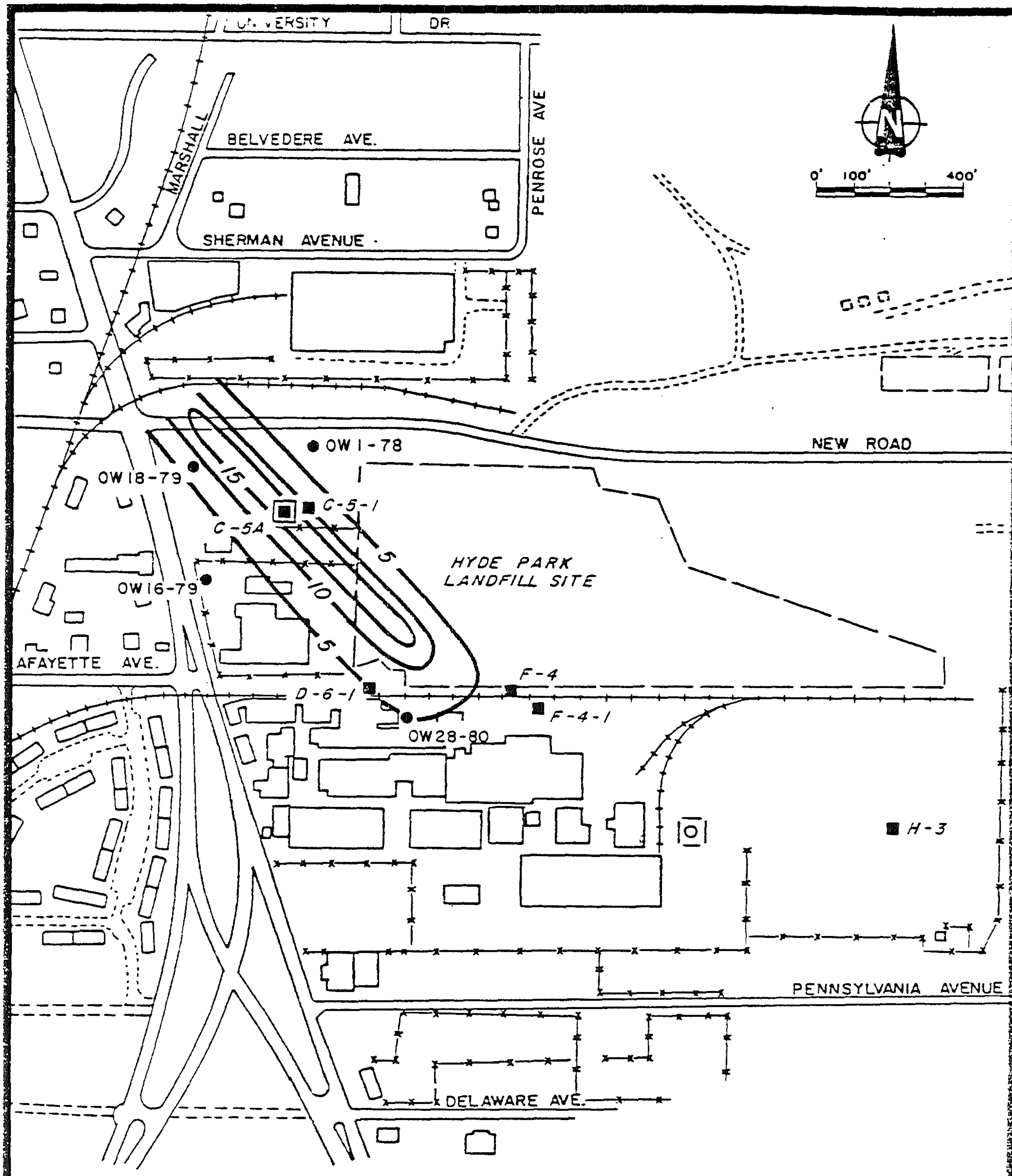
TABLE 7 (Cont'd)

SUMMARY OF PUMP TEST RESULTS

Pump Test	Well	T (gpd/ft) Pumping	Recovery	S	Orientation (from pumping well)	Distance
F-4A	C-5A(1)	3106	--	6.22×10^{-5}	N45°W	809'
	C-5A(2)	3882	--	4.45×10^{-5}	N45°W	809'
	C-5-1(1)	3667	--	9.74×10^{-5}	N40°W	782'
	C-5-1(2)	5867	--	9.00×10^{-5}	N40°W	782'
	D-6-1(1)	3105	--	1.97×10^{-4}	W	433'
	D-6-1(2)	3330	--	1.58×10^{-4}	W	433'
	H-3(1)	330	--	3.55×10^{-6}	575°E	1096'
	H-3(2)	1309	--	5.54×10^{-6}	575°E	1096'
	OW18-79(1)	3667	--	2.97×10^{-5}	N55°W	1123'
	OW18-79(2)	3667	--	2.97×10^{-5}	N55°W	1123'

*Bracketed numbers indicate pump test results for first and second time period respectively.

Geometric Mean Transmissivity Estimate = 1,890 gpd/ft
Arithmetic Mean Effective Storativity Estimate = 4.46×10^{-3}



LEGEND

—15— POSTULATED GROUNDWATER DRAWDOWN CONTOUR

● GROUNDWATER OBSERVATION WELL

■ BEDROCK SURVEY

▣ PUMPED WELL

CRA

figure 11

POSTULATED GROUNDWATER DRAWDOWN CONTOURS PUMP TEST C-5A

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Occidental Chemical Corporation

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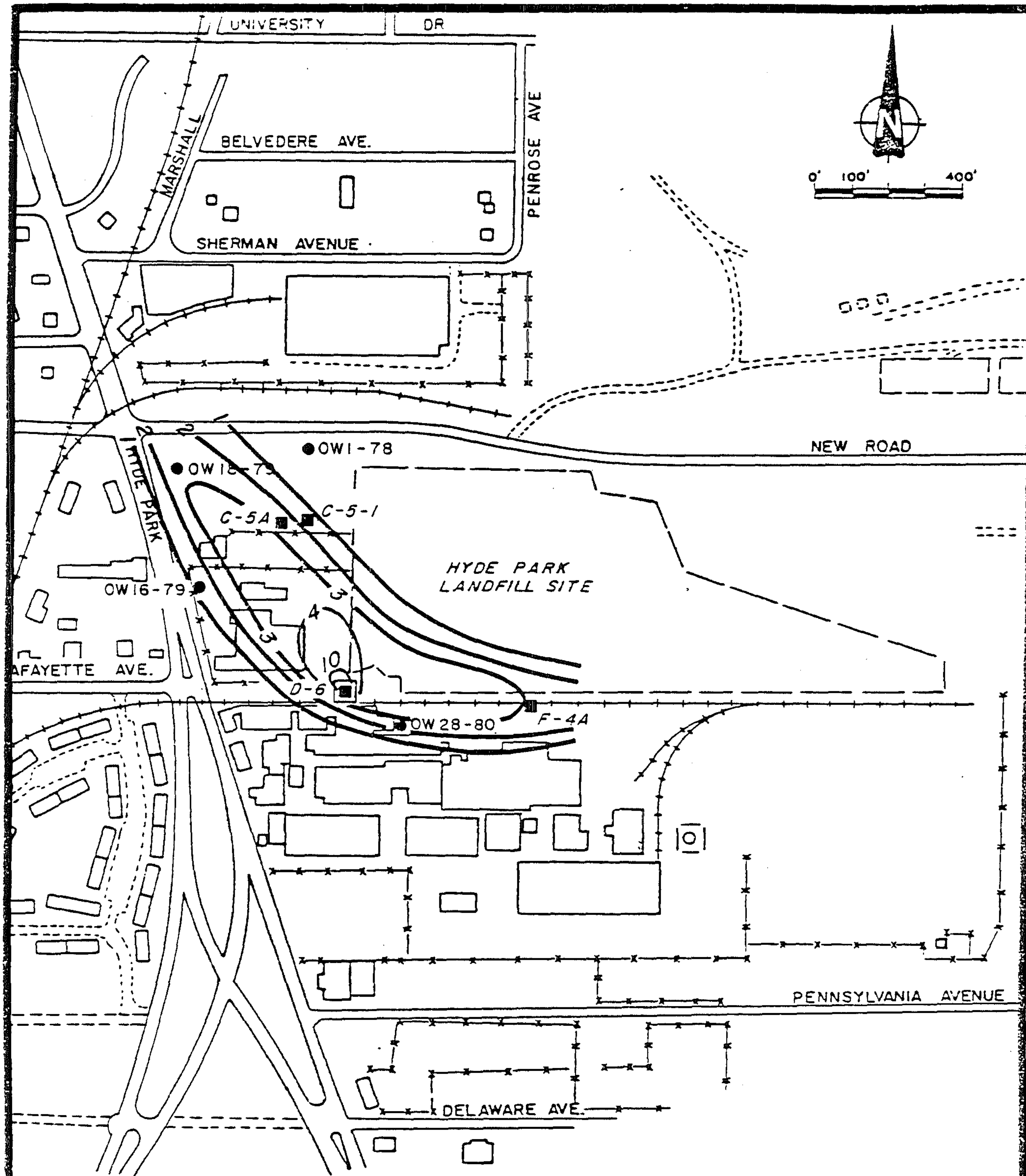

LEGEND

—2— POSTULATED GROUNDWATER DRAWDOWN CONTOUR

● GROUNDWATER OBSERVATION WELL

■ BEDROCK SURVEY

▣ PUMPED WELL

CRA

figure 12

POSTULATED GROUNDWATER DRAWDOWN CONTOURS PUMP TEST D-6

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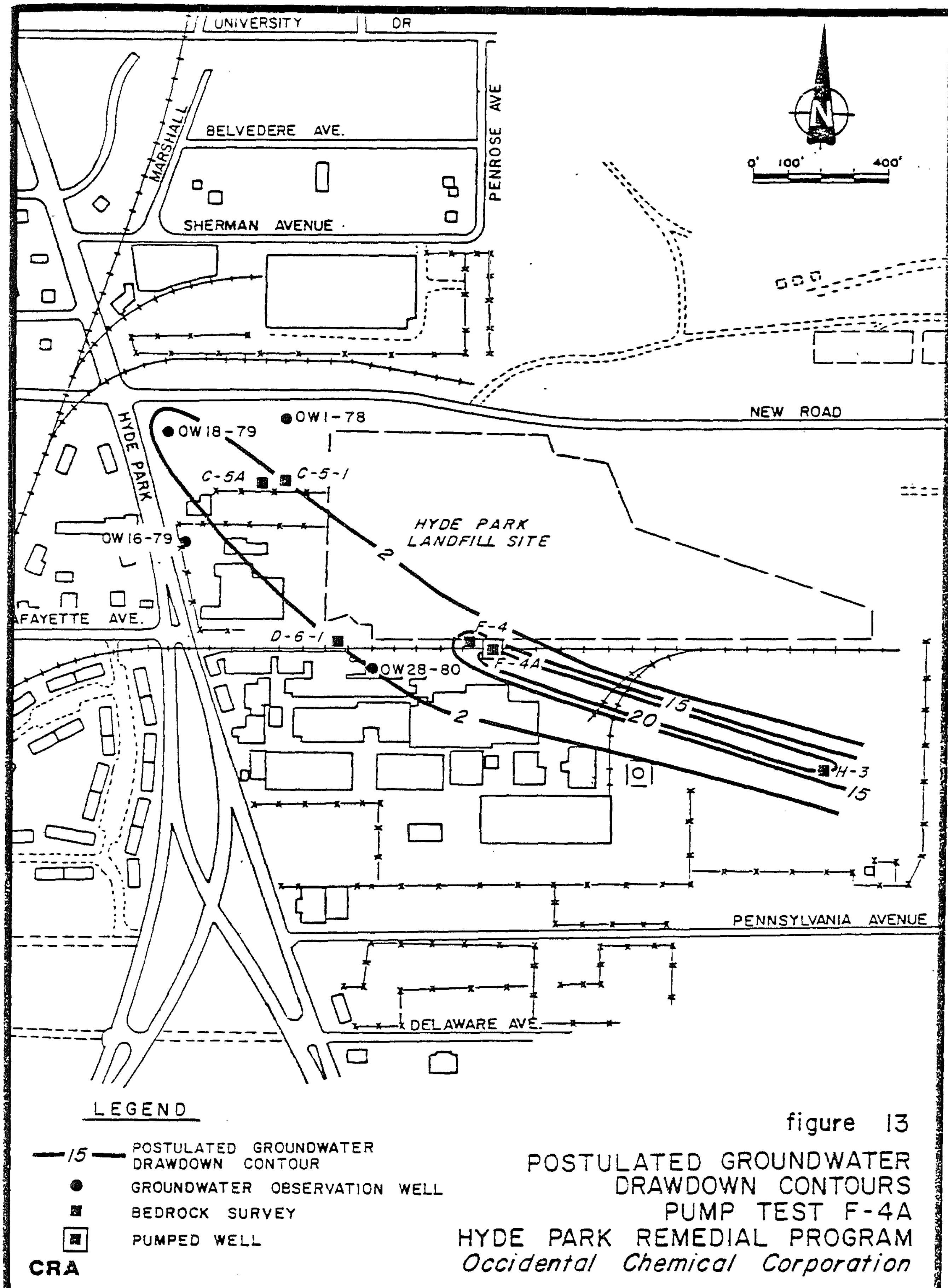

figure 13

POSTULATED GROUNDWATER DRAWDOWN CONTOURS PUMP TEST F-4A

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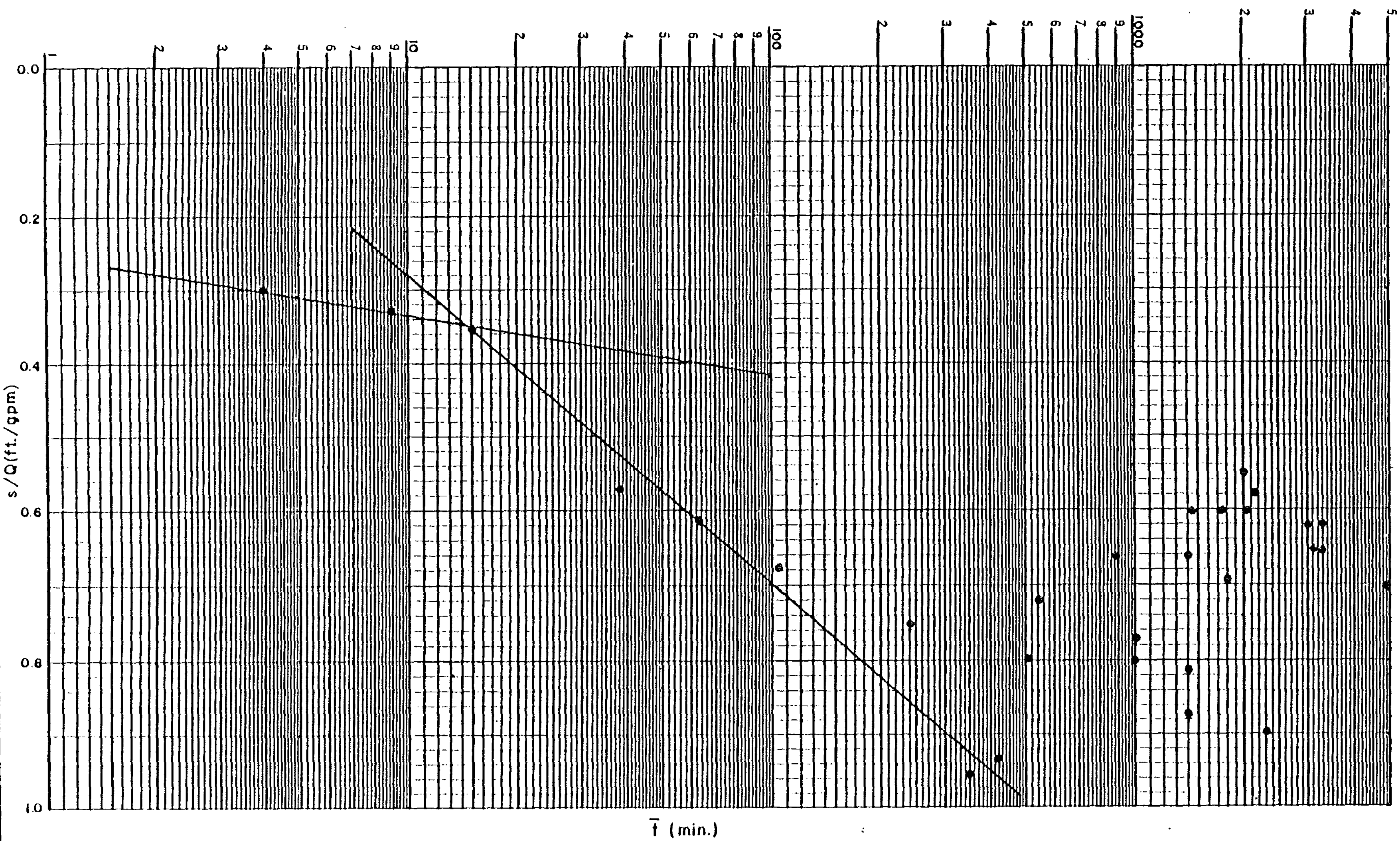

figure 14

AQUIFER RESPONSE
PUMP TEST C-5A
WELL C-5A

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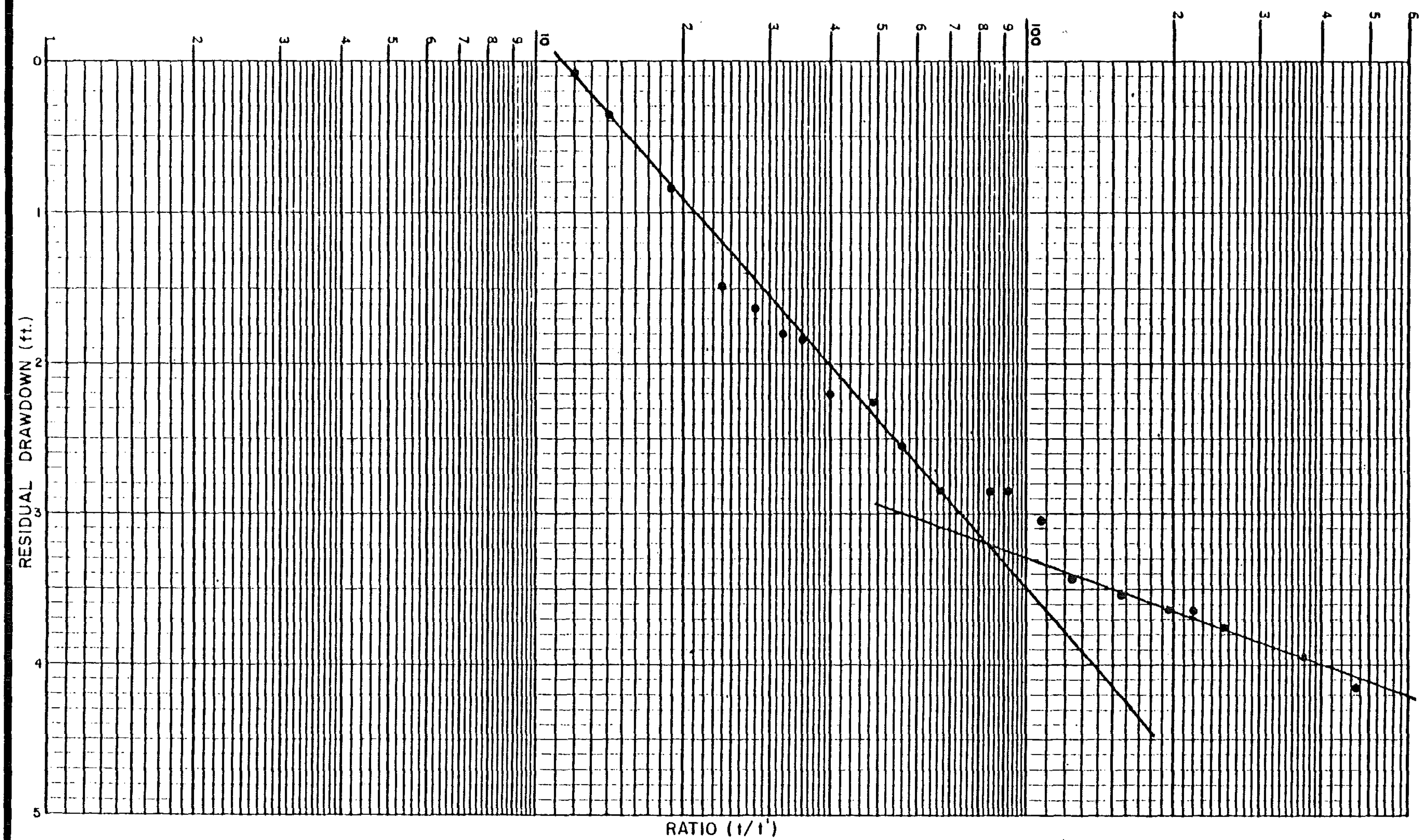

figure 15

AQUIFER RESPONSE
-RECOVERY TEST
PUMP TEST C-5A
WELL C-5A
Hyde Park Remedial Program

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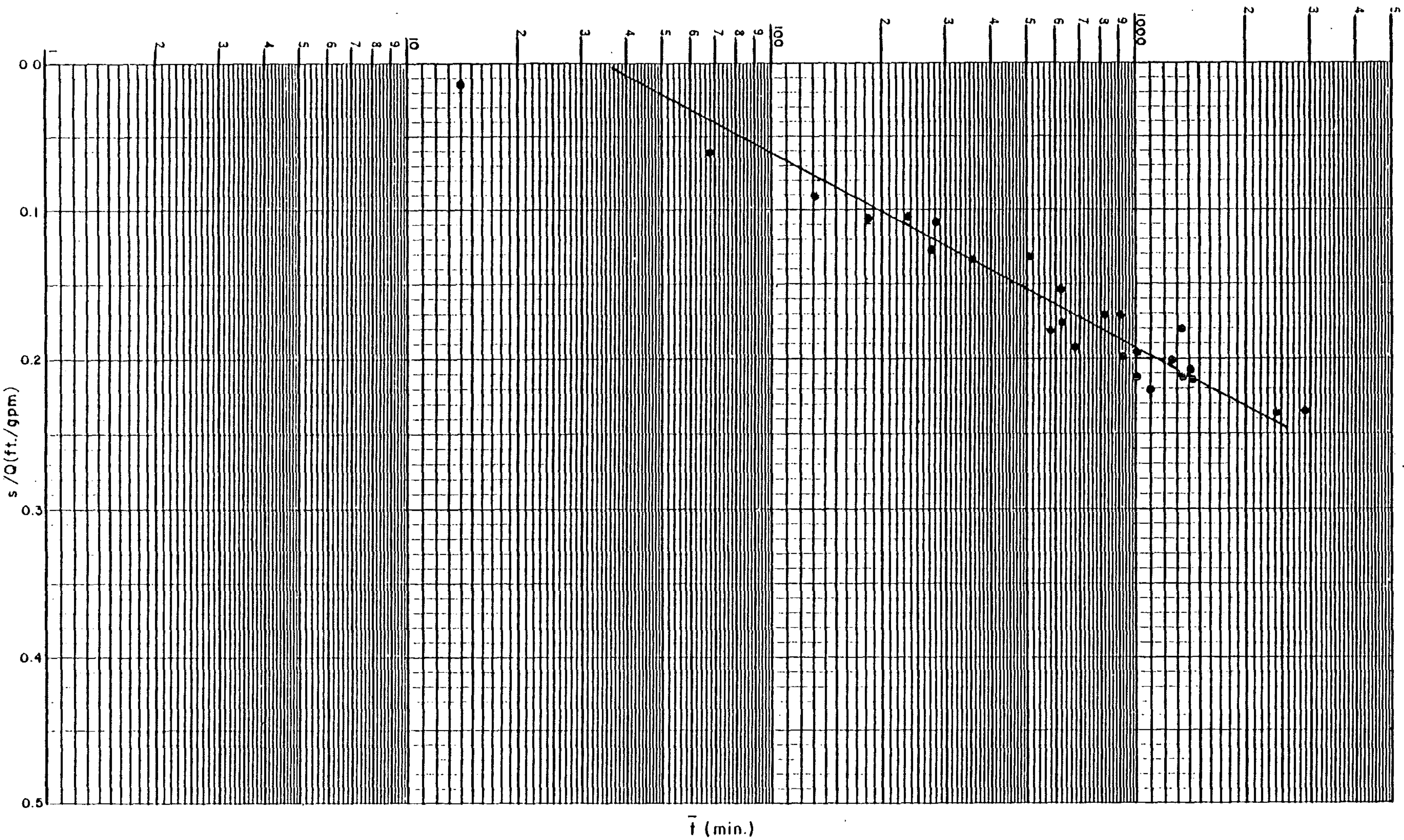

figure 16

AQUIFER RESPONSE
PUMP TEST C-5A
WELL C-5-I
Hyde Park Remedial Program

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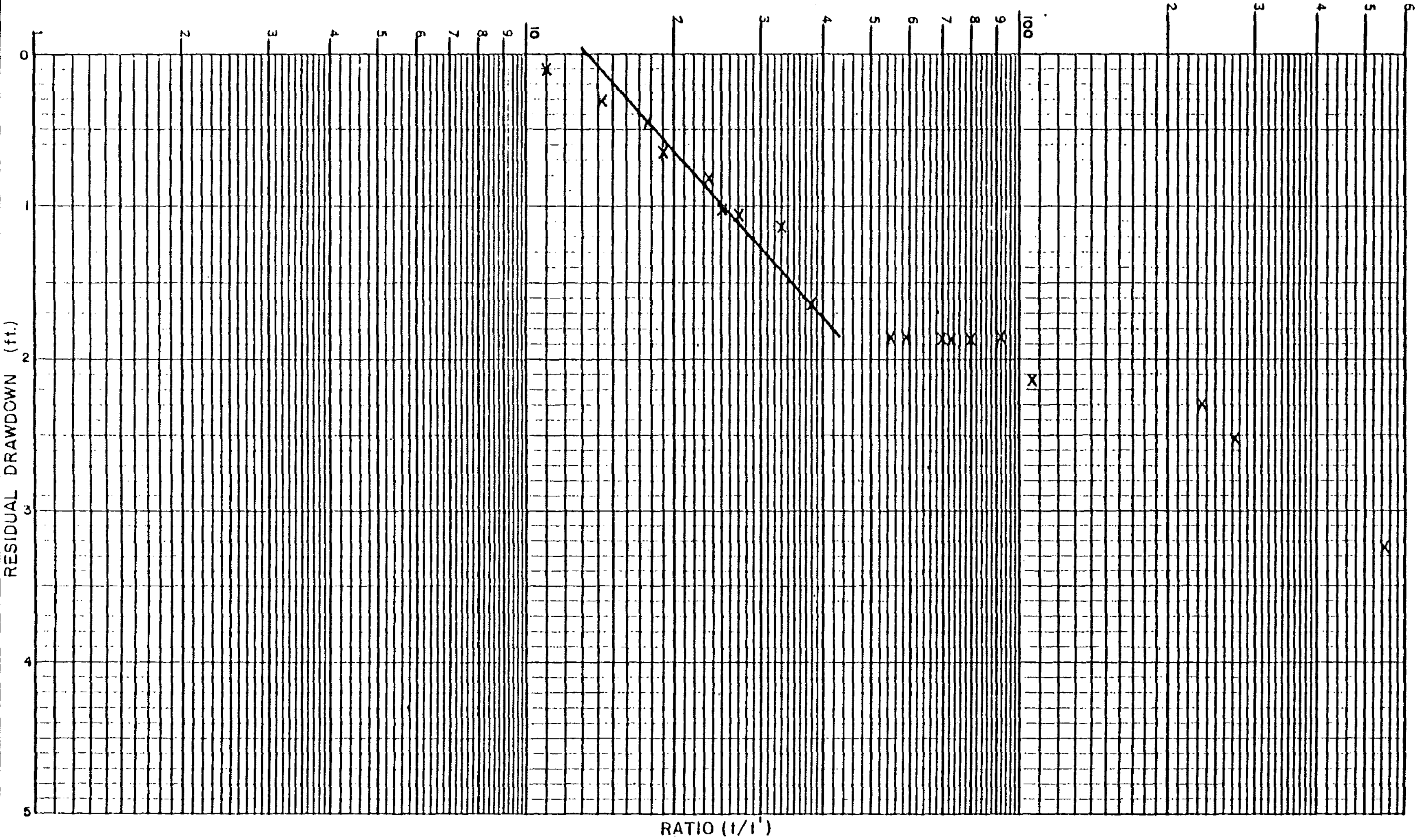

figure 17

AQUIFER RESPONSE
-RECOVERY TEST
PUMP TEST C-5A
WELL C-5-I
Hyde Park Remedial Program

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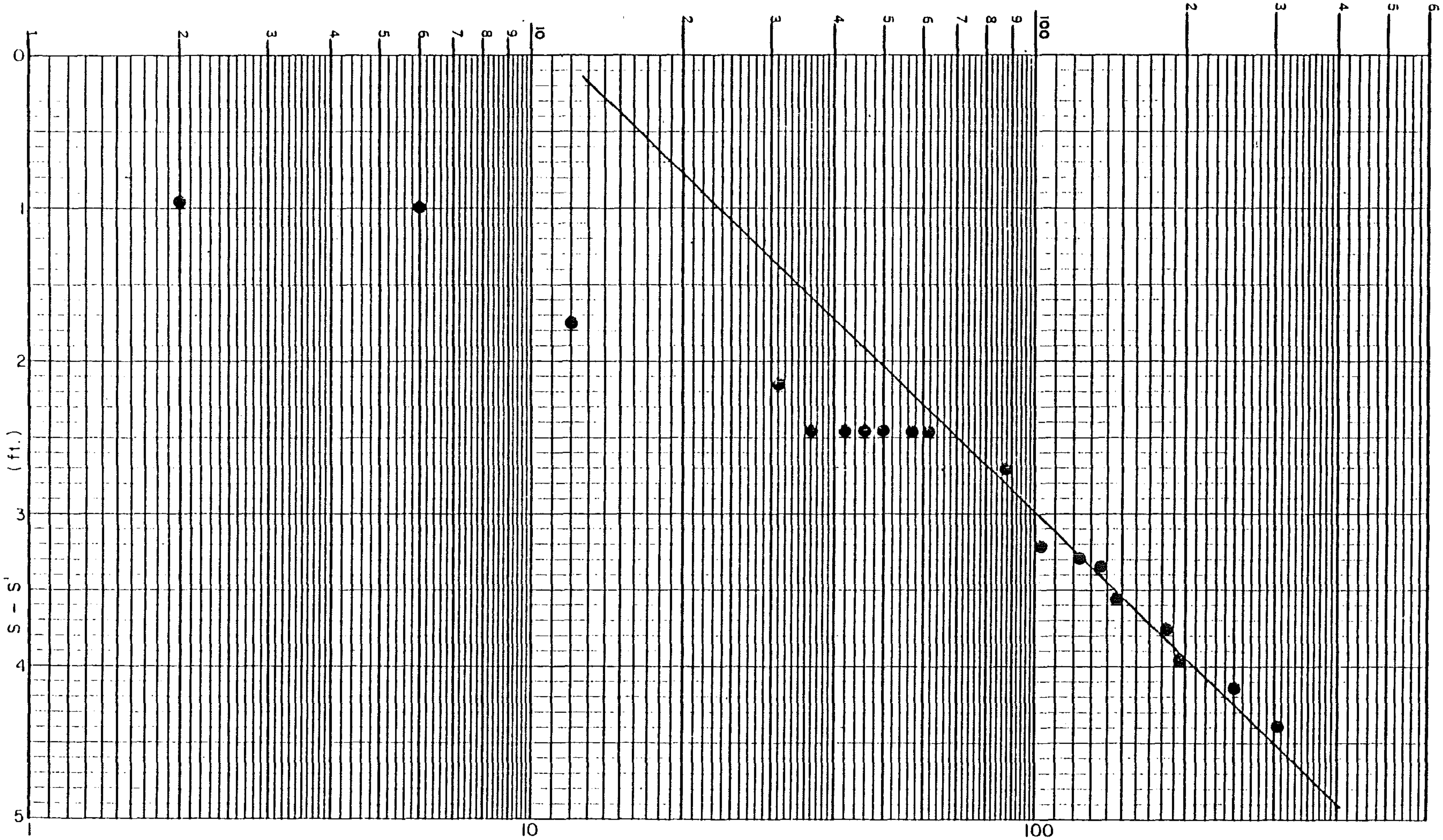

figure 18

AQUIFER RESPONSE
-RECOVERY TEST 2
PUMP TEST C-5A
WELL C-5-1
Hyde Park Remedial Program

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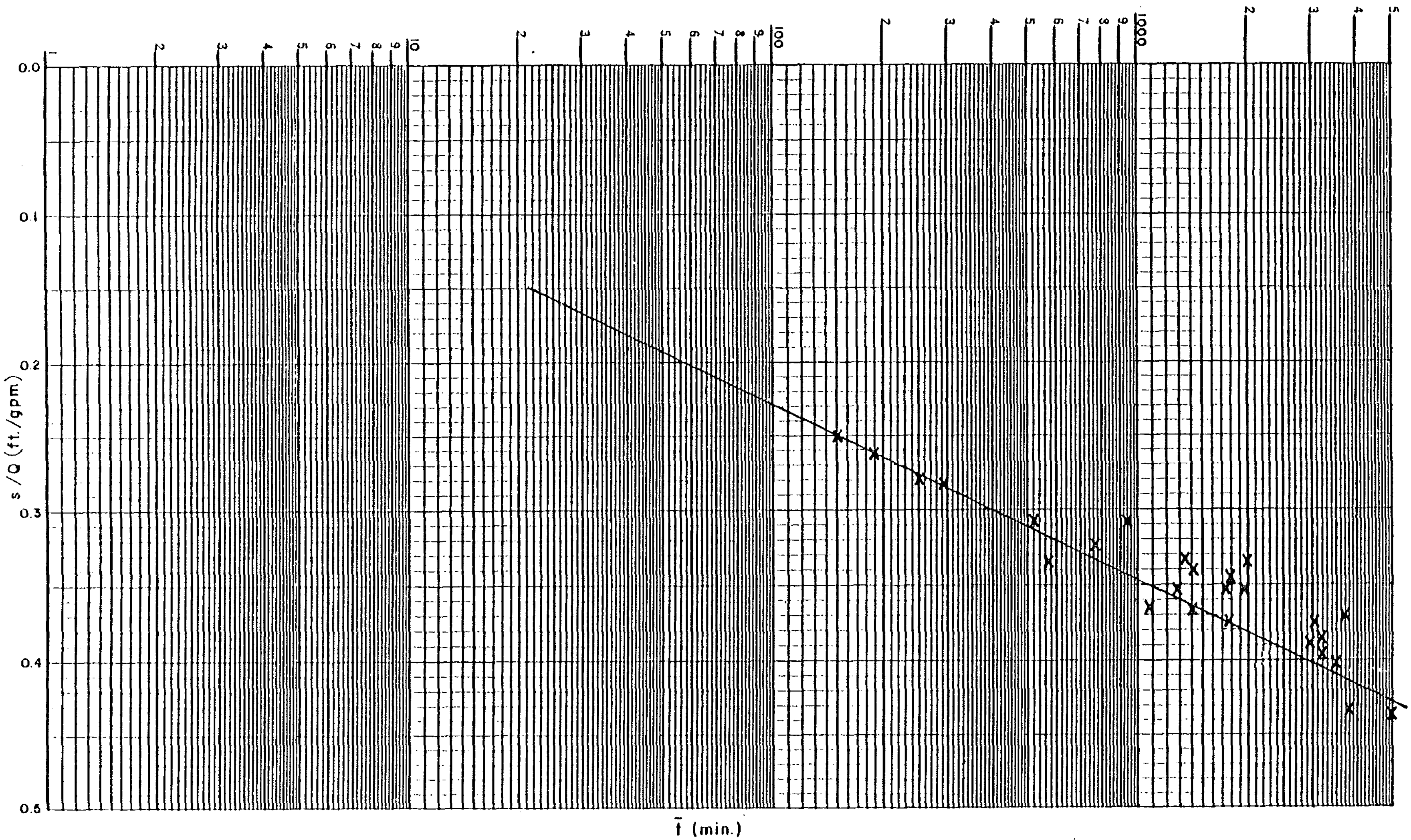

figure 19

AQUIFER RESPONSE
PUMP TEST C-5A
WELL D-6-1

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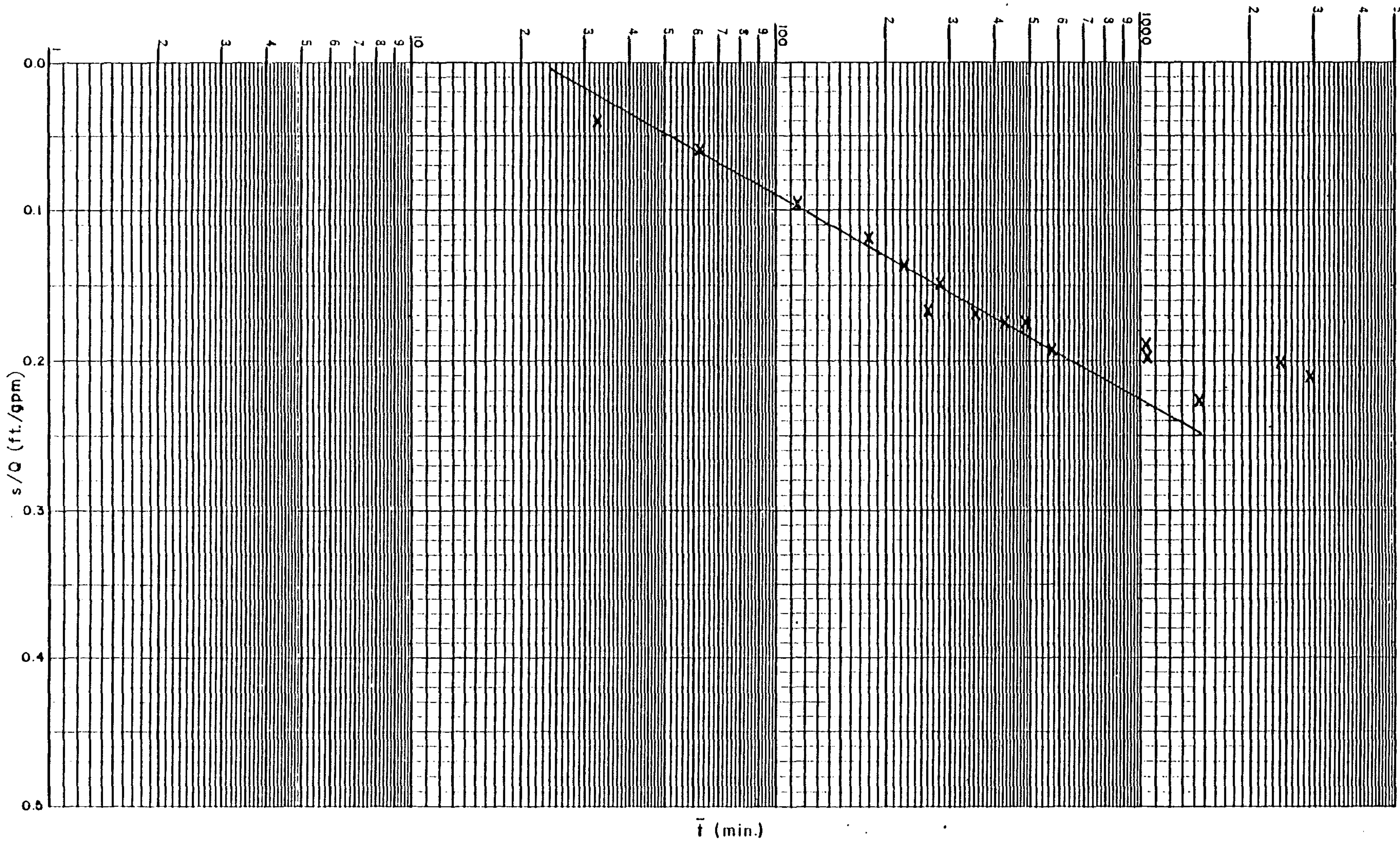

figure 20

AQUIFER RESPONSE
PUMP TEST C-5A
WELL OW18-79
Hyde Park Remedial Program

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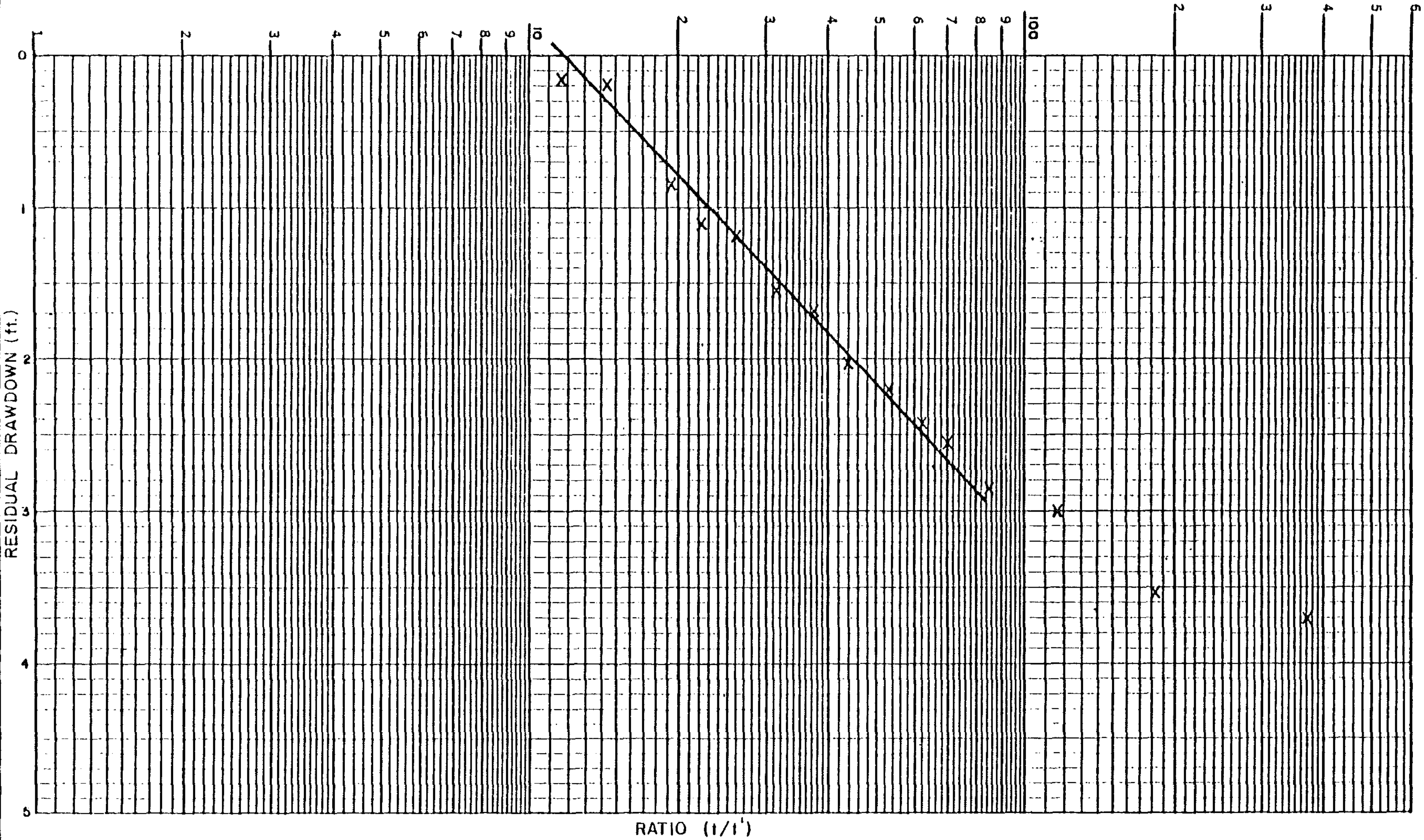

figure 21

AQUIFER RESPONSE
- RECOVERY TEST
PUMP TEST C-5A
WELL OW 18-79
Hyde Park Remedial Program

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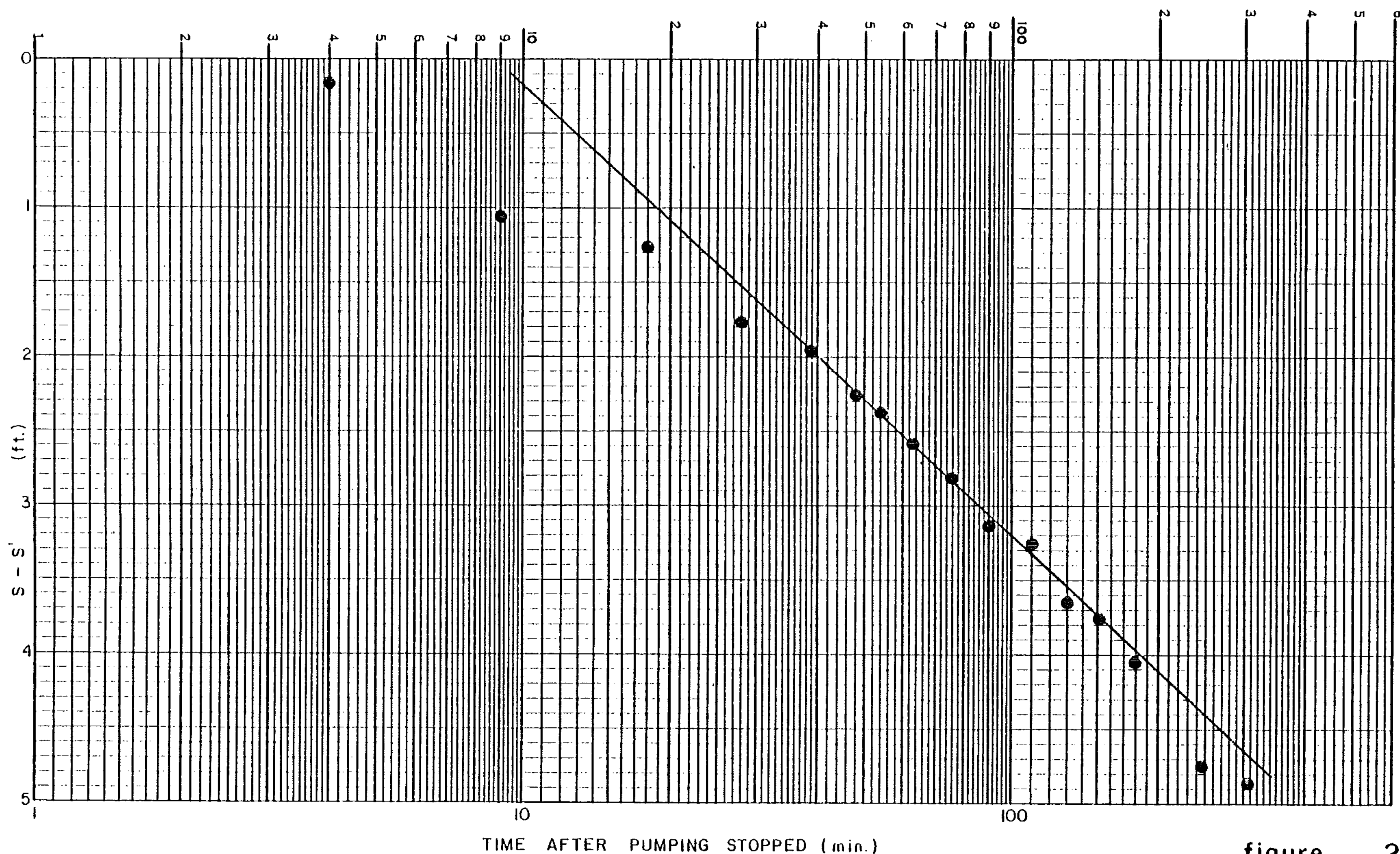

figure 22

AQUIFER RESPONSE
- RECOVERY TEST 2
PUMP TEST C-5A
WELL OW18-79

Hyde Park Remedial Program

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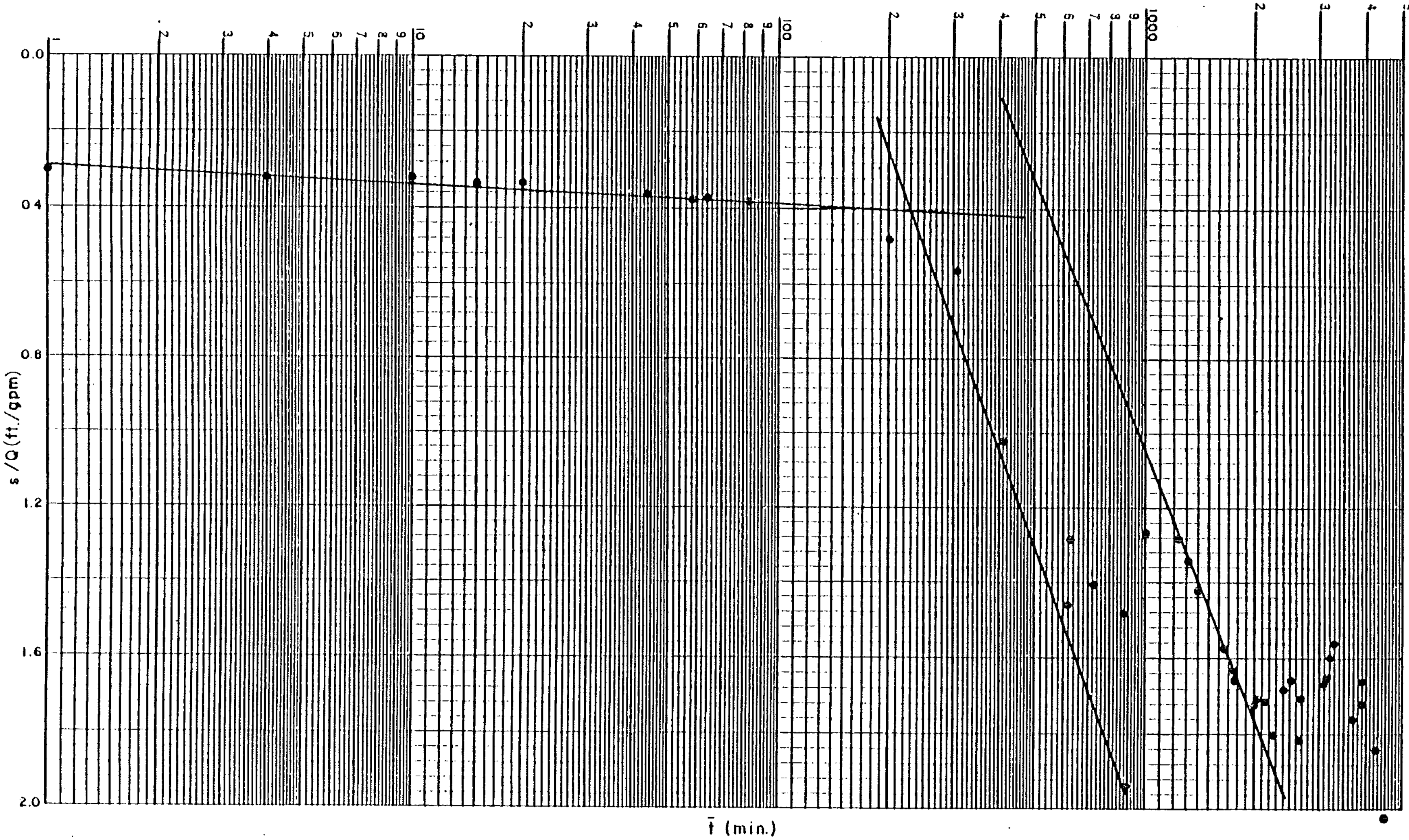

figure 23

AQUIFER RESPONSE
PUMP TEST D-6
WELL D-6
Hyde Park Remedial Program

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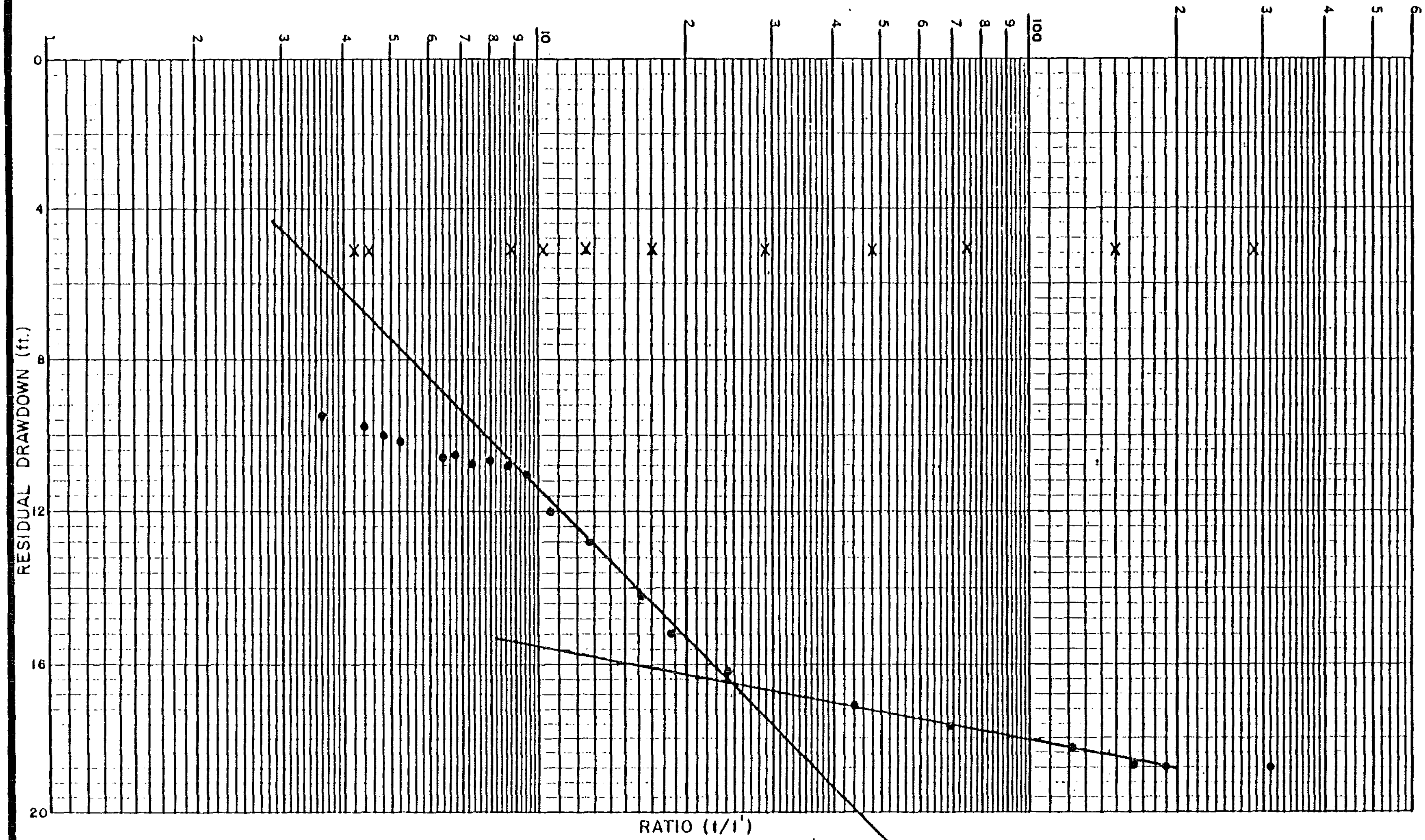

LEGEND

- ● PUMP TEST D-6
- X WELL OW13-79

figure 24

AQUIFER RESPONSE
- RECOVERY TEST
PUMP TEST D-6
WELL OW13-79

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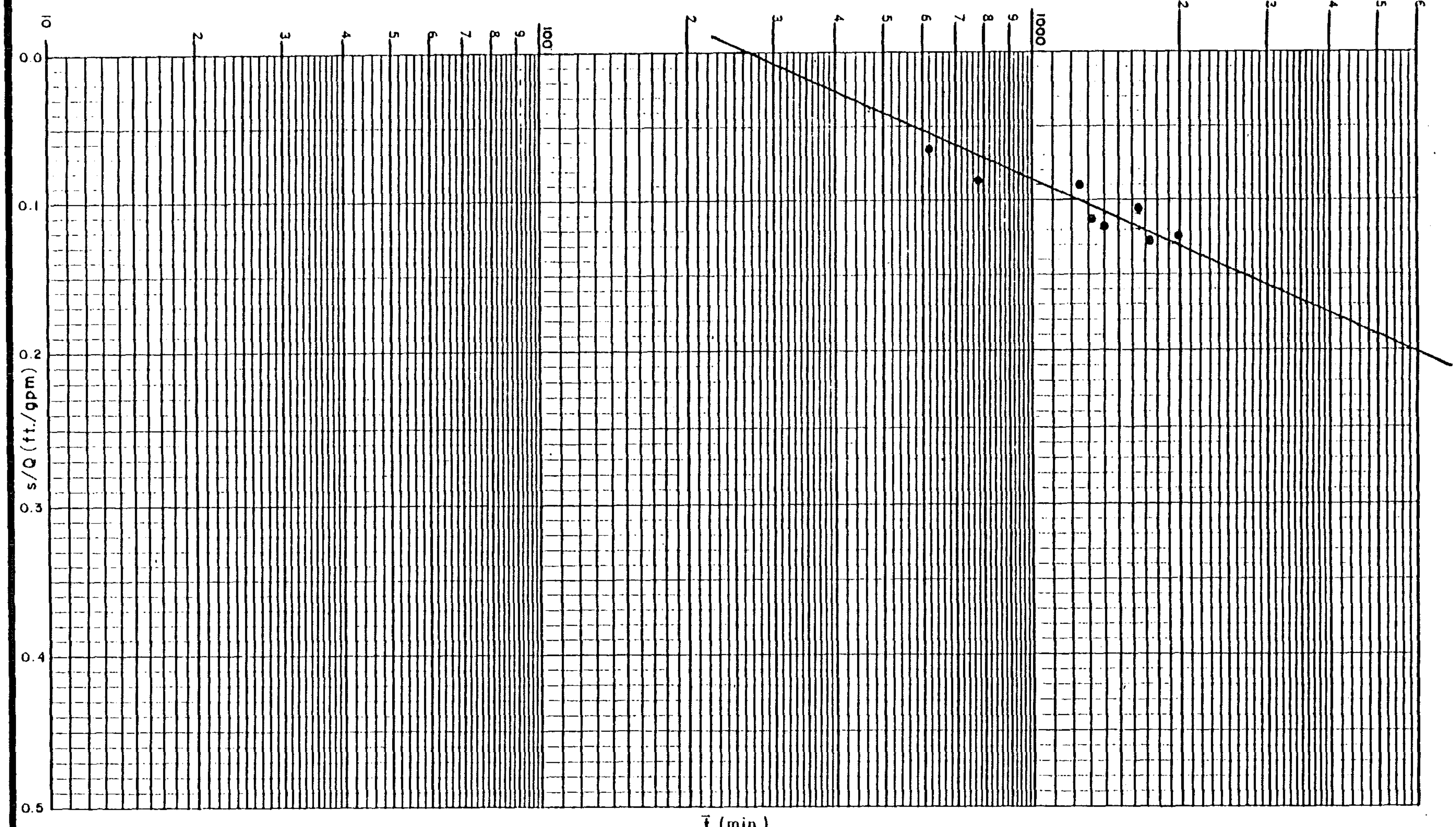

figure 25

AQUIFER RESPONSE
PUMP TEST D-6
WELL C-5A
Hyde Park Remedial Program

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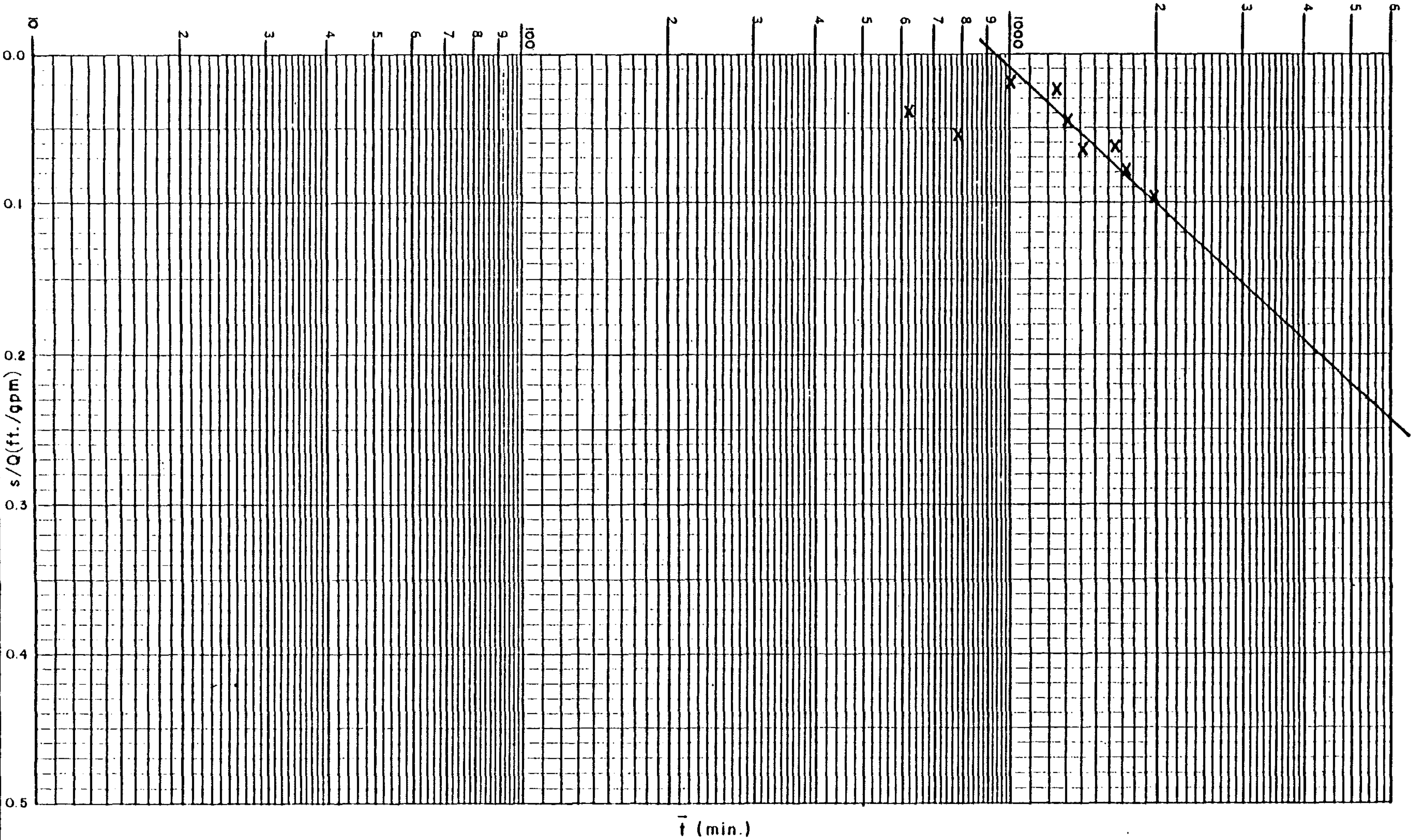

figure 26

AQUIFER RESPONSE
PUMP TEST D-6
WELL C-5-1
Hyde Park Remedial Program

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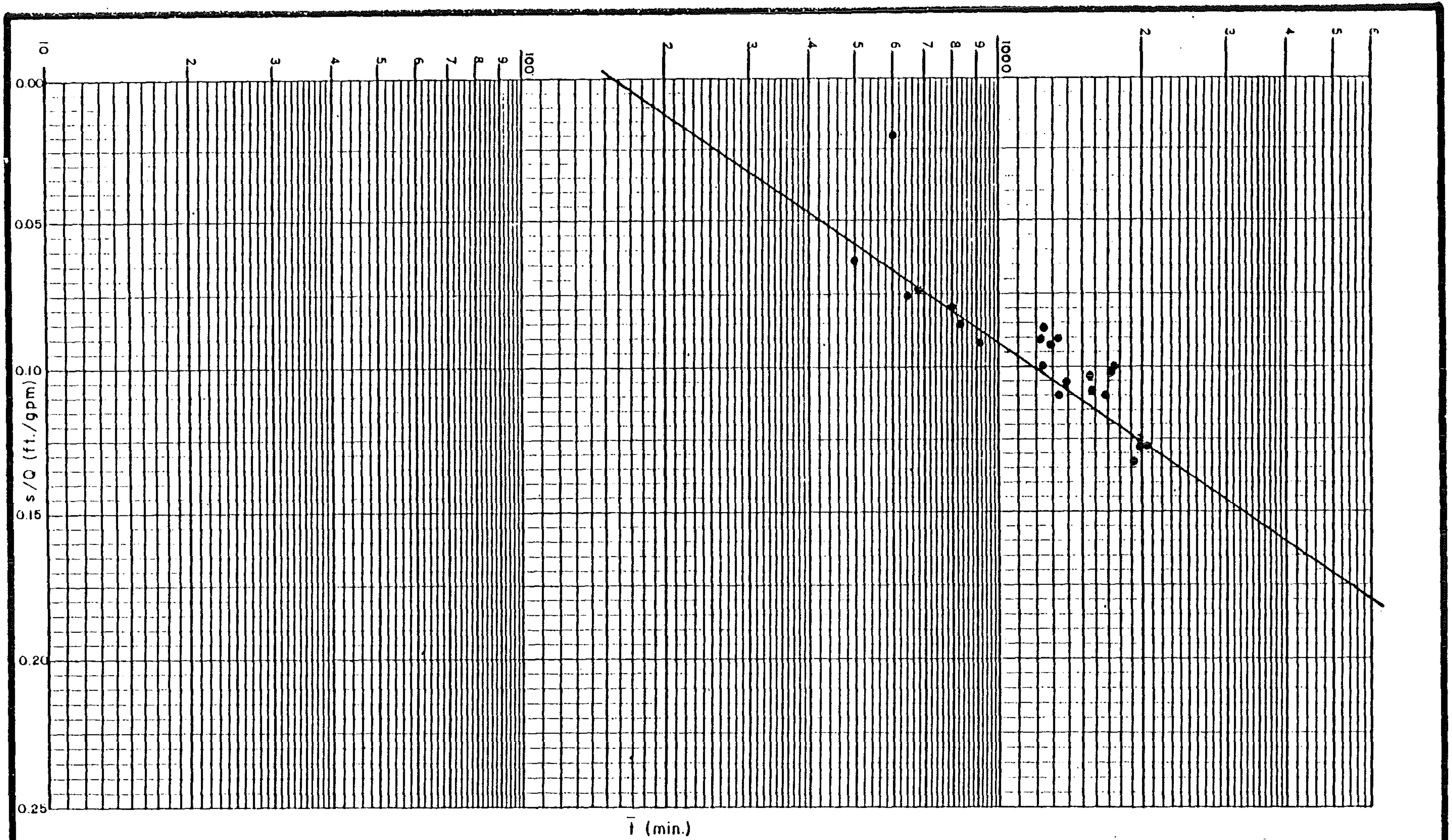

figure 27

AQUIFER RESPONSE
PUMP TEST D-6
WELL D-6-1
Hyde Park Remedial Program

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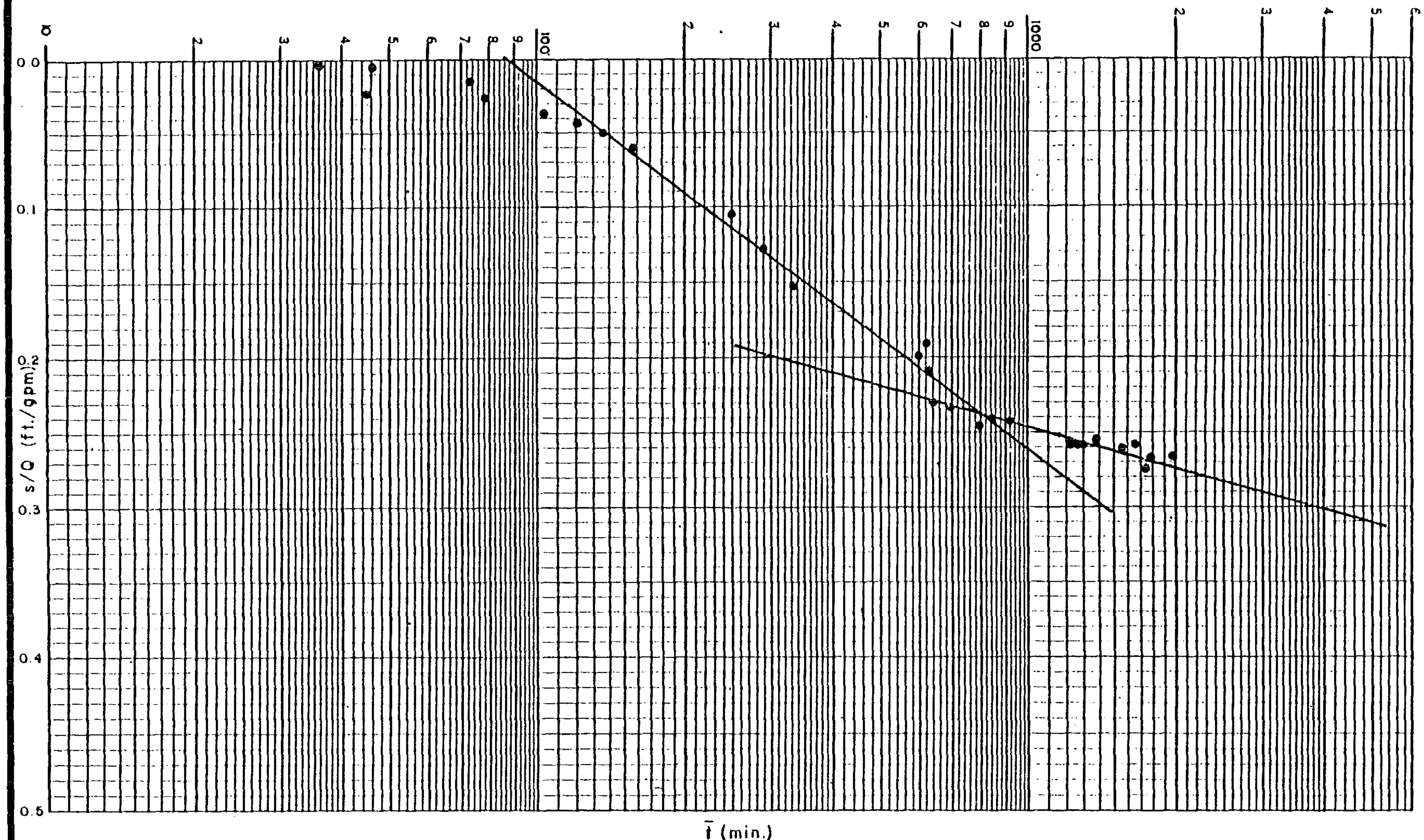

figure 28

AQUIFER RESPONSE
PUMP TEST D-6
WELL OW 13-79
Hyde Park Remedial Program

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1069 - 21/03/84

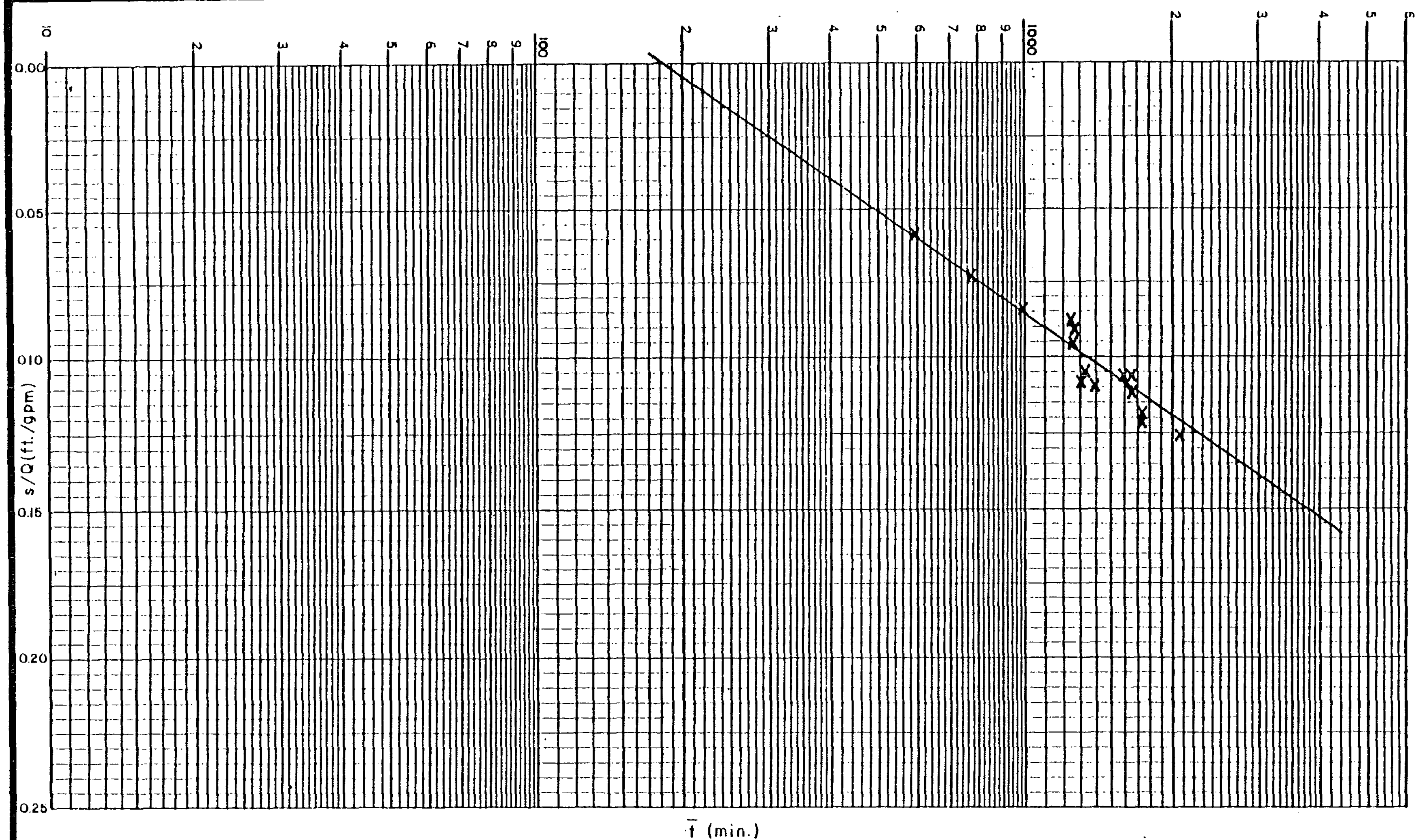

figure 29

AQUIFER RESPONSE
PUMP TEST D-6
WELL OW 18-79
Hyde Park Remedial Program

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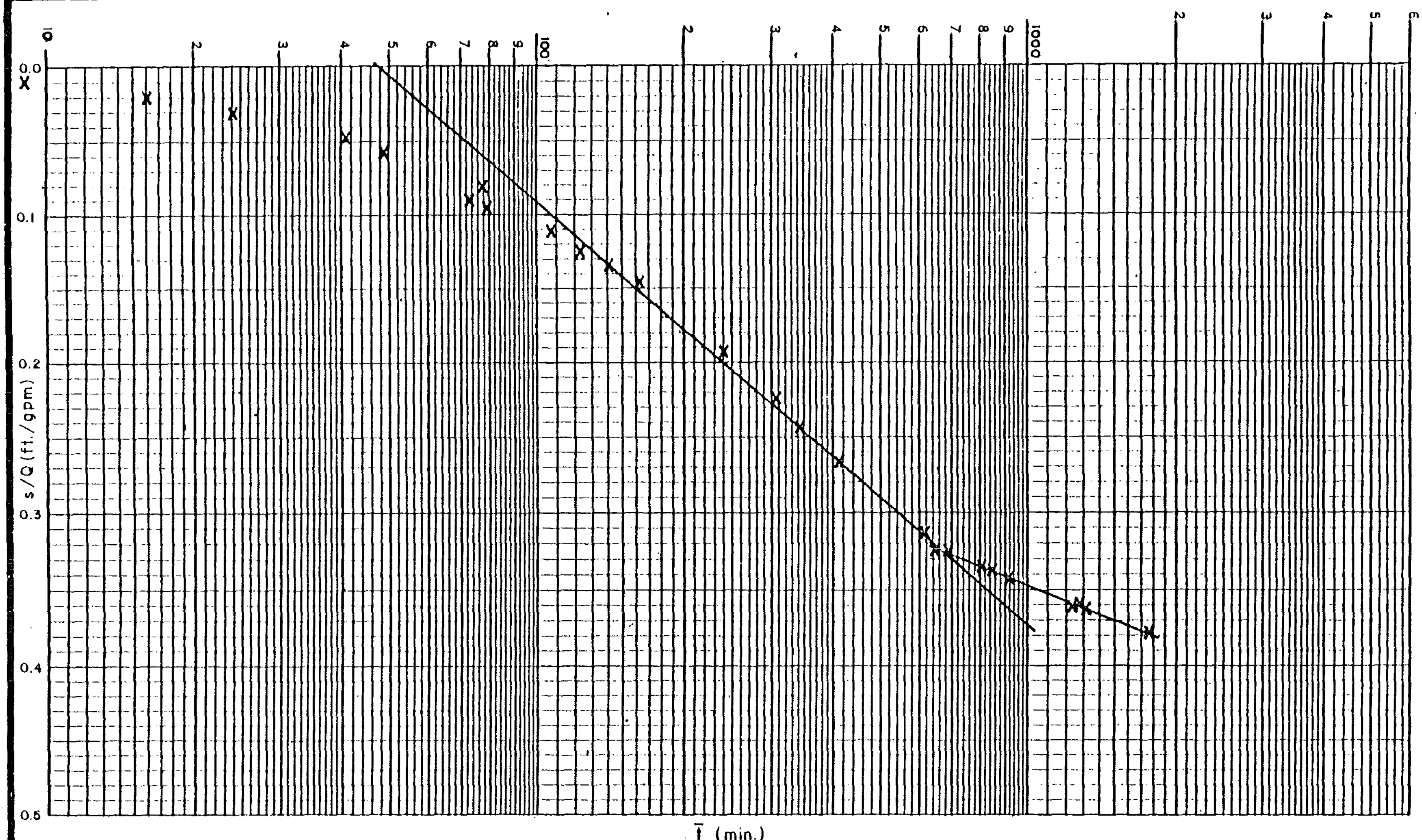

figure 30

AQUIFER RESPONSE
PUMP TEST D-6
WELL OW 28-80
Hyde Park Remedial Program

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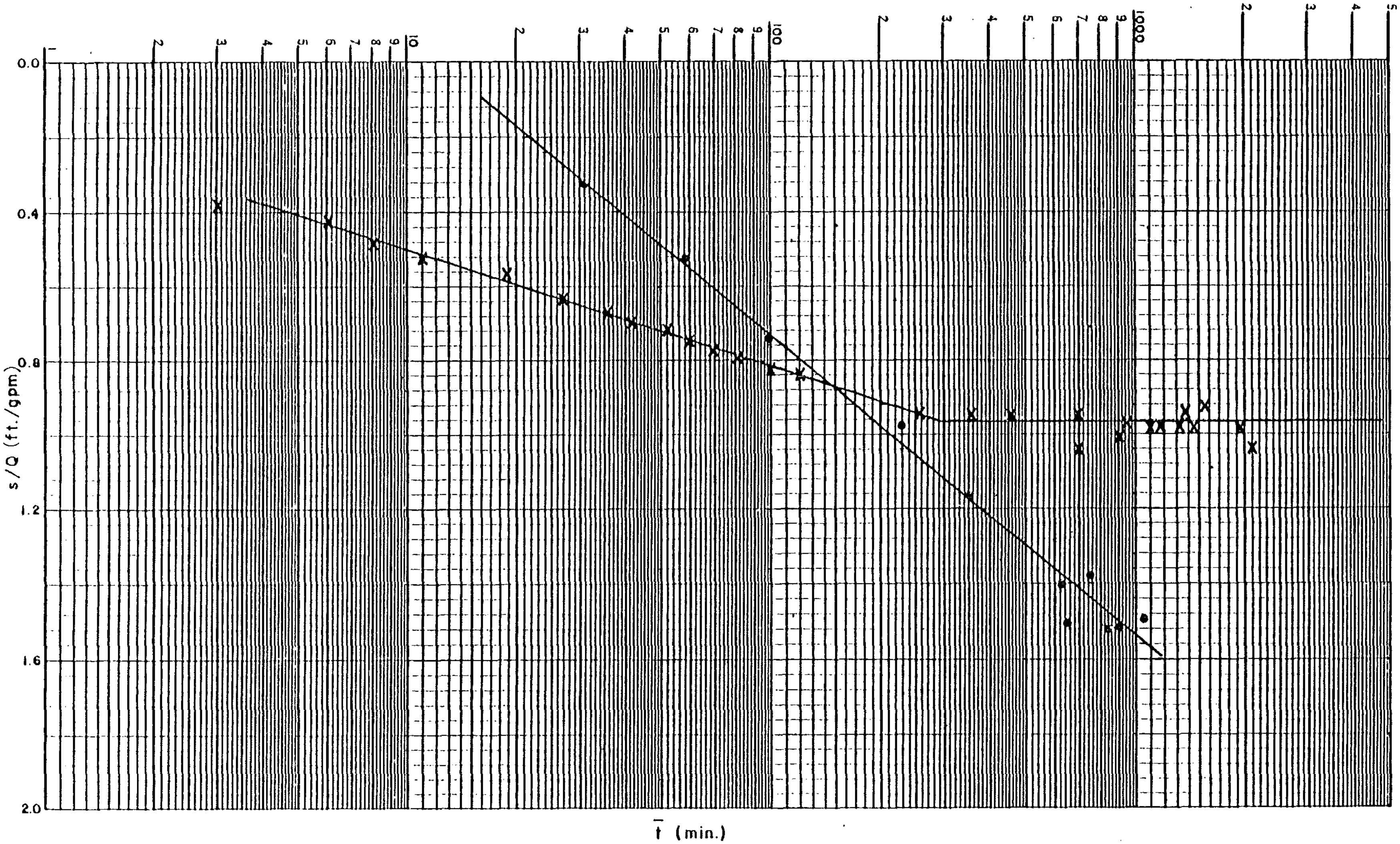

LEGEND

• TEST (1) - NOV. 15 - 12:57 am to 6:00 pm

X TEST (2) - NOV. 17 - 10:28 pm to NOV. 17 - 6:11 pm

figure 31

AQUIFER RESPONSE
PUMP TEST F-4A
WELL F-4A(1)&(2)
Hyde Park Remedial Program

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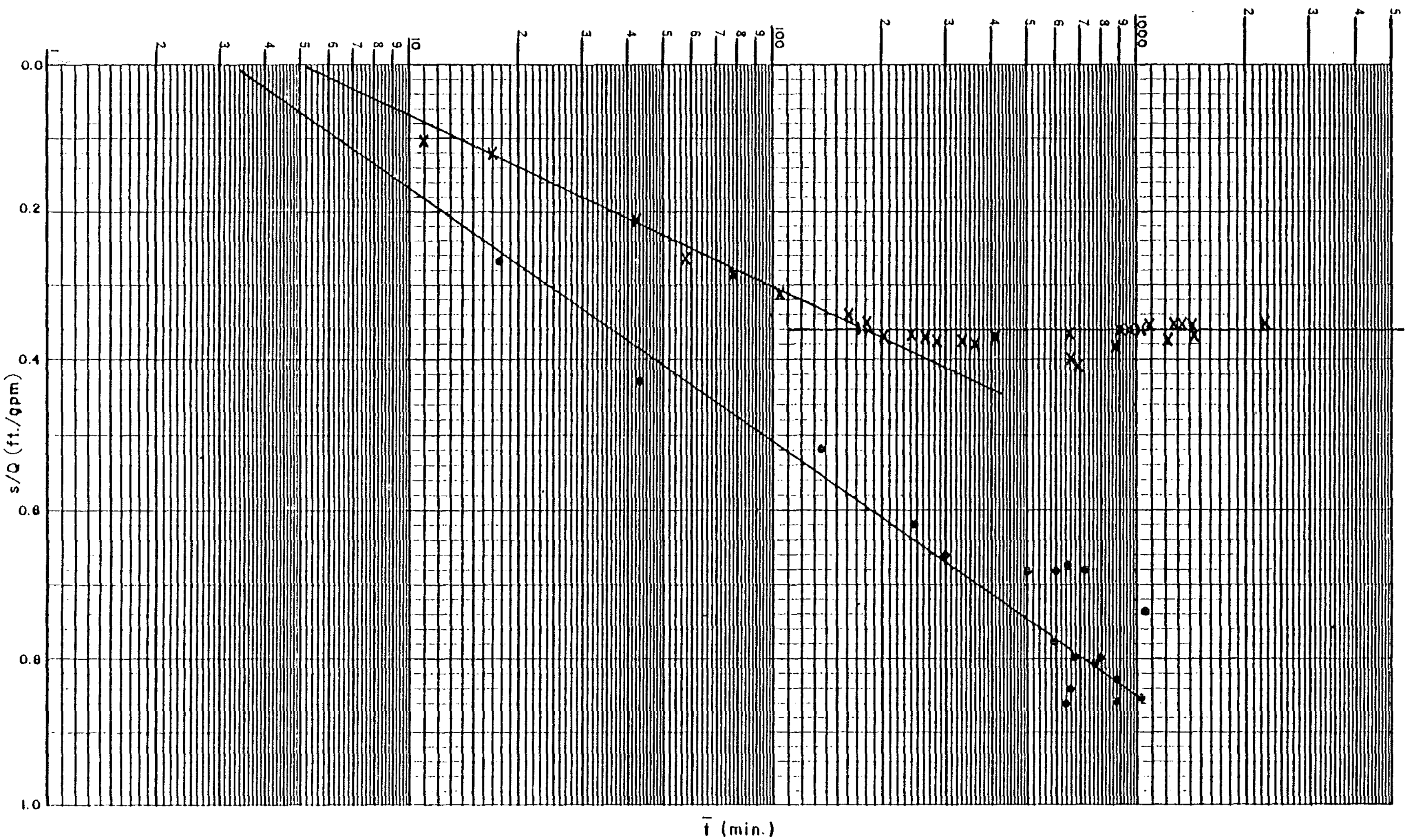

figure 32

AQUIFER RESPONSE
PUMP TEST F-4A
WELL F-4(1)&(2)
Hyde Park Remedial Program

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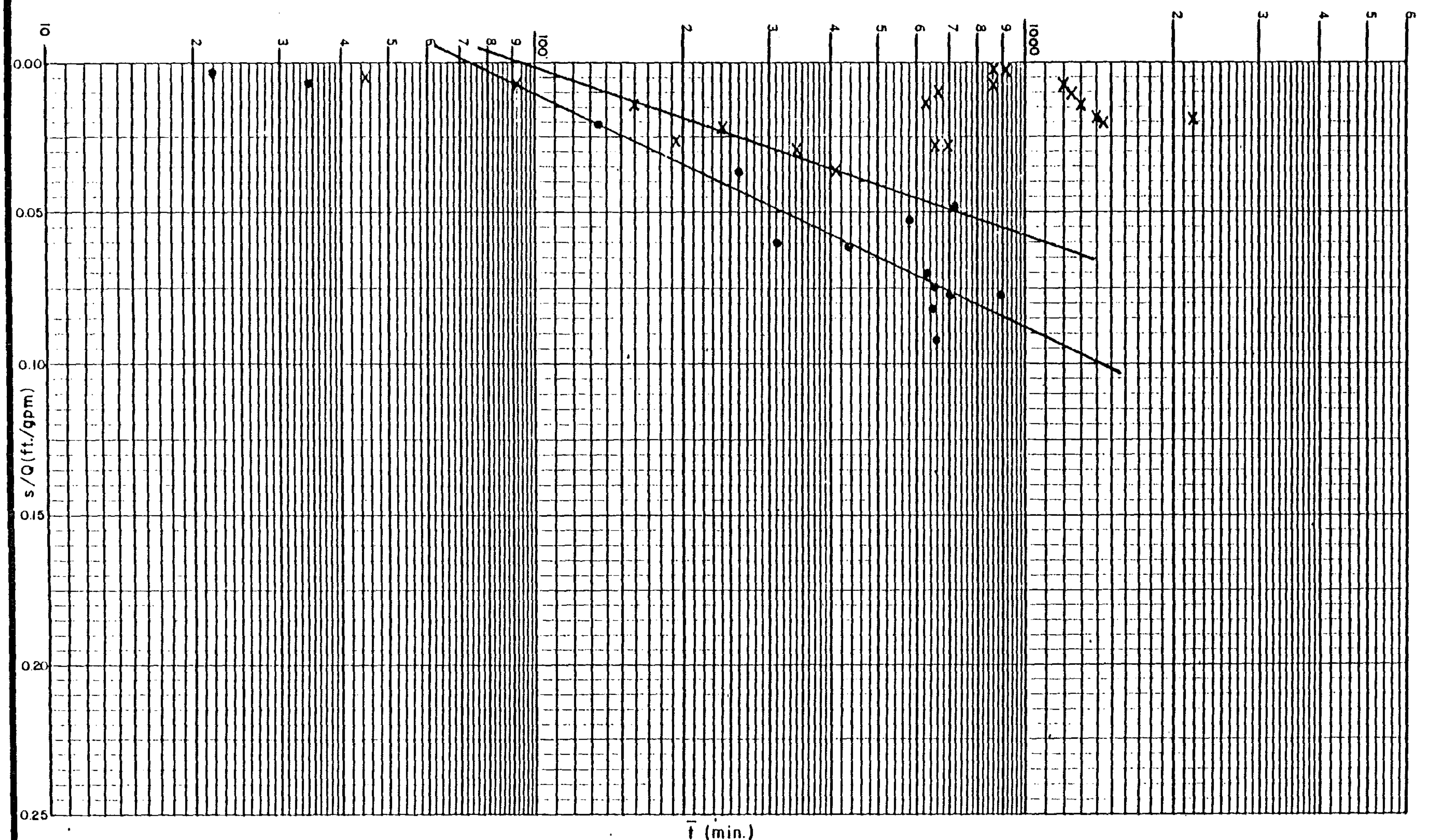

LEGEND

- ● TEST (1)
- X TEST (2)

figure 33

AQUIFER RESPONSE
PUMP TEST F-4A
WELL F-4-1(1) & (2)

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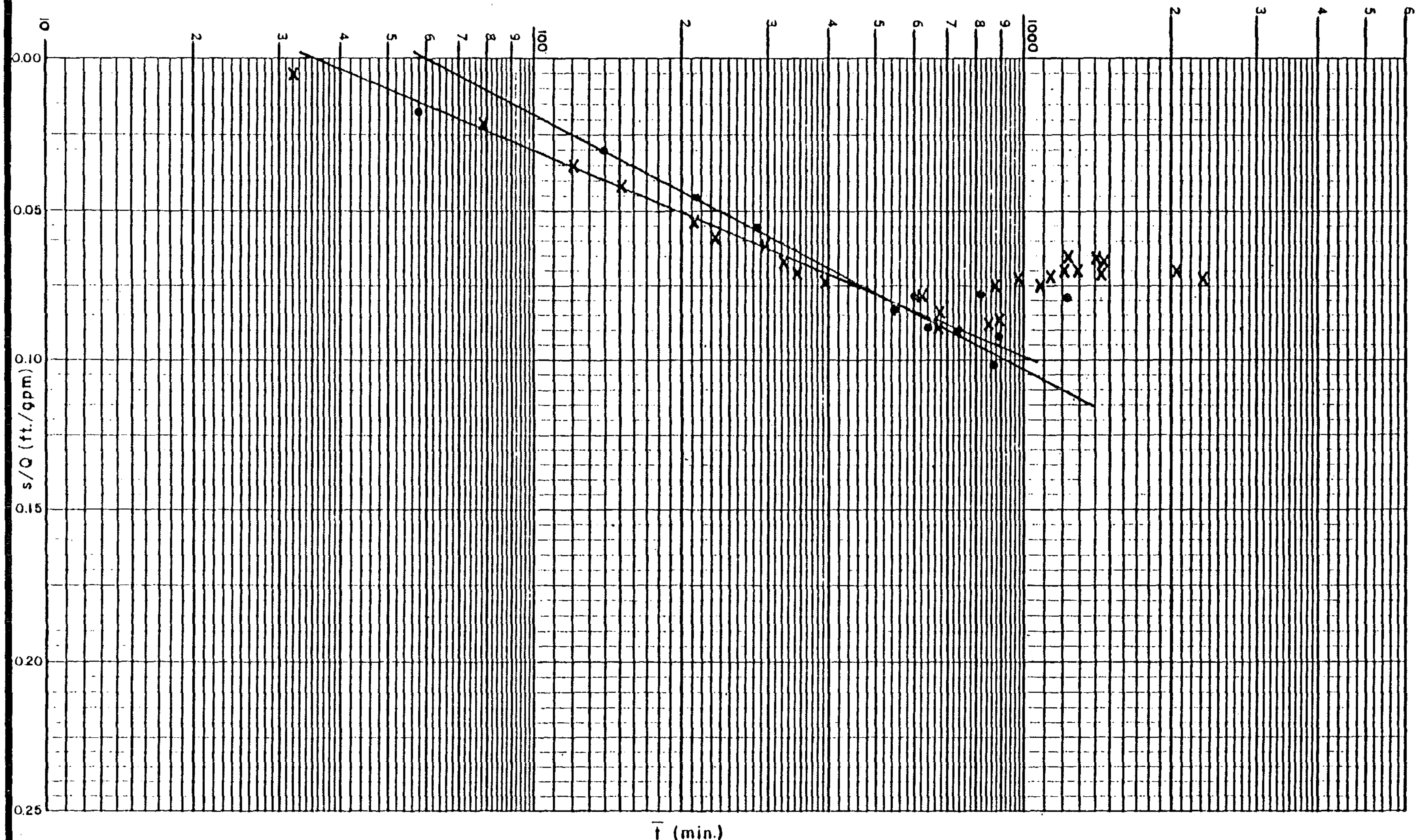
1059 - 31/03/84



LEGEND

● TEST (1)

X TEST (2)

figure 34

AQUIFER RESPONSE
PUMP TEST F-4A
WELL C-5A (1) & (2)
Hyde Park Remedial Program

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1069 - 21/03/84

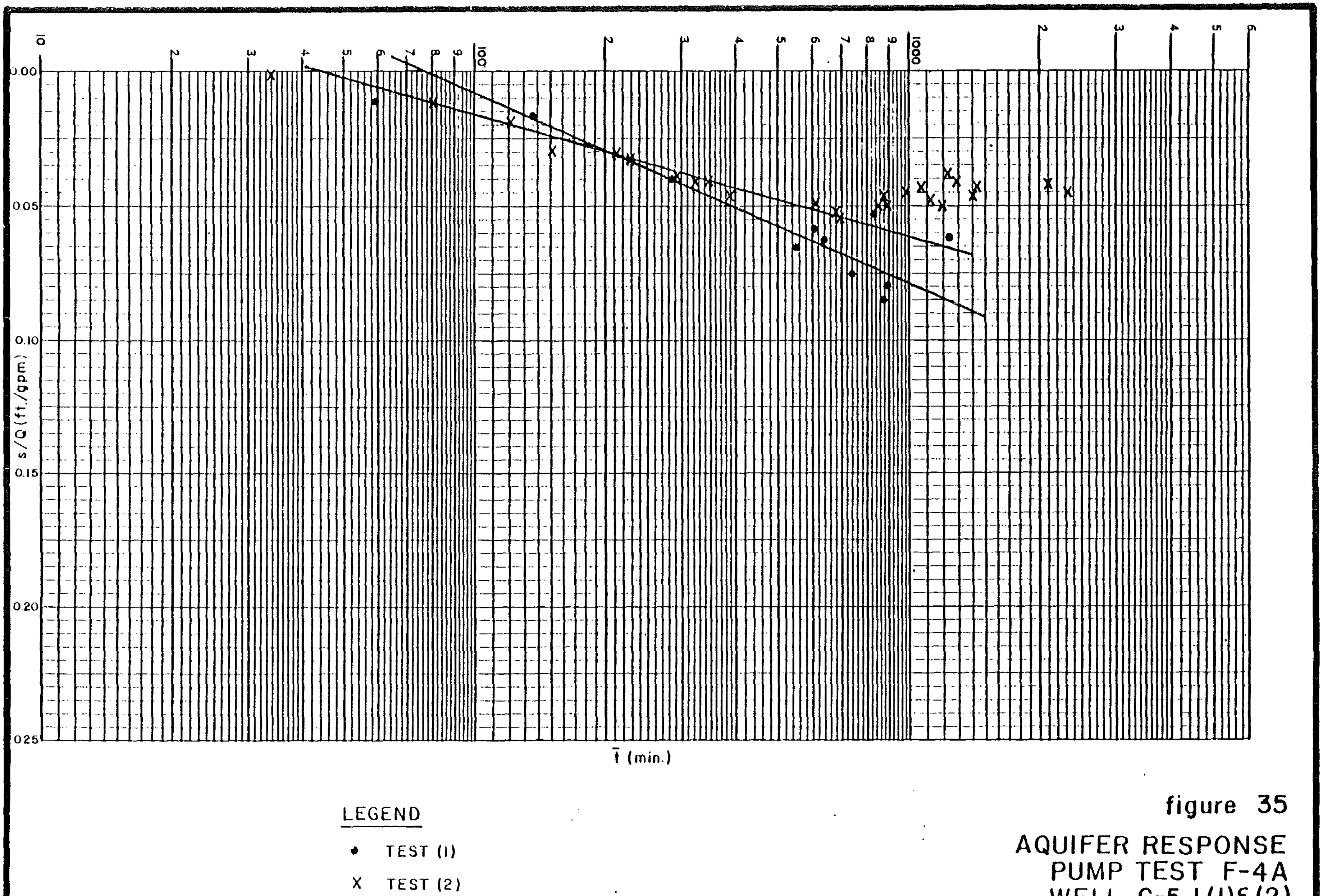

figure 35

AQUIFER RESPONSE
PUMP TEST F-4A
WELL C-5-1 (1)&(2)
Hyde Park Remedial Program

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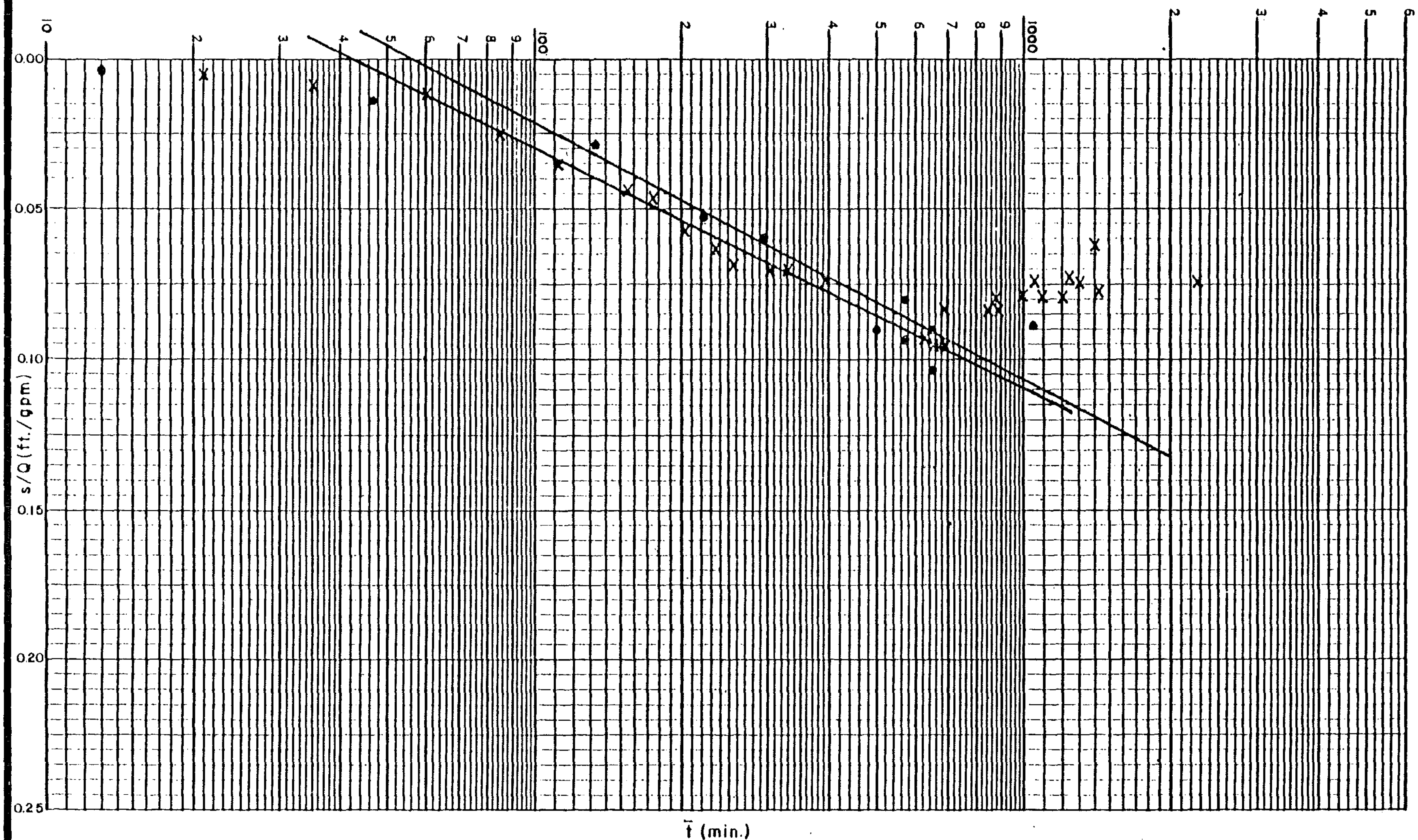

LEGEND

- ● TEST (I)
- X TEST (2)

figure 36

AQUIFER RESPONSE
PUMP TEST F-4A
WELL D-6-1(1)&(2)
Hyde Park Remedial Program

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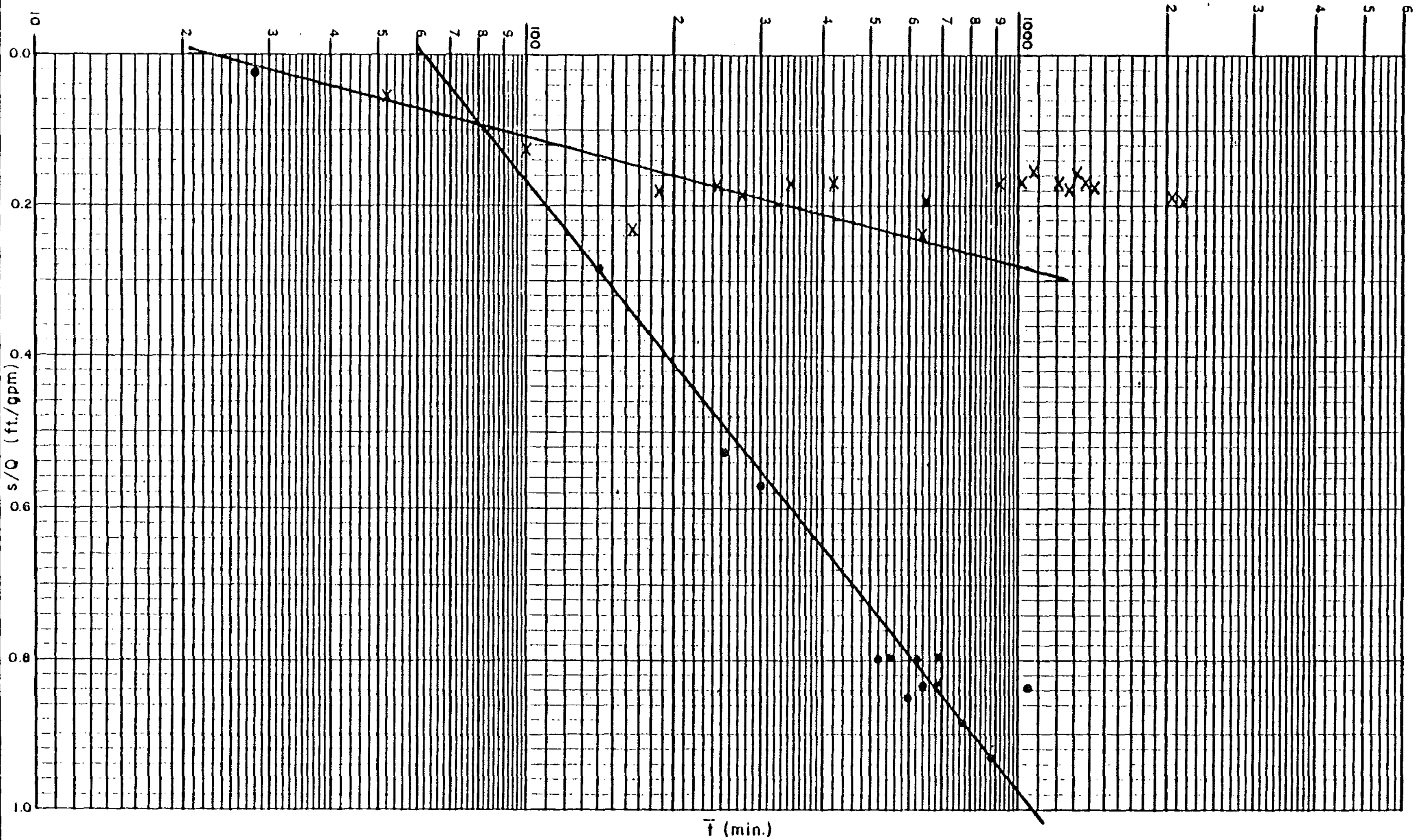
1069 - 21/03/84



LEGEND

- ● TEST (1)
- X TEST (2)

figure 37

AQUIFER RESPONSE
PUMP TEST F-4A
WELL H-3 (1) & (2)
Hyde Park Remedial Program

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1069 - 21/03/84

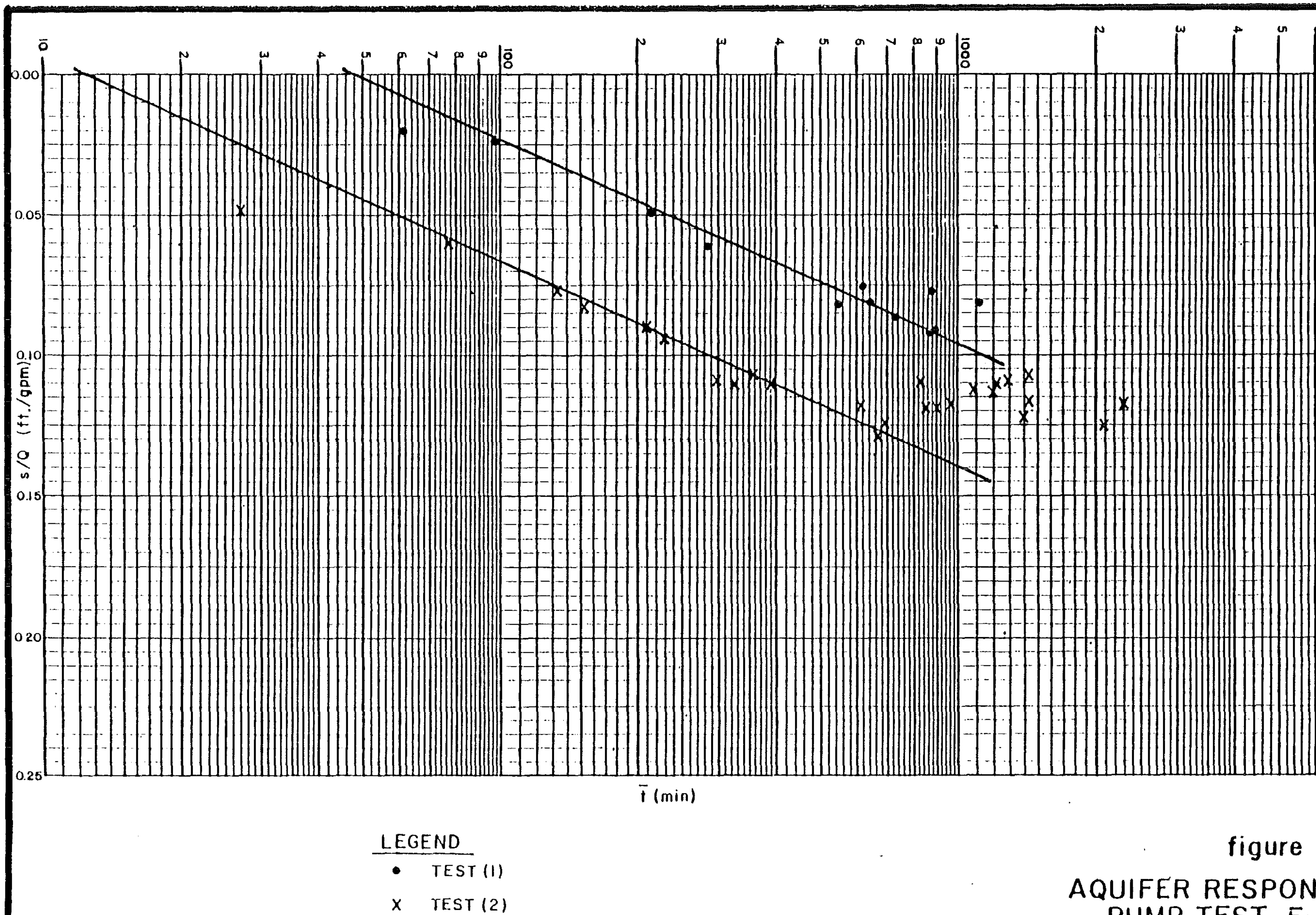

figure 38

AQUIFER RESPONSE
PUMP TEST F-4A
WELL OW18-79 (1)&(2)
Hyde Park Remedial Program

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indicating the same transmissivity value but a higher storativity value. This may have been caused by dislodging a plug of NAPL that had previously isolated a section of the aquifer thus preventing flow from this area from reaching the pumping well. It should be noted that the earlier a change in slope occurs, the closer the point of observation is to the phenomena causing the change.

During pump test D-6, wells C-5A, C-5-1, D-6-1 and OW18-79 did not show any response until ten hours after the start of the test. For distant wells a possible explanation is that time is required for the cone of influence to reach the well. However, well D-6-1, is only 51 feet from pump test well D-6. A possible explanation may be that a plug of NAPL was dislodged when sufficient gradient was created and that this opened a connection between D-6 and D-6-1 as previously discussed.

The results of wells OW13-79 and OW28-80 during pump test D-6 also illustrate an interesting feature. The initial estimates of T and S are used to determine which data meet the criteria of Equation (1). Data not meeting the criteria are deleted and new estimates of T and S are calculated using data from a flatter portion of the curve. Using these new estimates of T and S, a new value of time is calculated and it is found that data previously deleted meet the new time criteria. This phenomenum may have been created by the cone of influence meeting a recharge zone or approaching equilibrium causing the slope to flatten. Using a flatter slope will increase the estimate of T, decrease the estimate of S and decrease the magnitude of t for a given well to meet the criteria. For these wells, the second estimate of T and S will be employed to ensure that the constraint imposed by Equation (1) is satisfied.

The plot of pump test well F-4A (2), Figure 31, flattens at approximately 250 minutes. Again, this could be caused by either a recharge boundary or the system approaching equilibrium. The remaining observation wells during pump test F-4A show a decrease in the value of s/Q at $\bar{t}$ values ranging from 300 to 700 minutes, with the longer times associated with more distant wells. The "rebound" of the system could be caused by the opening of additional flow paths when plugs of NAPL were moved by an increasing gradient.

The estimates of transmissivity values in Table 7 range from 330 gpd/ft to 5867 gpd/ft with a geometric mean value of 1890 gpd/ft. Assuming an average saturated aquifer thickness of 100 feet for the areas influenced by the pump tests, the geometric mean is equivalent to a hydraulic conductivity of 2.52 feet per day. The geometric mean was chosen due to the abundance of research indicating that transmissivity estimates are lognormally distributed (Freeze 1975).

The estimates of effective storativity indicated in Table 7 range from 2.23×10^{-6} to 3.86×10^{-2} with an arithmetic mean value of 4.46×10^{-3}. Freeze (1975) lists various papers that indicate that storativity estimates are normally distributed.

It is to be noted that the results on Table 6 are representative of previously reported values for the Lockport Dolomite (Johnston, 1964; Mercer, Faust and Silka, 1981; and Maslia and Johnston, 1982).

Table 7 indicates that the transmissivity and storativity values for a particular well can vary considerably depending on the well being pumped. Since the Lockport Dolomite is a fractured flow system, pump wells in different areas of the

Lockport could be drawing water from different water bearing zones that have varying degrees of connection, both horizontally and vertically. ~~This would~~ affect the path the drawdown follows to reach the particular observation well. In addition NAPL, with its high viscosity, could effectively isolate sections of the aquifer from one well but not another. Observations indicating the effect of dislodging a plug of NAPL have been previously discussed.

The displacement of the recovery curves to the right indicate that recharge is significant in the Hyde Park area.

The results on Table 7 indicate typically exaggerated aquifer response in a Northwest by Southeast trending, revealing the possible presence of a lineament in a Northwest orientation.

6.3 AQUIFER AND FLUID PROPERTIES

For the initial steady-state groundwater flow simulation, a horizontal hydraulic conductivity value (K) for the APL of 3.0 feet per day (equivalent to a transmissivity of 2,300 gpd/ft) and a recharge value of 5 inches per year were employed.

For NAPL the estimate of conductivity was determined by employing:

$$K = k \frac{\rho g}{\mu} \qquad (8)$$

where: k = intrinsic permeability

ρ = density (gm/cm3)

μ = dynamic viscosity (centipoise)

g = gravitational acceleration

Since k is a porous medium property and g is constant, the only parameters to consider are ρ and μ. Thus we can use:

$$K_{NAPL} = K_{Water} \left(\frac{\mu}{\rho}\right)_{Water} \left(\frac{\rho}{\mu}\right)_{NAPL} \qquad (9)$$

where

The reported average values for the NAPL are ρ = 1.216 and μ = 500 to 1,000 centipoise at 25°C. By comparison, ρ = 1.0 and μ = 1.0 centipoise at 25°C for water. Thus, for the same gradient, the NAPL will move approximately 500 to 1,000 times more slowly than the groundwater.

6.4 BEDROCK EFFECTIVE PORE VOLUME

6.4.1 Introduction

In order to assess the potential impact of chemical plumes in the bedrock aquifer at the Hyde Park Landfill Site, it is necessary to estimate the quantity of liquids in the plumes.

6.4.2 Porosity of Lockport Formation

Primary permeability of rock represents the intergranular movement of groundwater. The primary porosity of unfractured carbonate rock ranges from negligible to about 1% (Freeze and Cherry, 1979). Measurable primary porosity occurs in vuggy rock usually related to ancient reefs. These features are usually small and discontinuous.

Secondary permeability of rock represents the movement of groundwater through fractures, joints and bedding planes. The secondary permeability of carbonate rocks near the ground surface is usually significant.

The range of porosities discussed below are taken from the current literature for fractured rock. These porosities do not reflect the degree of hydraulic connection through the rock formation.

Freeze and Cherry (1979) have noted that fractured rock, with well defined joint sets, have porosities up to 10%. Coarse blocky limestone has porosities up to 20% near the ground surface. Jointed limestone with solution openings have porosities which can range from 1-10% (Australian Water Resources Council, 1975). These porosities often decrease significantly with depth.

Pump tests conducted for a dewatering project at Welland Ontario (Farvolden and Nunan, 1970) done in the Lockport Formation revealed an average porosity of 16%.

An extensive number of pump tests were conducted in the Lockport Formation in the Niagara Escarpment near Milton, Ontario (Nadon, 1981). Porosities ranged from 4.3 to 9.0% with an average porosity of 6.5%.

At the Hyde Park site, the bedrock was noted to be highly fractured with frequent vuggy sections for the upper 10 to 30 feet. The porosity is expected to range from 10-20% in this zone. The remainder of the formation was observed to be infrequently fractured with little evidence of typical precipitate deposition or dissolution in the fractures which would be indicative of groundwater movement along the bedding planes. The porosity for this zone was estimated to be 1 to 10%.

6.4.3 Bedrock Pore Volume Calculations

While the above references indicate that porosities as high as 20% may occur, these values are not reflective of the actual volume of pumpable liquid within the bedrock. Much of the porosity of the Lockport Formation is encapsulated in solution cavities and discontinuous bedding planes and fracture apertures. As a result, the effective porosity of the bedrock would be considerably lower. As noted, the pump tests conducted near Milton, Ontario observed effective porosities of 4.3 to 9 percent. The 72 hour pump tests conducted at the Hyde Park site indicate considerably lower effective porosities. The observed effective porosities ranged from 4 to 2 x 10^{-4} percent with an average value of approximately 0.45 percent.

Based on an average effective porosity of 0.45 percent, the volume of recoverable liquids within the limits of the chemical plumes can be estimated. Application of an effective pore space to a specific rock mass in contact with the NAPL, APL or clean water results in an approximation of the chemical plume volumes. The enclosed Map 1 presents an areal breakdown of the groundwater volumes calculated.

It is to be noted that the volume of NAPL contained in the bedrock was typically less than one percent of the return water collected from those bedrock intervals which contained NAPL during the 15 foot incremental pump tests conducted for the aquifer survey.

6.5 ADDITIONAL PUMP TESTING

Because of interferences due to NAPL presence in the bedrock it is concluded that further small scale pump testing of the bedrock would collect data of limited value.

7.0 MODELLING OF THE HYDE PARK LANDFILL

7.1 INTRODUCTION

The following describes the models to be used to assist in the design of the remedial plans for the Hyde Park Landfill. Also detailed is the conceptual model of the site.

The use of models is advantageous for several reasons:

1. Assists in the identification of those processes or process interactions which are most likely to affect the migration of chemicals from the landfill, and thereby guide the data collection and analysis program.

2. Assists in the assessment of remedial alternatives to control chemical migration.

The two dimensional modelling approaches are as follows:

i) horizontal unconfined flow;

ii) cross-sectional variably saturated flow; and

iii) cross-sectional two-phase flow.

All models assume porous media flow.

7.2 TWO-DIMENSIONAL FLOW IN THE HORIZONTAL PLANE

Flow of water in the Lockport Formation is assessed using a two-dimensional steady state integrated depth phreatic model. The model is valid for aquifers in which the variations in the vertical plane are small relative to the average saturated thickness. Areal recharge/discharge are included in the model. The governing flow equation of the model is

$$\frac{\partial}{\partial x_i}\left((h - \eta(x))K_{ij}\frac{\partial h}{\partial x_j}\right) + Q = 0 \;; \quad i,j = 1,2 \qquad (10)$$

where:

h = hydraulic head (L)

$\eta(x)$ = elevation of the aquifer base (L)

K_{ij} = saturated hydraulic conductivity tensor (L/T)

Q = areal recharge/discharge (L/T)

x_i = components of cartesian coordinate directions x_1, and x_2

The boundary conditions used for the solution of Equation (10) are:

$$h = \hat{h} \text{ on } \Gamma_1 \tag{11a}$$

$$(h - \eta(x))K_{ij} \frac{\partial h}{\partial x_j} \cdot n_i = \hat{q}_n \text{ on } \Gamma_2 \tag{11b}$$

where:

$\hat{h}$ = prescribed hydraulic head on surface Γ_1

$\hat{q}_n$ = prescribed fluid flux normal to surface Γ_2

n_i = components of unit outward normal to surface Γ_2

For the spatial domain, the external boundary Γ is defined by $\Gamma = \Gamma_1 + \Gamma_2$. The model assumes isothermal conditions and that the density and viscosity of the aquifer fluid are constant.

The system equations are solved using the Galerkin finite element method (Sykes, 1980). The model uses a mixed isoparametric quadrilateral element for discretization. Element sides can be linear, quadratic or cubic. The elevation of the aquifer base $\eta(x)$ is treated as a functional coefficient. Recharge/discharge and components of the hydraulic conductivity tensor K_{ij} can vary from element to element.

Sensitivity of a performance measure or system response function to the system parameters is accomplished using the adjoint method (Sykes et al., 1983). Two performance measures are considered:

$$P = \{ g \}^T \{ h \} \tag{12}$$

and

$$P = (q_1^2 + q_2^2)^{1/2} \tag{13}$$

where $\{g\}^T$ is a row vector of user defined nodal weights; $\{h\}$ is a column vector of nodal hydraulic heads; and q_1 and q_2 are the components of Darcy velocity in a user selected element. Equation (12) represents a hydraulic head performance measure while Equation (13) is the magnitude of Darcy velocity in an element. The marginal sensitivity of P to a parameter α is given by $dP/d\alpha_k$. The normalized sensitivity coefficients are defined by $d(\ln P)/d(\ln \alpha_k)$. The adjoint model calculates the sensitivity of the selected performance measures to the element hydraulic conductivities, element recharge/discharge, and the prescribed hydraulic heads $\hat{h}$ on Γ_1.

The primary flow problem of Equation (10) has been validated in Sykes (1980).

7.3 FLOW IN VARIABLY SATURATED CROSS-SECTIONS

Steady-state groundwater flow is assessed in selected cross-sections of the Hyde Park site. The governing variably saturated flow equation is:

$$\frac{\partial}{\partial x_i}\left[K_{ij}K_r(Sw)\left(\frac{\partial \psi}{\partial x_j}+\frac{\partial Z}{\partial x_j}\right)\right] = 0 \; ; \; i,j = 1,2 \qquad (14)$$

with boundary conditions

$$\psi = \hat{\psi} \text{ on } \Gamma_1 \qquad (15a)$$

$$K_{ij}K_r(Sw)\left(\frac{\partial \psi}{\partial x_j}+\frac{\partial Z}{\partial x_j}\right)\cdot n_i = \hat{q}_n \text{ on } \Gamma_2 \qquad (15b)$$

subject to $\Gamma = \Gamma_1 + \Gamma_2$

In Equations (14) and (15),

$K_r(Sw)$ = relative hydraulic conductivity, a function of the degree of water saturation Sw with $0 < K_r(Sw) < 1.0$

ψ = pressure head (L)

Z = datum (L)

K_{ij} = saturated hydraulic conductivity tensor (L/T)

Equation (15a) represents prescribed pressures on surface Γ_1 and Equation (15b) represents a prescribed flow normal to boundary Γ_2.

Equations (14) and (15) assume that the water is the wetting fluid at pressure ψ and the air the non-wetting phase at atmospheric pressure throughout the unsaturated zone. The solid matrix is assumed to be rigid and stable. The pore fluid is assumed to have a constant density and viscosity and to be incompressible over the range of pressures considered.

The solution of Equation (14) with (15) is based on the Galerkin finite element method and the isoparametric quadrilateral element (Sykes, 1980). Saturated hydraulic conductivities can vary from element to element. The relative hydraulic conductivity $K_r(Sw)$ is a functional coefficient in the model.

7.4 TWO-PHASE FLOW

7.4.1 Model Description

A two-phase flow model WSTIF (Waterloo Simulator for Two Immiscible Fluids) developed by Osborne (1984) is used to analyze the simultaneous flow of APL and NAPL in a selected cross-section at the Hyde Park site. The model equations are:

$$\Theta \frac{dSw}{dPc} \left(\frac{\partial Pn}{\partial t} - \frac{\partial Pw}{\partial t} \right) = \frac{\partial}{\partial x_i} \left[\lambda^w_{ij} \left(\frac{\partial Pw}{\partial x_j} + \rho_w \; g \frac{\partial Z}{\partial x_j} \right) \right] ; \quad i,j = 1,2 \tag{16}$$

$$\Theta \frac{dSw}{dPc} \left(\frac{\partial Pw}{\partial t} - \frac{\partial Pn}{\partial t} \right) = \frac{\partial}{\partial x_i} \left[\lambda^n_{ij} \left(\frac{\partial Pn}{\partial x_j} + \rho_n \; g \frac{\partial Z}{\partial x_j} \right) \right] ; \quad i,j = 1,2 \tag{17}$$

$$Sn + Sw = 1 \tag{18}$$

$$Pc = Pn - Pw \tag{19}$$

where

Θ = effective porosity

Sw = degree of saturation of the wetting fluid (APL)

Pn = pressure of the non-wetting fluid (NAPL)

Pw = pressure of the wetting fluid

Pc = capillary pressure, a nonlinear function of the degree of saturation of the wetting fluid (Sw)

λ^w_{ij} = $k_{ij} \dfrac{k_r(Sw)}{\mu_w}$ = mobility of the wetting fluid

k_{ij} = components of permeability tensor

$k_r(Sw)$ = relative permeability to wetting fluid subject to $0 < k_r(Sw) < 1.0$

μ_w = dynamic viscosity of the wetting fluid

λ^n_{ij} = $k_{ij} \dfrac{k_r(Sn)}{\mu_n}$ = mobility of the non-wetting fluid

$k_r(Sn)$ = relative permeability to the non-wetting fluid subject to $0 < k_r(Sn) < 1.0$

Sn = degree of saturation of the non-wetting fluid

μ_n = dynamic viscosity of the non-wetting fluid

ρ_w = density of wetting fluid

ρ_n = density of non-wetting fluid

g = gravitational constant

Z = datum

x_i = components of cartesian coordinate directions x_1 and x_2

Initial conditions for the model are developed from

$$\frac{\partial}{\partial x_i}\left[\lambda^w_{ij} \left(\frac{\partial Pw}{\partial x_j} + \rho_w g \frac{\partial Z}{\partial x_j} \right) + \lambda^n_{ij} \left(\frac{\partial Pc}{\partial x_j} + \frac{\partial Pw}{\partial x_j} + \rho_n g \frac{\partial Z}{\partial x_j} \right) \right] = 0 \qquad (20)$$

$$Pw = \hat{P}w \text{ on } \Gamma_1 \qquad (21a)$$

$$\lambda^w_{ij} \left(\frac{\partial Pw}{\partial x_j} + \rho_w g \frac{\partial Z}{\partial x_j} \right) . n_i = \hat{q}_w \text{ on } \Gamma_2 \qquad (21b)$$

and Pc specified over $x_i \varepsilon D$. Equation (20) is a result of the addition of Equations (16) and (17) with Equation (19). The boundary condition of Equation (21a) represents a prescribed initial pressure Pw on boundary Γ_1 for the wetting fluid while Equation (21b) represents a flux q_w of the wetting fluid on Γ_2 at $t = 0$.

The boundary conditions for the transient problem of Equations (16) to (19) are

$$Pw = \hat{P}w \quad \text{on } \Gamma_1 \tag{22a}$$

$$Pn = \hat{P}n \quad \text{on } \Gamma_2 \tag{22b}$$

$$\lambda^{w}_{ij} \left(\frac{\partial Pw}{\partial x_j} + \rho_w \, g \, \frac{\partial Z}{\partial x_j} \right) . \; n_i = \hat{q}_w \text{ on } \Gamma_3 \tag{22c}$$

$$\lambda^{n}_{ij} \left(\frac{\partial Pn}{\partial x_j} + \rho_n \, g \, \frac{\partial Z}{\partial x_j} \right) . \; n_i = \hat{q}_n \text{ on } \Gamma_4 \tag{22d}$$

where the boundary of the domain D is Γ with $\Gamma = \Gamma_1 + \Gamma_3$ and $\Gamma = \Gamma_2 + \Gamma_4$. Equations (22a) and (22b) represent prescribed pressures for the wetting fluid and non-wetting fluid on surfaces Γ_1 and Γ_2 respectively. Equations (22c) and (22d) are prescribed fluxes for the wetting fluid and non-wetting fluid on surfaces Γ_3 and Γ_4 respectively.

The two-phase model described in the preceding paragraphs analyzes the simultaneous flow of two immiscible fluids in the porous medium domain. The interfacial tension between the wetting phase and non-wetting phase fluid is nonzero with a capilliary pressure difference existing across the fluid-fluid interface. The capillary pressure is a non-hysteretic function of the degree of saturation of the wetting fluid in the current model.

The two-phase flow system equations are solved implicitly using the finite element method and the mixed isoparametric element (Sykes, 1980).

Upstream weighting as described by Huyakorn and Pinder (1978) was used for the spatial terms of Equations (16) and (17). The wetting fluid mobility, non-wetting fluid mobility and quantity dSw/dPc are functional coefficients in the model. The permeability tensor k_{ij} components, the porosity Θ, the non hysteretic Sw-Pc relationship, the Sw-k_r(Sw) relationship and the Sn-k_r(Sn) function are constant within an element but can vary from element to element thus allowing the modelling of heterogeneous systems. Rapid convergence of the nonlinear equations is achieved using a Newton-Raphson solution scheme and a Gauss Elimination solver.

7.4.2 Model Verification

The two-phase flow model WSTIF described in the preceeding section was verified by comparing the model results with those of the one-dimensional finite difference two-phase flow simulator used by Arthur D. Little Inc. (1983). A one-dimensional vertical column, 23 feet in length, intersecting three lithologies, as shown in Figure 39, was modelled. The boundary conditions, for both models include prescribed aqueous phase pressures (Pw) and degrees of saturation (Sw) at the upper and lower ends of the column. The vertical boundaries of the column were specified in WSTIF as zero flux boundaries to both the aqueous and non-aqueous phases.

The initial NAPL distribution at time t_0 used by Arthur D. Little Inc. was duplicated for WSTIF. The media and liquid properties used in both models are listed below:

porosity	20 percent for all lithologies
viscosity, NAPL	1 centipoise
viscosity, APL	1 centipoise
density, NAPL	1.5 g/cc
density, APL	1.0 g/cc

The functions relating capillary pressure and relative permeabilities of both phases to the degree of saturation were similar for both models. Simulation of the downward migration of the NAPL under the effects of gravity and a downward APL head of two

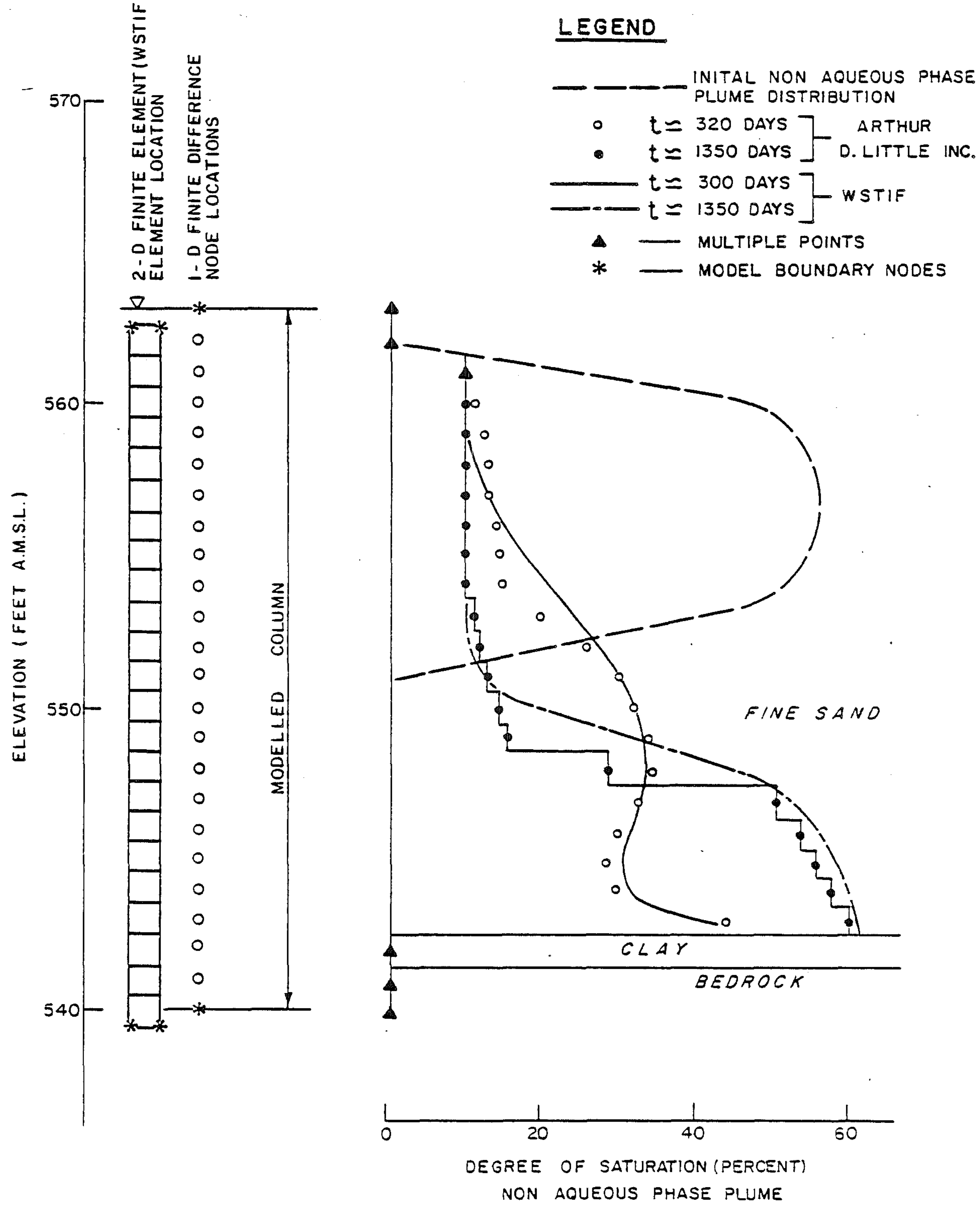

figure 39

ONE - DIMENSIONAL MODEL VERIFICATION
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feet results in a redistribution of the degree of saturation profile with time.

Comparision of the saturation profiles predicted by the two models indicate close agreement as shown in Figure 39.

7.5 CONCEPTUALIZATION OF GROUNDWATER FLOW

The regional flow system has been previously documented by Johnston (1964).

The Lockport Formation in the vicinity of Hyde Park is modelled as an equivalent porous medium phreatic aquifer. This conceptualization was chosen due to:

1) insufficient data describing fracture widths, orientation and interconnection and
2) the measured potentiometric surface is within the Lockport Formation throughout much of the Hyde Park area.

Analyses in the plan view assumed a homogeneous, isotropic aquifer. Even though these conditions may not exist, the analysis of the site based on these assumptions will provide insight to system response under varying conditions.

The base of the aquifer is considered to be the elevation of the top of the Rochester Shale as shown in Figure 40. The thickness of the Lockport Dolomite is shown in Figure 41.

The geographic area simulated by the two-dimensional areal model, as shown on enclosed Map 2, is approximately 12,000 feet wide at the southern boundary narrowing to 4,000 feet at the northern boundary in a distance of approximately 8,700 feet. The areal discretization consists of 340 elements and 378 nodes.

Various types of boundary conditions can be employed in the integrated depth finite element flow model. These include prescribed (constant) head, constant flow and no flow boundaries. The west, north and east boundaries are specified as prescribed head boundaries. The west boundary has been initially set at 2 feet above the aquifer base elevation due to the significant vertical fracturing (Maslia and Johnston 1982) of the Lockport Formation near the Niagara River Gorge. The north and east boundaries are set at the average water elevations in

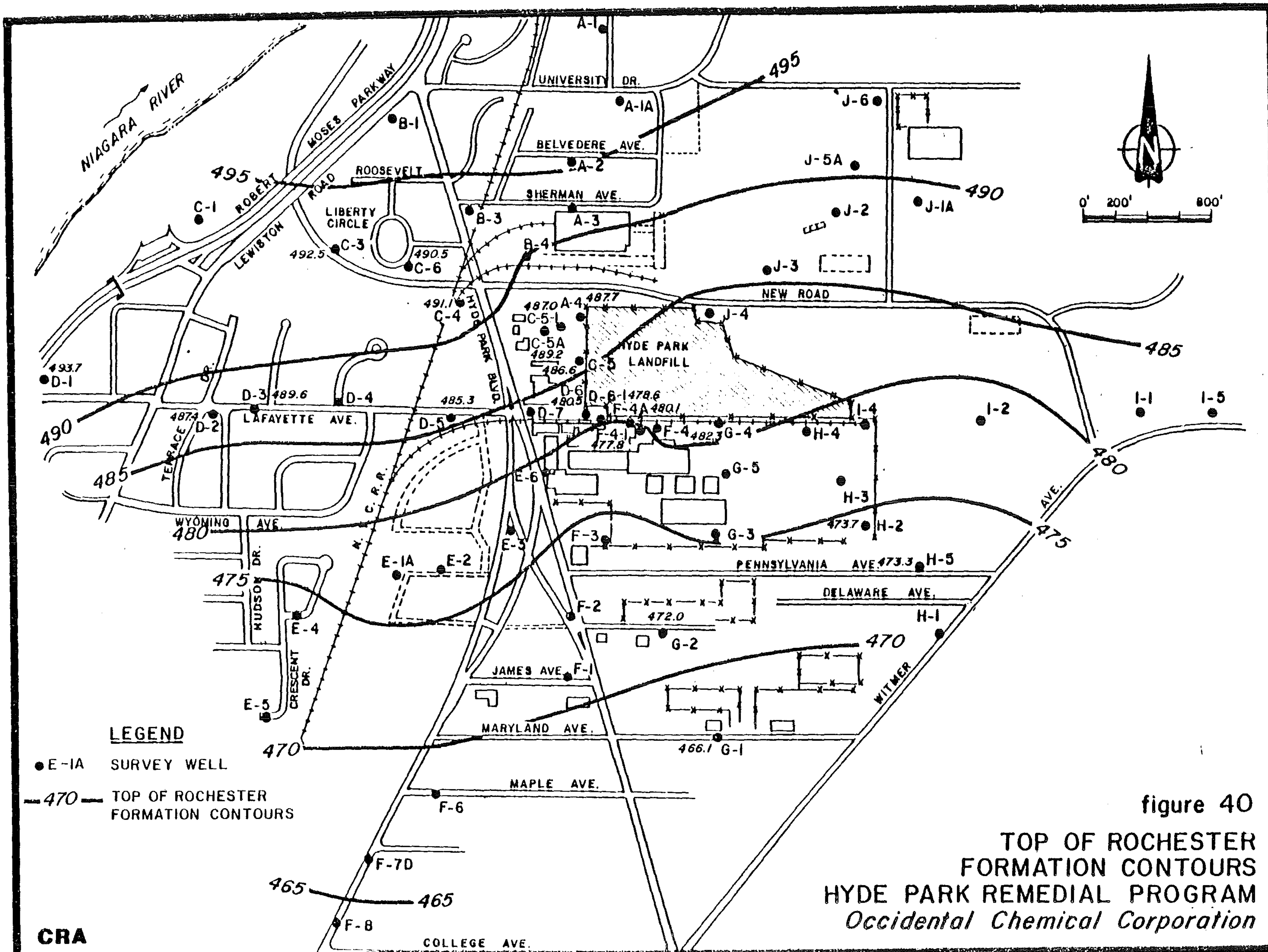

figure 40

TOP OF ROCHESTER FORMATION CONTOURS
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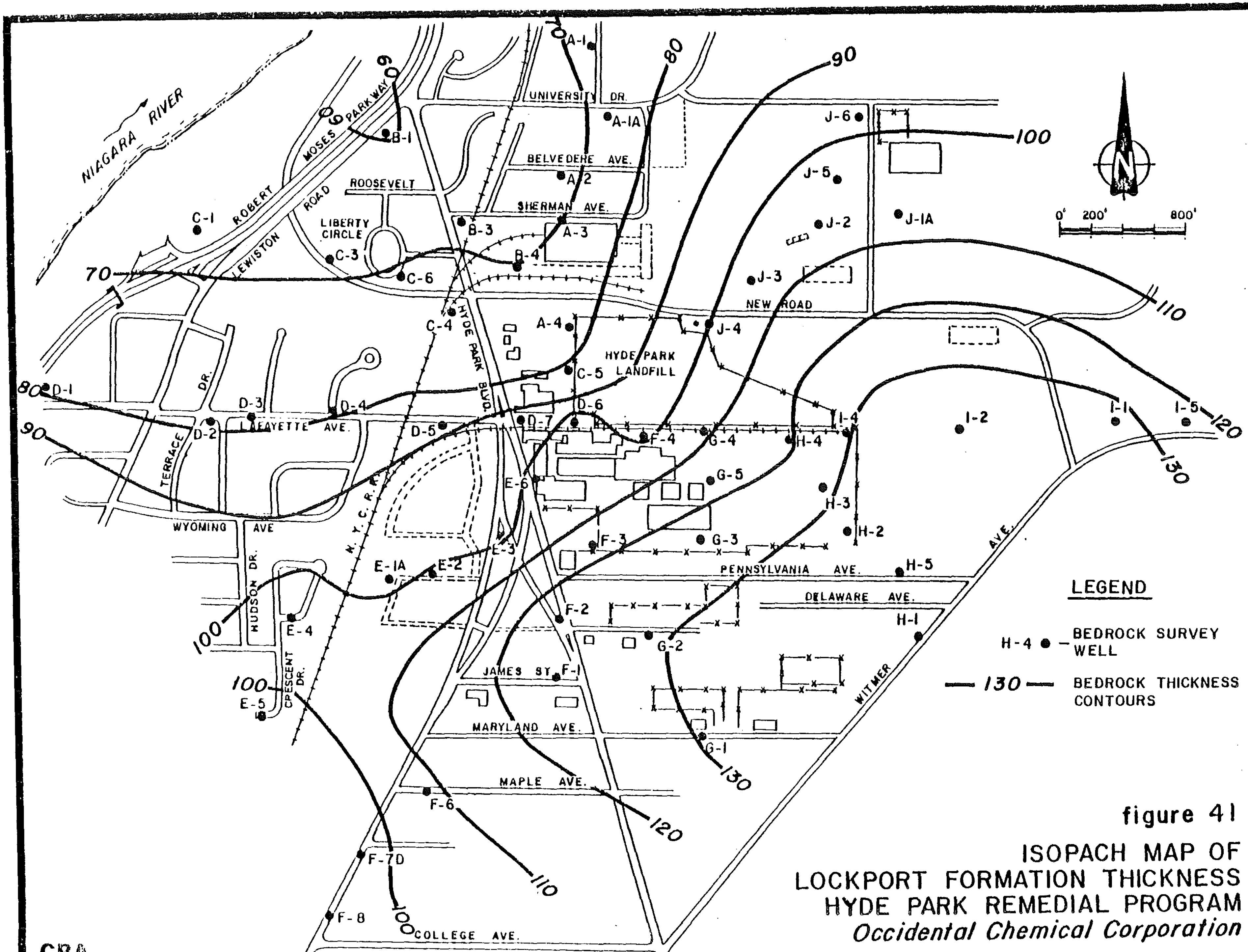

figure 41

ISOPACH MAP OF LOCKPORT FORMATION THICKNESS
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the power canal of 554 feet as indicated in Uhl et al (1960). Due to a lack of information on the southern boundary, both prescribed-head and no-flow boundary conditions were simulated to determine which best represents the boundary.

The areal model is an integrated-depth phreatic model and considers saturated thicknesses only. Recharge to the modelled area is due to infiltration from precipitation only. Discharge occurs at the prescribed head boundaries.

For cross-section analyses, sections AA-AA' and BB-BB' are employed. Section AA-AA' as illustrated on Map 3 is employed in the simulation of single phase flow. Section BB-BB' as illustrated on Map 4 is employed in the simulation of two phase flow.

The vertical discretization consists of five units:

1) The glacial till overburden
2) An upper Lockport unit
3) A lower Lockport unit
4) Gasport unit
5) Decew unit

The Lockport was divided into two units due to the significant degree of weathering in the upper portion of the unit. This weathered zone corresponds approximately to the upper 30 feet of the Lockport Formation.

The single phase cross-sectional model considers variably saturated conditions. Two suction head (ψ) and degree of saturation (Sw) functions were used in the calibration process. In the absence of better information the curves employed are those illustrated on Figure 5 of the report entitled, "S-Area Two-Phase Flow Model" by Arthur D. Little Inc., dated June, 1983.

The boundary conditions employed in the single phase cross-sectional model are seepage face conditions for boundary AB, spatially varying flux for boundary BC, a groundwater divide (no flow) boundary for CD and an impermeable boundary for AD as shown on Map 3.

The spatial discretization consists of 441 elements and 500 nodes.

7.6 SITE ASSESSMENTS

7.6.1 Two Dimensional Areal Model

Review of the preliminary results of the areal two-dimensional single-phase flow model indicate that a prescribed head condition is more representative of the southern boundary than a no-flow condition. When a no-flow boundary was employed, a groundwater mound was created on the southern boundary that does not exist in the field. Employing a prescribed-head boundary eliminated this phenomenum.

To reproduce the measured phreatic surface, the modelled area was divided into areas of developed and undeveloped land as indicated on Map 1. Recharge values of 3 and 7 inches per year were used for the developed and undeveloped land respectively. In comparison, Maslia and Johnston (1982) used a spatially constant recharge of 5 in/yr. A hydraulic conductivity of 1.0 ft/day was used for the entire modelled domain. This hydraulic conductivity is within the range of the values estimated from the pumping tests. Field data are insufficient to develop spatially varying hydraulic conductivities.

Figures 42 and 43 illustrate the phreatic water table elevations and Darcy velocities respectivly obtained using the selected values for hydraulic conductivity and recharge. The results indicate a modelled groundwater mound centered at node 174 with a water table elevation of 605.14 feet AMSL. The highest measured water table elevations for wells in the bedrock aquifer consistently occur at OW11-79 ranging from 601.84 to 613.14 feet AMSL. OW11-79 is approximately 100 feet east of node 174. The modelled water table elevations reflect the average yearly conditions.

Figures 42 and 43 indicate a groundwater divide running from node 13 on the southern boundary to node 353 near the northern boundary. Thus the groundwater velocities through the area of the landfill itself are small as the principal source of water to the landfill is infiltration due to precipitation. Little base flow enters the landfill area due to the proximity of the groundwater divide.

Sensitivity analysis by the adjoint method (Sykes et al 1983) was performed for the performance measures of phreatic head and Darcy velocity utilizing the calibrated parameter values.

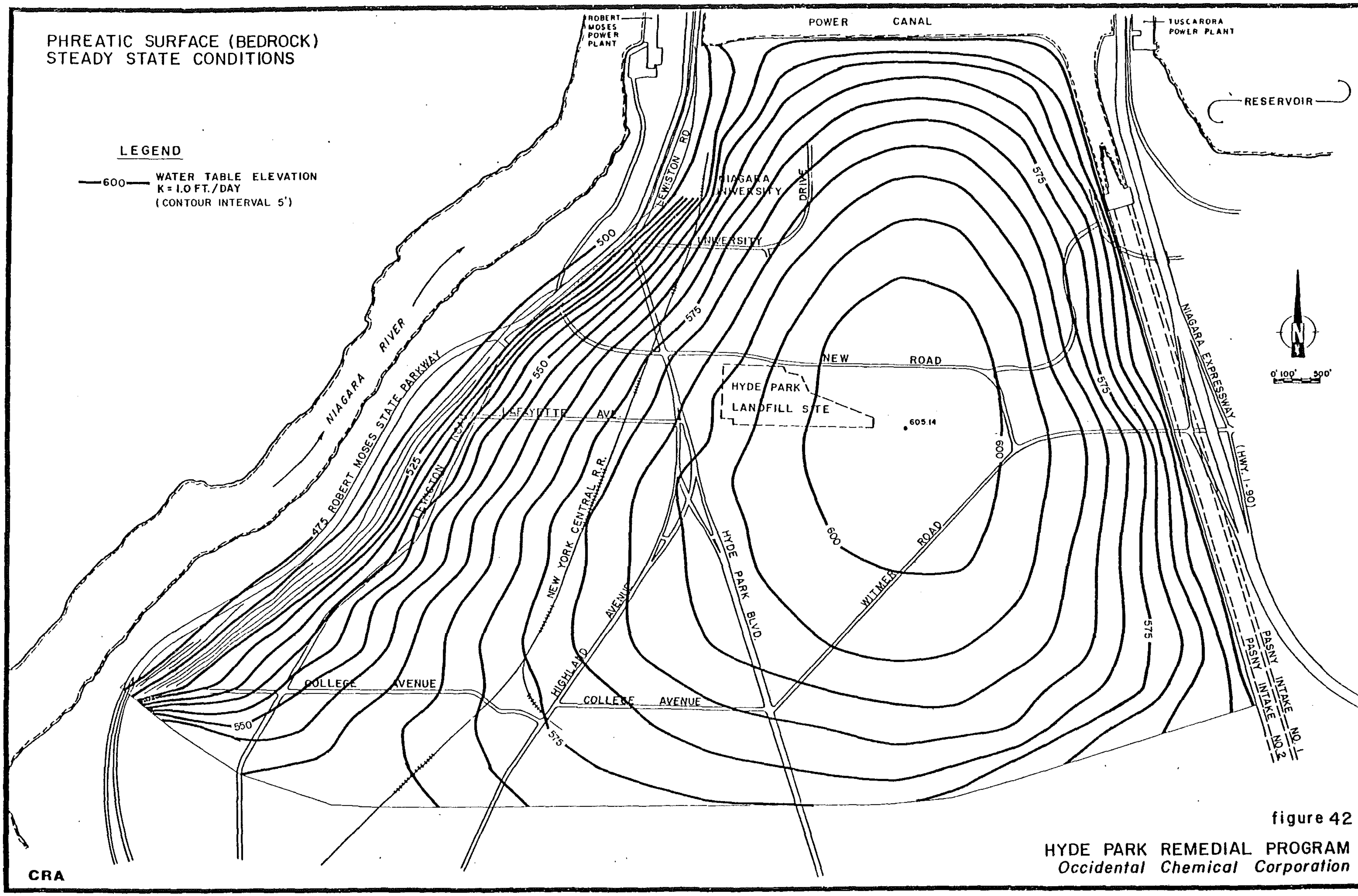

figure 42

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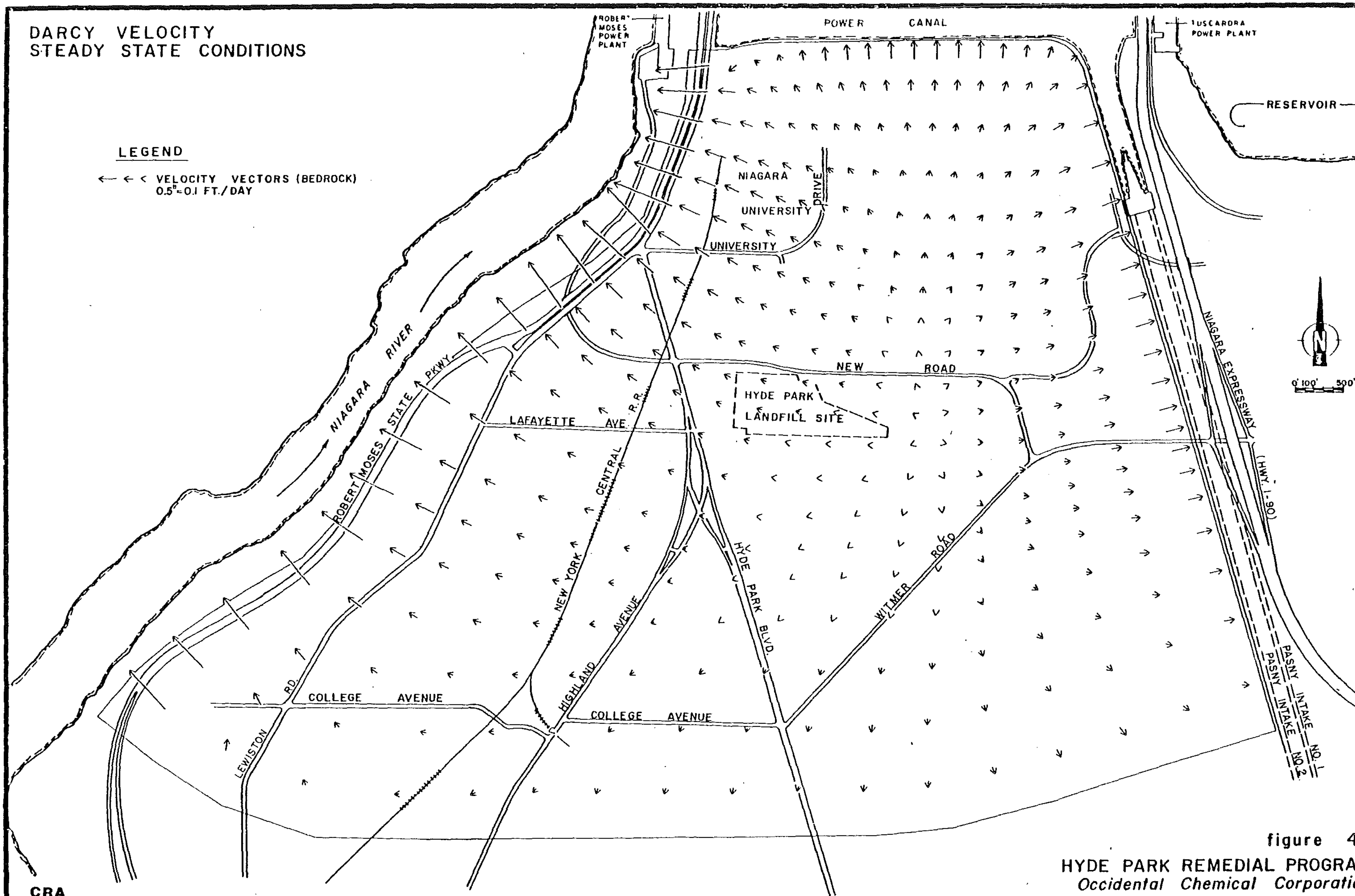

figure 43

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For the phreatic head performance measure

$$P = \{ g \}^T \{ h \}$$

the vector $\{g\}^T$ of user defined nodel weights were set equal to unity at nodes 170, 171, 188 and 189 and zero elsewhere.

For the Darcy velocity performance measure

$$P = (q_1^2 + q_2^2)^{\frac{1}{2}}$$

element 161 was chosen as the element of concern. These nodes and elements were chosen since it is the water table elevations and velocities in the area of the landfill that provide the driving force for mass flux of chemicals from the site.

The results of the normalized sensitivity coefficients, previously defined, are illustrated in Figures 44 to 47. The normalized sensitivity coefficients represent the percentage change in the performance measure for a one percent change in the parameter magnitude. To obtain the percent change in the performance measure that would occur if the parameter values were changed for an entire area, the individual values of the normalized

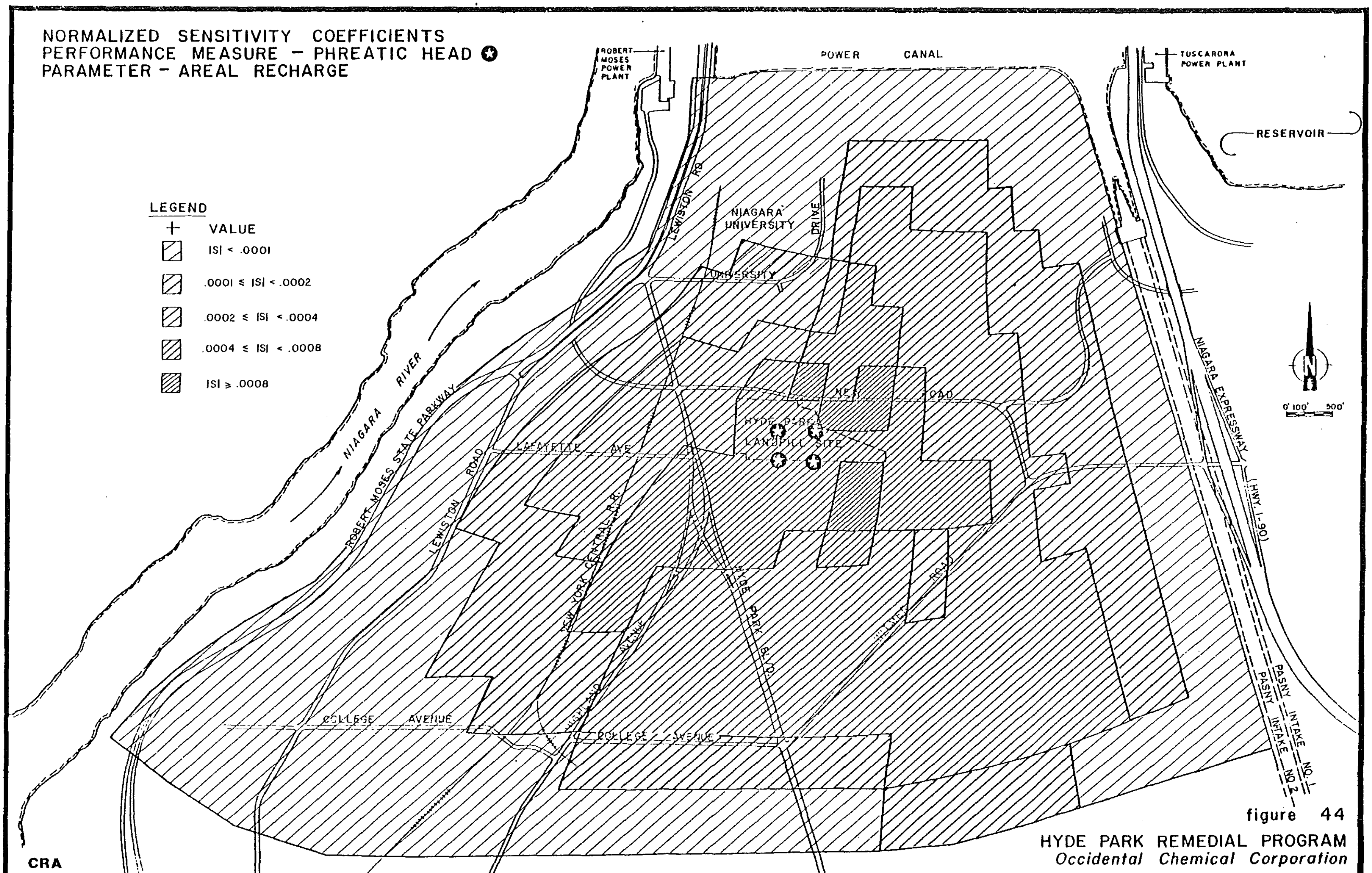

figure 44

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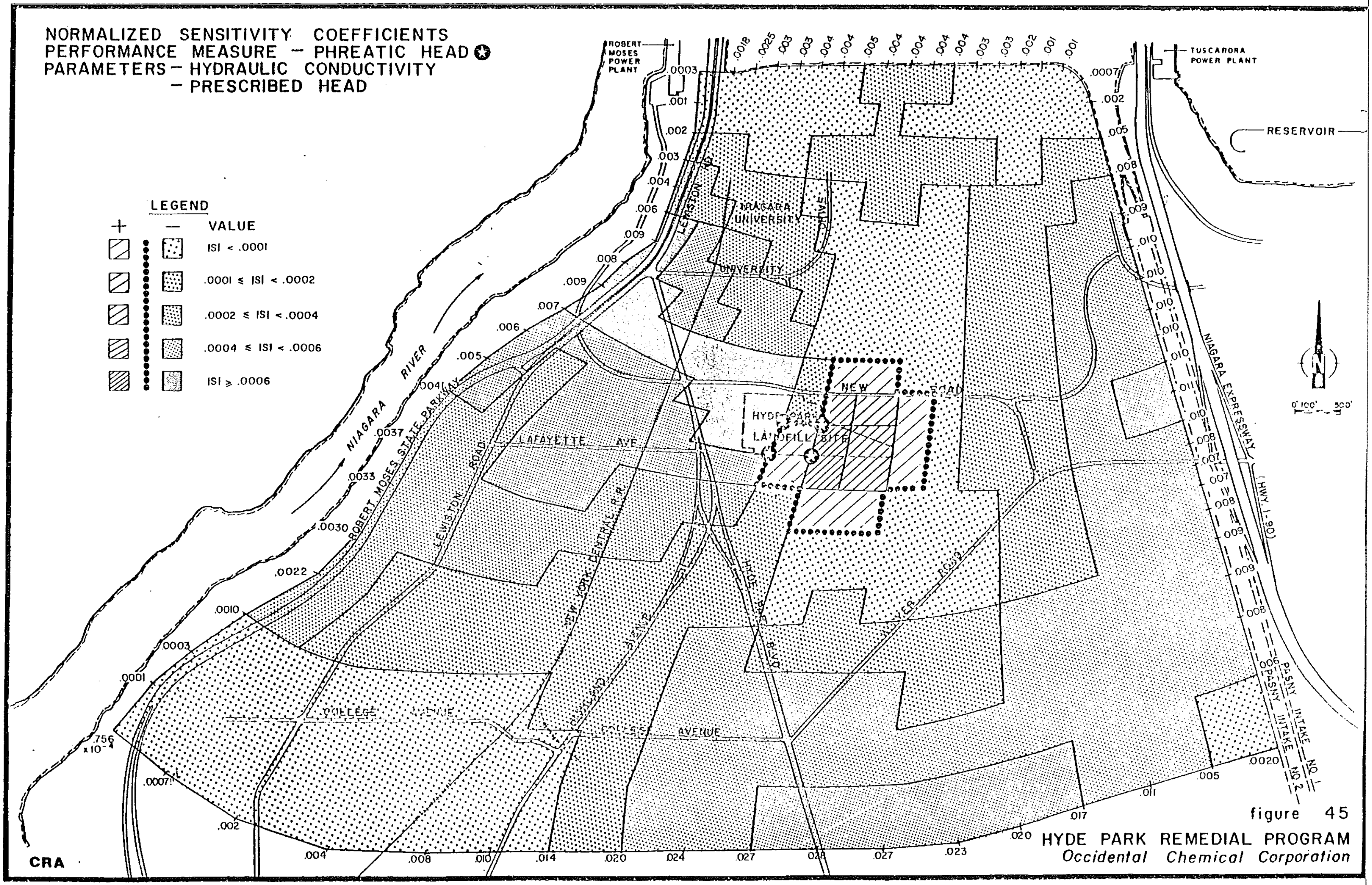

NORMALIZED SENSITIVITY COEFFICIENTS
PERFORMANCE MEASURE – PHREATIC HEAD
PARAMETERS – HYDRAULIC CONDUCTIVITY
– PRESCRIBED HEAD
LEGEND
+ – VALUE
|S| < .0001
.0001 ≤ |S| < .0002
.0002 ≤ |S| < .0004
.0004 ≤ |S| < .0006
|S| ≥ .0006
ROBERT MOSES POWER PLANT
TUSCARORA POWER PLANT
RESERVOIR
NIAGARA RIVER
ROBERT MOSES STATE PARKWAY
LEWISTON ROAD
NIAGARA UNIVERSITY
UNIVERSITY
NEW ROAD
HYDE PARK LANDFILL SITE
LAFAYETTE AVE
NEW YORK CENTRAL R.R.
HYDE PARK BLVD
COLLEGE AVENUE
NIAGARA EXPRESSWAY (HWY I-90)
PASNY INTAKE NO 1
PASNY INTAKE NO 2
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figure 45
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NORMALIZED SENSITIVITY COEFFICIENTS
PERFORMANCE MEASURE – DARCY VELOCITY ✪
PARAMETER – AREAL RECHARGE

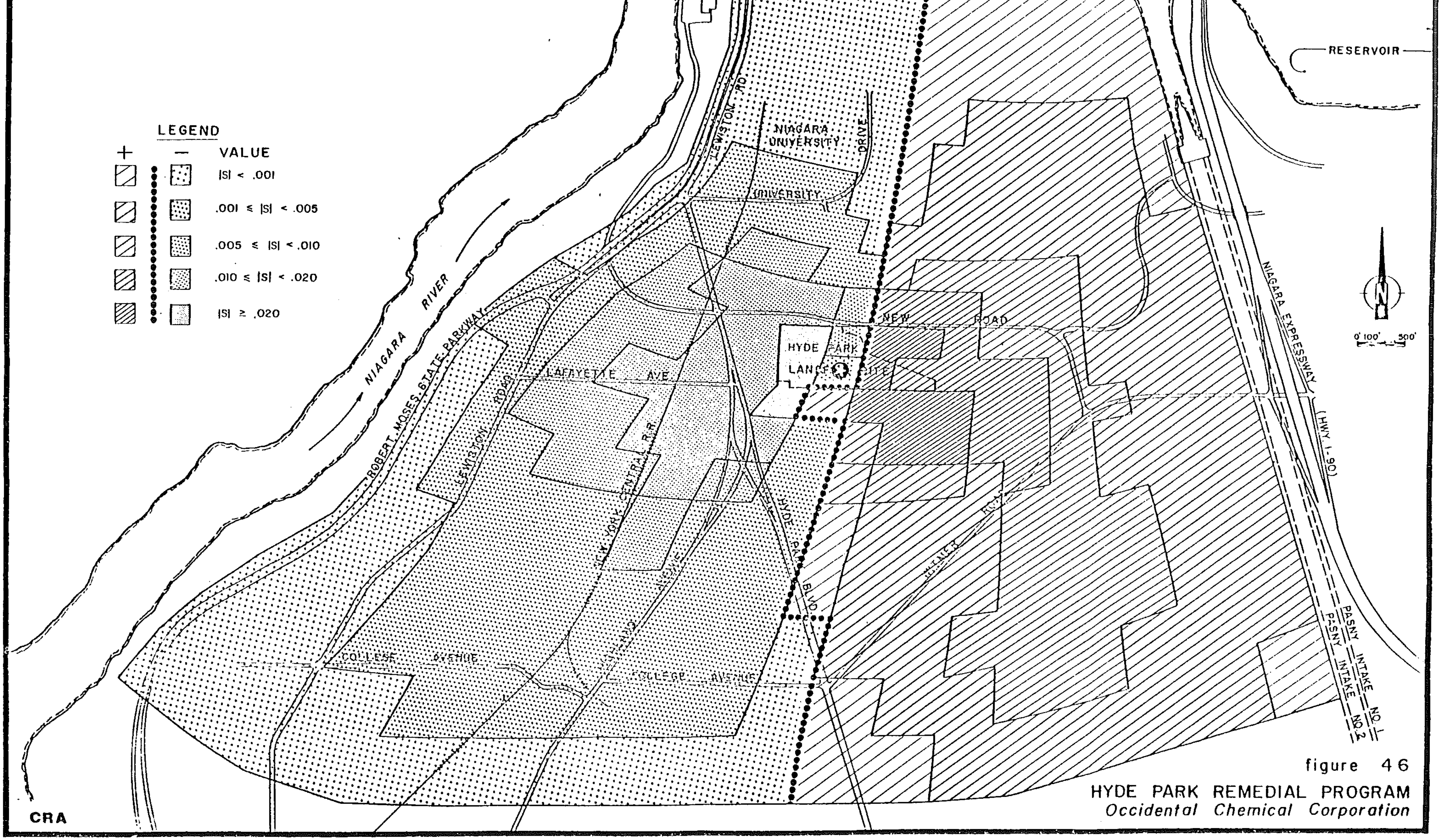

figure 46
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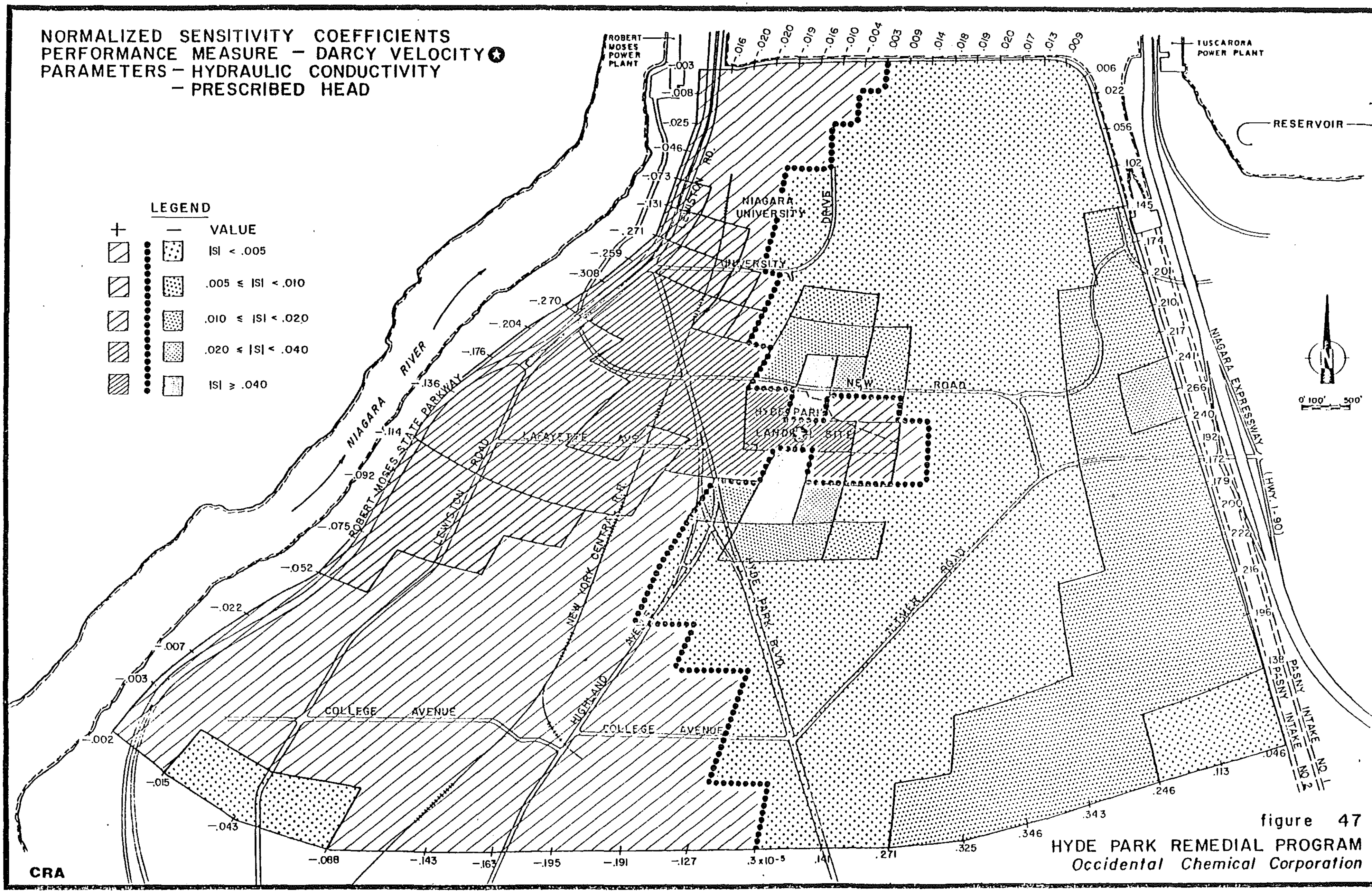

figure 47

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sensitivity coefficient for the elements (nodes) are summed. Negative sensitivity coefficients indicate that parameter increases/decreases will result in a decrease/increase in the performance measure. By comparing the normalized sensitivity coefficients the relative importance of the system parameters to the performance measures can be determined. For example, the importance of the element hydraulic conductivities and recharge and the prescribed nodal heads in determining the groundwater velocity beneath the landfill can be readily calculated. As a consequence the sensitivity coefficients can be used as an indicator of the regions and parameters where accurate characterization is beneficial in reducing performance measure uncertainty. The normalized sensity coefficients represent local derivatives. That is, they are a function of the parameters and discretization used in the primary flow problem.

A check of the sensitivity coefficients at several design points was made. Table 8 shows the result of varying infiltrations on the normalized sensitivity coefficients. For the analysis, a constant hydraulic conductivity of 1.0 ft/day was used. The values of Table 8 indicate that the sensitivity coefficients are relatively constant over the range of feasible recharge perturbations. The

TABLE 8

SAMPLE OF ADJOINT RESULTS DURING AREAL
TWO-DIMENSIONAL FLOW MODEL CALIBRATION

Run	Zone	Recharge (in./year) (R)	Normalized Sensitivity Coefficient Summed over the zone (S)	Percent Change in Recharge from Previous Run (Y)	Performance Measure Estimated (Pe)	Performance Measure Calculated (Pc)
1	1	4.0	.0351			2407.34
	2	6.5	.0343			
2	1	4.0	.0344	0.0	2413.69	2412.32
	2	7.0	.0362	+ 7.7		
3	1	3.0	.0272	-25.0	2391.57	2391.90
	2	7.0	.0381	0.0		

$$P_{e_i} = {}_j P_{c_{i-1,j}} \left((R_{i,j} - R_{i-1,j}) / R_{i-1,j} * S_{i-1,j} + 1 \right)$$

$$= {}_j P_{c_{i-1,j}} * \left(\frac{Y_{ij}}{100} * S_{i-1,j} + 1 \right)$$

i = 1, ..., # of runs

j = 1, ..., # of zones

sensitivity coefficients can thus be used to calculate, with a relatively small error, the expected change in the performance measure from parameter perturbations.

Figure 44 illustrates the normalized sensitivity coefficient of phreatic head with respect to areal recharge. The results indicate that the stated performance measure is most sensitive to changes in recharge in the areas immediately surrounding the location of the chosen performance measure with the most sensitive areas being directly upgradient. As one moves away from the site, the performance measure becomes less sensitive to recharge. The normalized sensitivity coefficient with the largest magnitude occurred in element 129 with a value of .0011. This indicates that a one percent change in recharge would produce a .0011 percent change in the sum of the phreatic head at nodes 170, 171, 188 and 189, the chosen performance measure.

Figure 45 illustrates the normalized sensitivity coefficients of phreatic head with respect to hydraulic conductivity and prescribed head. The results for the hydraulic conductivity indicate two major trends; one following the north-south groundwater divide and the other following the east-west flow line that passes through the element described by the nodes of the performance measure. The values along the flow line east of the groundwater divide are negative and increase in magnitude as one travels from the divide to the PASNY conduits. This indicates that an increase in hydraulic conductivity will decrease the magnitude of the performance measure. West of the divide to the location of the performance measure the values are positive and increase in magnitude for those elements immediately surrounding the performance measure location. All the remaining values for the normalized sensitivity coefficients are negative with the zone of highest sensitivity occurring along the flow line downgradient of the chosen performance measure. Thus the results confirm that hydraulic conductivities along the streamline passing through the site are most important. As the perpendicular distance from this streamline increases, the importance of the hydraulic conductivity generally decreases.

The normalized sensitivity coefficients for prescribed head indicate that the values of phreatic head within the landfill itself are relatively insensitive to the prescribed boundary conditions. The maximum value occurs at node 11 and indicates that a one percent change in the prescribed boundary (574 feet ASML) will produce a .028 percent change in the head at the landfill. Overall, the sensitivity results indicate that the head at the Hyde Park Landfill is controlled by the local recharge and the hydraulic conductivity of the area.

Figure 46 illustrates the normalized sensitivity coefficients of Darcy velocity in element 161 with respect to areal recharge. It presents the same pattern as Figure 44 with the largest magnitude immediately surrounding element 161 and decreasing away from the element. West of element 161 the values are all negative indicating that additional recharge would decrease the gradient and thus the velocity through the element. East of the element, the values are positive indicating that additional recharge would increase the gradient and thus the velocities.

Figure 47 illustrates the normalized sensitivity coefficients of Darcy velocity with respect to hydraulic conductivity and prescribed head conditions. Along the east-west flow line, the results demonstrate the same pattern as Figure 45.

The normalized sensitivity coefficients for the prescribed head indicate maximum negative and positive values at the nodes on the west and east boundaries respectively where they are intersected by the flow line through element 161. For the northern and southern boundaries, in a west to east direction, the values are initially negative, increasing in magnitude and then decreasing. The values become positive near the mid-point of the boundaries with the values initially increasing and then decreasing in magnitude.

Results of the pump tests indicate that zones of higher hydraulic conductivity exist at the Hyde Park Landfill Site. The areal extent of the higher hyraulic conductivity zones was assumed to coincide with the lineaments described by Woodward-Clyde (1983). The hydraulic conductivity estimate assigned to the corresponding elements was obtained from the recovery data of wells F-4A and H-3 after cessation of pump test F-4A. It was hypothesized that F-4A and H-3 were hydraulically connected due to the rapid and mirror image drawdown responses at H-3 with the drawdown responses at F-4A. The value obtained was 1985 gpd/ft. which, for a saturated depth of 100 feet, is equivalent to 2.65 feet per day.

The elements with higher hydraulic conductivities are illustrated in Figure 48. Figures 49 and 50 illustrate the phreatic surface and Darcy velocities with the inclusion of the higher hydraulic

HIGHER HYDRAULIC CONDUCTIVITY ZONES

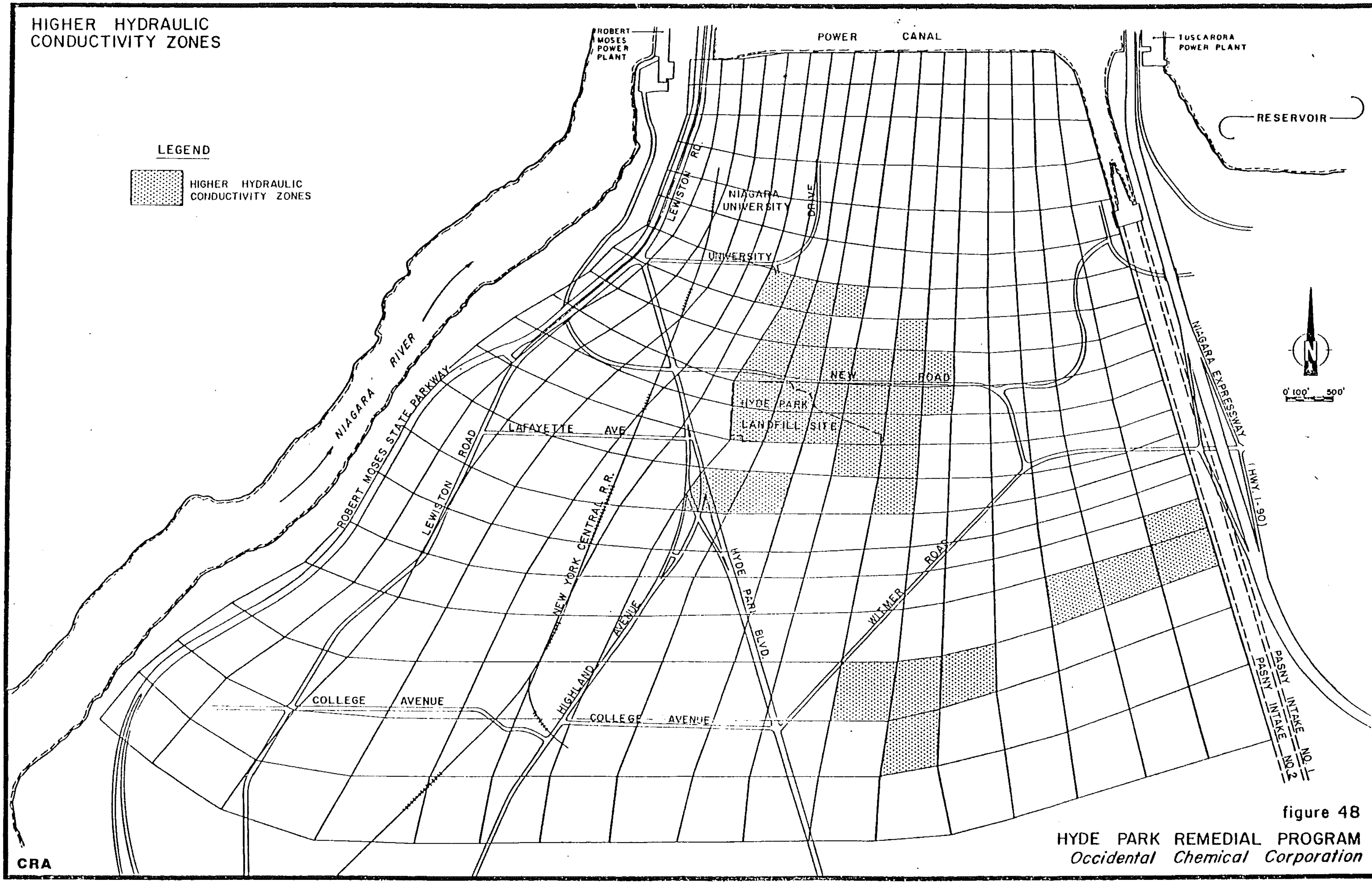

figure 48
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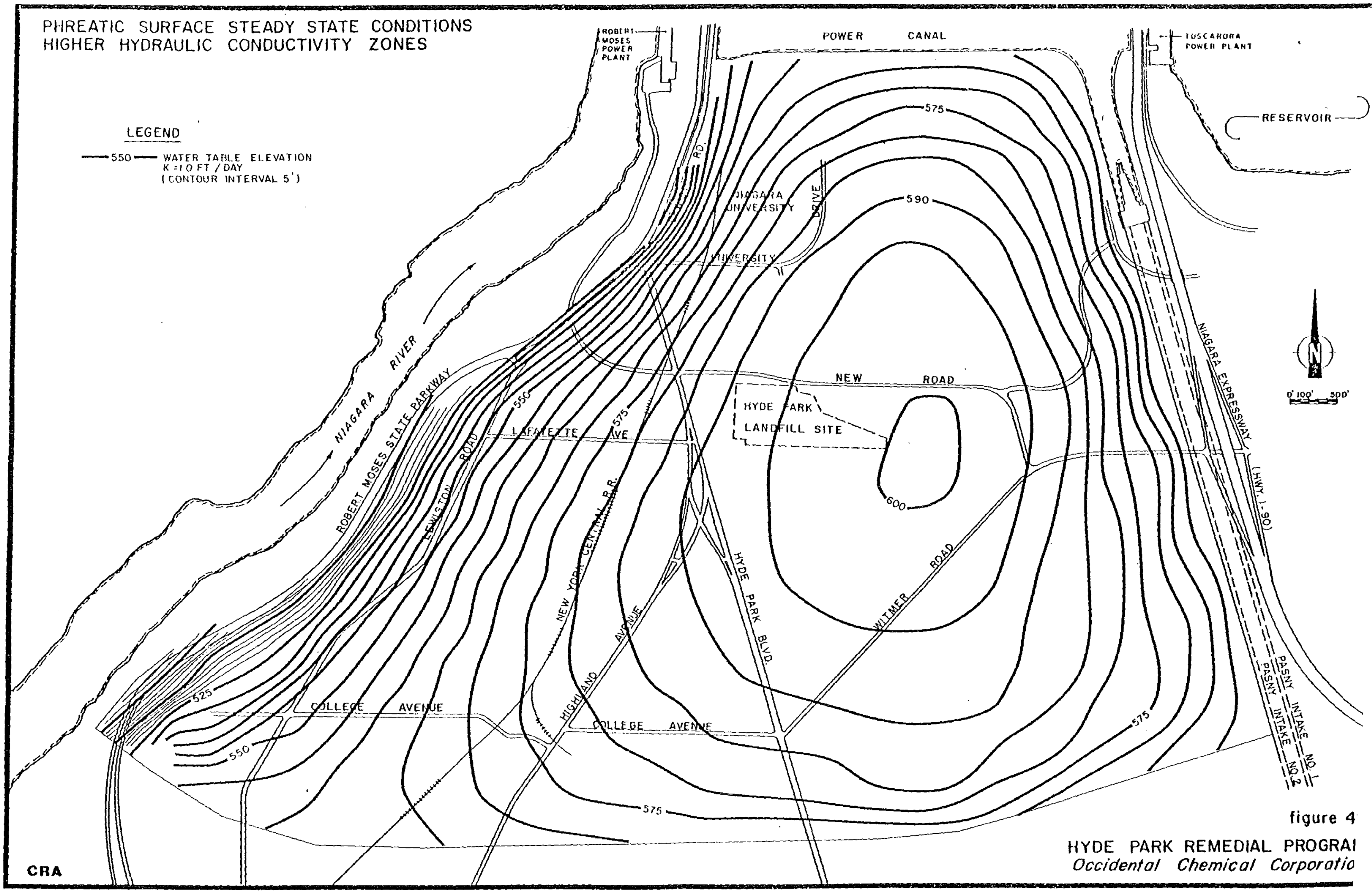

figure 4

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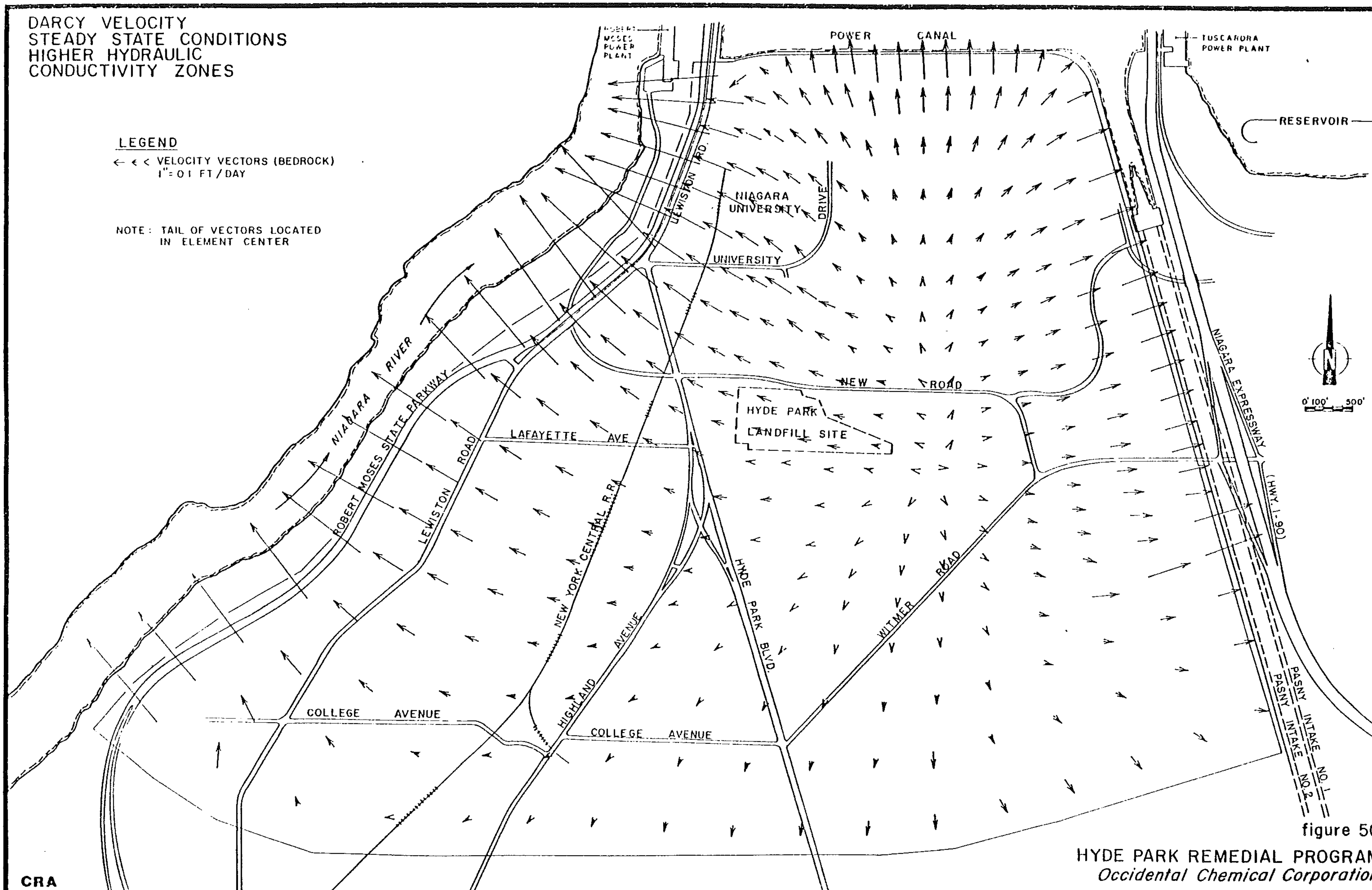

figure 50

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conductivity zones, respectively. Comparison between Figures 42 and 49 and between 43 and 50 indicate that the maximum phreatic surface elevation has changed from 605.14 feet AMSL to 600.58 feet AMSL and has shifted location from node 174 to node 156 which is 370 feet to the south. The maximum change in Darcy velocity magnitude occurred in elements 68 and 85 where the Darcy velocities doubled. In the other elements with higher hydraulic conductivities, the Darcy velocity magnitudes increased by less than a factor of two. The velocities did not increase in magnitude by the ratio of high conductivity estimate to low conductivity estimate ie. 2.65:1, as the gradient in the higher hydraulic conductivity zones decreased. It should be noted that even through the Darcy velocities approximately doubled their magnitudes in the immediate area of the Hyde Park Landfill are still very small. This is due to the small gradient through the site.

In summary, the general flow pattern for the entire modelled area remained basically the same for both scenarios.

7.6.2 Two Dimensional Single Phase Cross-Sectional Model

Calibration of the two dimensional single phase cross-sectional model is being performed by adjusting the hydraulic conductivity values for the various lithologic units. The horizontal to vertical hydraulic conductivity ratios from the calibrated results of Maslia and Johnson (1982) are employed initially and are listed in Table 9. It is to be noted that the lower Lockport, Gasport and Decew units were assigned the same hydrologic properties. Consistent with the work of Maslia and Johnston (1982), the first 200 feet from the gorge face were assigned K_{xx}/K_{zz} ratios of 1:10. Recharge along boundary BC was kept consistent with the calibrated recharge values for the areal model. Results to date indicate horizontal hydraulic conductivities of 0.0283, 5.0 and 1.7 feet per day in the till, Upper Lockport and remaining units respectively. At the narrowest section, this produced a weighted average value of 3.07 feet per day. This value is higher than the 1.0 foot per day employed for the calibrated areal model. The reason for this is that the cross-sectional model does not allow flow perpendicular to the cross-section. Increases in hydraulic conductivity are necessary to compensate for this effect. Adjustment of the K_{xx}/K_{zz} ratios has indicated that lower ratios will decrease the phreatic surface elevation. To reproduce existing conditions with the lower ratios the hydraulic conductivity values are decreased. Calibration is continuing by adjusting the K_{xx}/K_{zz} ratios and hydraulic conductivity values.

TABLE 9

HYDROLOGIC PARAMETERS FOR
SINGLE-PHASE FLOW SIMULATION

Geologic Unit	Horizontal Hydraulic Conductivity (ft/day)	Ratio of Anisotopy K_{xx}/K_{zz}
Glacial Till	0.0203	2.5:1
Upper Lockport	5.0 1.7x10^{-3}	100:1
Lower Lockport	1.7	1000:1
Gasport	1.7	1000:1
Decew	1.7	1000:1

In the calibration analysis it was observed that the upper till layer controlled the results to a very large extent by controlling the rate of infiltration. When the sand suction head-degree of saturation function was employed, the prescribed recharge mounded on top of the till layer creating large hydraulic leads. This was the result the rapid decline in water saturation with respect to suction head and the corresponding small relative permeability at the low degree of saturations. This low relative permeability combined with the low value of hydraulic conductivity for the till effectively stopped the water from percolating into the system. When the clay curve was employed, higher relative permeabilities were obtained and the water percolated into the system. Use of the clay curve is justified in that for a majority of the site, the water table is at or near the interface of the till layer and the Lockport Formation.

The modelling of the proposed RRT remedial plan for the Hyde Park Site will be discussed in Section 10.

7.6.3 Two Dimensional Two-Phase Cross-Sectional Model

The migration of the NAPL in the saturated zone was modelled using an equivalent porous medium approach. This method assumes homogeneous (anisotropic) soil characteristics in each defined lithologic zone.

In section BB-BB', two such zones were considered. The upper zone is bounded on top by the Lockport Formation/overburden interface and is approximately thirty feet thick, representing the highly weathered portion of the dolomite. An effective horizontal intrinsic permeability of $1.9 \times 10^{-11} ft^2$, a horizontal to vertical permeability ratio of 100 and an effective porosity of 3% were used to model this zone.

The estimate of effective porosity was based on the premise that the maximum effective porosities obtained from the pump test results are representative of the upper unit. For the lower unit, a smaller effective porosity estimate was employed as compression with increasing depth produces a denser, less porous media.

The lower zone extends from thirty feet into the Lockport Formation to the upper face of the Rochester Shale which is assumed to be essentially impermeable. Here an effective horizontal intrinsic permeability of $6.6 \times 10^{-12} ft^2$, a horizontal to vertical permeability ratio of 1000 and an effective porosity of 0.1 percent were used in the two-phase model. A relative density of 1.216 and an absolute viscosity of 500 centipoise were used in the model. The viscosity value represents the most mobile (worst case) NAPL suggested by laboratory analysis of fluid samples.

The two-phase cross-sectional model considers varying degrees of saturation of both the APL and NAPL. The function relating capillary pressure, Pc, to degree of aqueous phase saturation, Sw, used in the two-phase modelling are illustrated as Figure 5A in the report "S-Area Two-Phase Flow Model" (Arthur D. Little Inc., June, 1983). The functions relating relative permeabilities to the APL and NAPL with the degree of saturation of the wetting phase are shown in Figure 3 of the same report.

The boundary conditions for the two-phase model are, for the APL, zero flux on boundary AD of Map 4, which corresponds to the top of the Rochester Formation, and prescribed pressures, Pw, on all other boundaries. The prescribed pressures on boundary BC were derived from the results of the two-dimensional flow modelling in the horizontal plane, while boundaries AB and CD were assumed to be under hydrostatic aqueous pressure distributions.

For the NAPL, the entire domain boundary was prescribed as zero flux except for nodes 11, 22, 33 and 44 (Map 4) which are located directly below the Hyde Park landfill site. By assuming a constant degree of NAPL saturation of 80 percent at those nodes, a corresponding capillary pressure was determined. Since the APL pressure is also prescribed, prescribed NAPL pressures were calculated using Equation (19).

The spatial discretization of cross-section BB-BB' consists of 200 elements and 231 nodes.

With the preceeding fluid and continuum properties, and boundary conditions, the model was run to simulate a 31 year period representing the time from the inception of the Hyde Park Landfill Site to the present.

The results of the simulation indicate that under the assumed conditions, the furthest extent of the NAPL plume (assumed to be one percent or greater saturation of NAPL) in the horizontal plane would be approximately 660 feet south of the southern boundary of the landfill site. In the vertical plane the furthest extent of the NAPL plume would be approximately 55 feet below the top of the Lockport formation. Figure 51, a contour map showing degree of NAPL saturation, illustrates the plume.

The aquifer survey has indicated that observable NAPL concentrations exist as far as 1600 ft. south of the site and as deep as the upper face of the Rochester Formation (along cross-section BB-BB'). This suggests that isolated lineaments of high permeability exist in both the horizontal and vertical directions or that reduced viscosities are present. To illustrate the effects of higher hydraulic conductivity zones, the same hydrogeological scenario was simulated with the addition of a vertically fractured zone oriented perpendicular to section BB-BB', located directly below the Hyde Park Landfill site and intersecting the complete thickness of the Lockport Formation.

To simulate the fractured zone the vertical intrinsic permeability of elements 11 to 20 was increased to 6.6×10^{-10} ft^2. It was hypothesized that the vertical intrinsic permeability would be greater than the horizontal intrinsic permeability. As an estimate, a value of two orders of magnitude greater than the horizontal estimate was utilized. The results of this simulation, illustrated

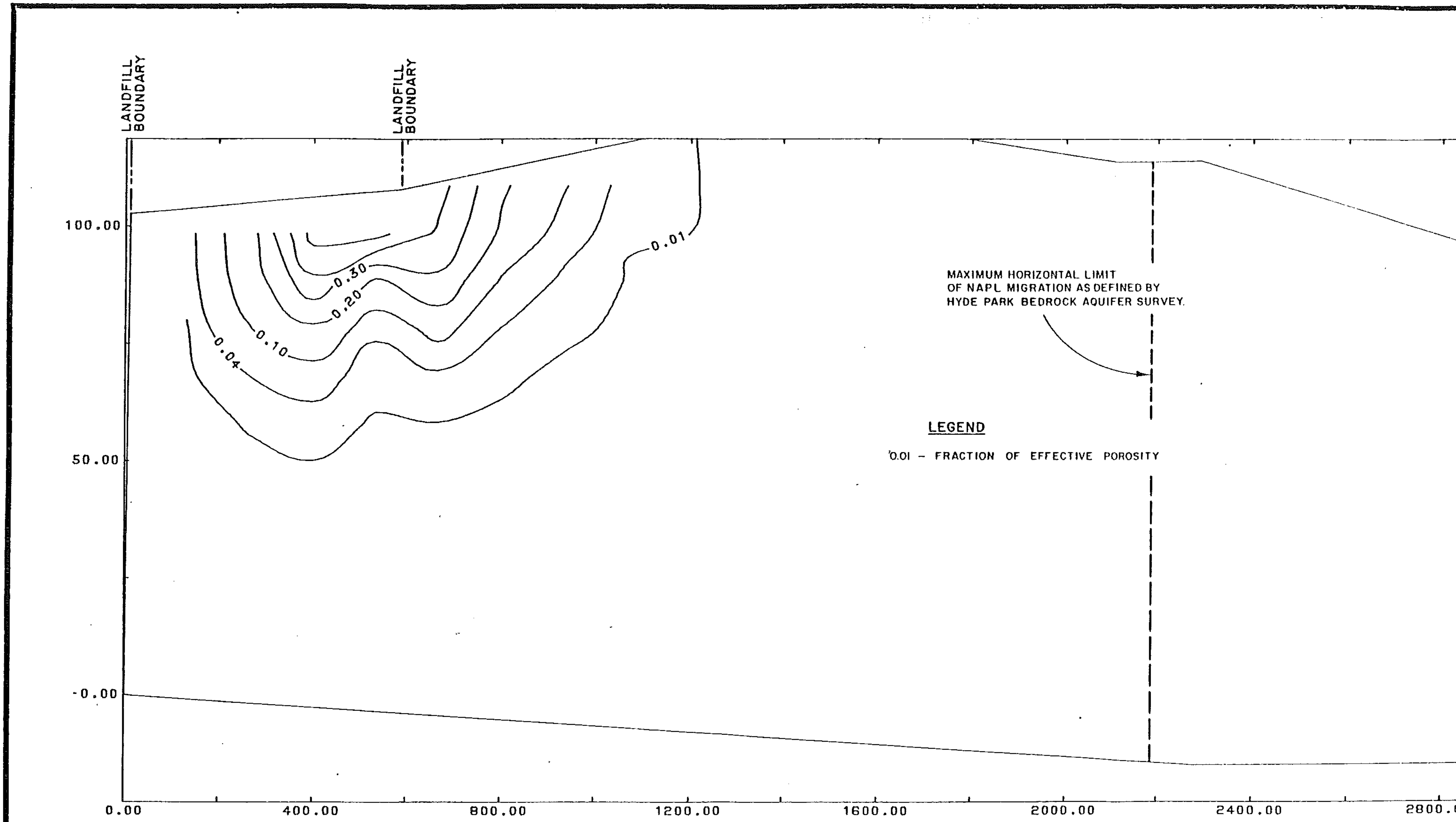

figure 51

SECTION BB - BB'

DEGREE OF NAPL SATURATION
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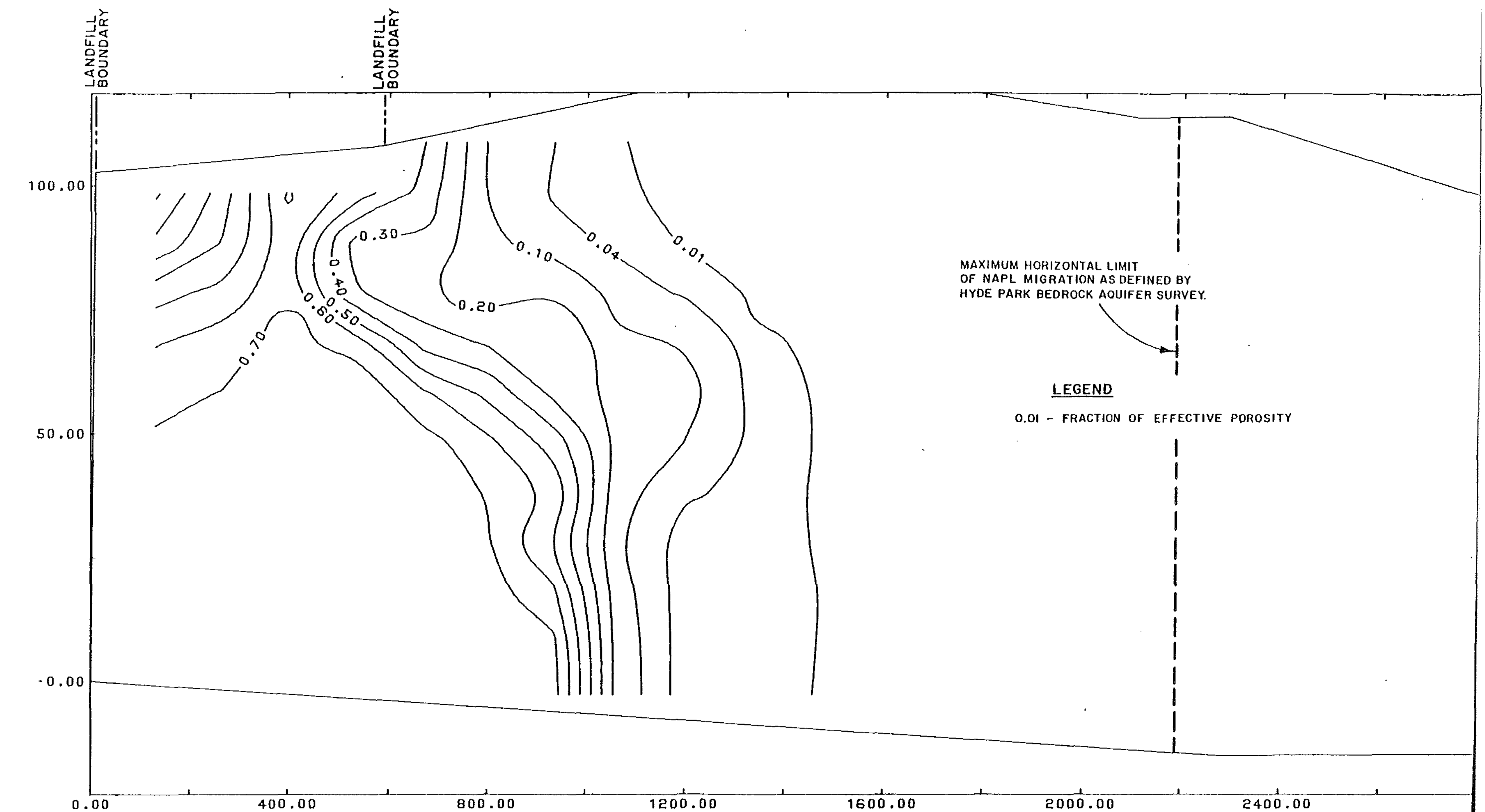
in Figure 52, indicate that the NAPL would move down the fractured zone to the top of the Rochester Formation where mounding would occur.

As the mound of NAPL in the fractured zone rises, increased pressure at the base causes increased lateral migration. Similarly, higher permeability zones in the horizontal plane, which may exist, for example, along bedding planes, would accelerate lateral migration of the NAPL. The rate and direction of migration therefore depends directly on the location and orientation of anomalous high permeability zones.

Figure 52 shows that after 31 years (the time since initial use of the site) the NAPL plume having a NAPL saturation of 1% would extend approximately 900 feet south of the landfill site boundary. Figure 52 also shows the defined outer limit extent of NAPL by visual and olfactory evidence in the bedrock. Figure 52 shows that the modelled extent of NAPL migration at 1% compares favorably with the field defined extent of NAPL migration based on visual and olfactory evidence which would be less than a 1% concentration.



SECTION BB - BB'

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figure 52

DEGREE OF NAPL SATURATION
VERTICAL FRACTURE
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7.7 SUMMARY

Analysis of the simulation results to date, indicate the following:

1) The calibrated parameters ~~represent the~~ best average estimate for the area of the Hyde Park Landfill. Further definition of these parameters through additional data collection in the field is not expected to significantly affect the average values.

2) The sensitivity analyses indicate that, for the chosen performance measures, the system is controlled by the element hydraulic conductivities and recharge and that the performance measures are relatively insensitive to the prescribed phreatic head boundaries. The normalized sensitivity coefficients indicate that the performance measures are most sensitive to the element values of recharge and hydraulic conductivity along the stream line passing through element 161 (Map 2) and in the elements immediately surrounding the location of the performance measures. This indicates the regions and the parameters where accurate characterization would reduce the uncertainty in the performance measure.

8.0 BEDROCK - ALTERNATIVE TECHNOLOGY EVALUATION

Under the Settlement Agreement an investigation is required to evaluate alternative remedial technologies to address identified chemical plumes in the bedrock. It is to be noted that remedial technologies deemed to be suitable by inclusion in the Settlement Agreement are not evaluated further. Tables 2 and 3 identify technologies considered and found to be unsuitable for reasons of such significance that a detailed evaluation herein was not warranted. The following containment alternatives were considered to warrant a detailed evaluation:

A) No action in Bedrock below 15 feet.

B) A grout curtain wall

C) Hydrofracturing

D) Grouting through controlled fractures

E) A purge well system

F) Injection and purging system

G) Non-aqueous phase removal

H) In place fixation/stabilization of waste

I) Waste relocation

J) Slurry/barrier wall

An evaluation of each of the above alternatives is briefly summarized in Table 10. The following section of this report presents discussions of each of the alternatives evaluated.

TABLE 10

ALTERNATIVE EVALUATION SUMMARY - BEDROCK CONTAINMENT PLUME

Alternative	Comments
Grout Curtain Wall	- Reduces groundwater flow through chemical source - Reduces groundwater flow toward purge well system - Could contain APL and NAPL migration - Costly - Technology could be used to grout specific fractures or areas - Requires further testing for Hyde Park conditions - Effectiveness could be enhanced by pumping
Hydrofracturing	- Can be used in conjuction with pressure grouting or purge well systems to increase effectiveness - Propagation of a vertical fracture extending through the Rochester Formation could be detrimental
Grouting through Controlled Fractures	- Typically involves hydrofracturing in conjunction with grouting - Propagation of a vertical fracture extending through the Rochester Formation could be detrimental

TABLE 10 (Cont'd)

ALTERNATIVE EVALUATION SUMMARY - BEDROCK CONTAINMENT PLUME

Alternative	Comments
Purge Well System	- Effective in controlling APL migration - Effectiveness in containing NAPL to be determined - Wastewater requires treatment - Effectiveness can be enhanced by - hydrofracturing - pressure grouting - recirculation - injection
Injection and Purging System	- Could accelerate aquifer rehabilitation - unknown NAPL rheological characteristics - high cost for heat injection - chemical injection technology not available - unknown reactions with injection chemicals
Non-Aqueous Phase Removal - Landfill	- Ineffective due to unsuitable hydraulic characteristics - Redundant if containment in bedrock is attainable
- Bedrock	- May be a secondary benefit achieved when controlling APL migration

TABLE 10 (Cont'd)

ALTERNATIVE EVALUATION SUMMARY - BEDROCK CONTAINMENT PLUME

Alternative	Comments
In Place Fixation/Stabilization of Waste	- Technology not available
Waste Relocation	- High cost - Health and environmental risk - Secure landfill availability
Slurry Wall - Overburden	- Redundant
- Bedrock	- Not appropriate technology
No Action in Bedrock Below 15 feet	- Not Environmentally Acceptable

8.1 NO ACTION IN BEDROCK BELOW 15 FEET

As described in subparagraph D(3) of Addendum I of the Stipulation and Judgment Approving Settlement Agreement, a Bedrock Barrier Collection System (BBCS) is to be designed for the Hyde Park Landfill Remedial Program: "The Bedrock Barrier Collection System shall be designed to be capable of collecting aqueous phase liquids in the Overburden below the Overburden Barrier Collection System and in the Lockport Bedrock Zone which have migrated from the Landfill Site to the same extent as a tile drain collection system installed to a depth equal to the greater of:

a) seven feet below the top of the Lockport Bedrock Zone; or

b) the maximum depth of water bearing zone confirmed by the above-described testing; provided, however, that nothing in this paragraph shall require the installation of a Bedrock Barrier Collection System designed to collect liquids more than 15 feet below the top of the Lockport Bedrock Zone."

8.1.1 Effectiveness of No Action in Bedrock Below 15 Feet

The Bedrock Aquifer Survey conducted for the Hyde Park Landfill Remedial Program identified that the plume of NAPL is not limited to the upper 15 foot interval of the Lockport Bedrock Zone. The design and installation of such a collection system will therefore not be effective in containing the entire plume. Subsequent remedial works to address chemicals below the 15 foot depth would render the 15 foot BBCS redundant. As a result, the proposed BBCS will be omitted in favor of a system designed to address the entire bedrock chemical plume as per the Settlement Agreement subparagraph D(7) Addendum 1. Therefore no action in the bedrock below 15 feet is not a viable remedial alternative.

8.2 GROUT CURTAIN WALLS

Grout curtain walls have been successfully used to control the seepage of groundwater through bedrock regimes. The wall is formed by pumping grout under pressure into a series of predrilled boreholes in the bedrock. As the grout is pumped into the boreholes it is forced out into the voids and fractures of the surrounding bedrock formation. Once set, the physical presence of the grout in the voids and fractures precludes the continued use of these passages as routes of groundwater and chemical flow.

The alignment of the boreholes and the water bearing characteristics of the bedrock determine the effectiveness of the grout curtain wall as a containment system.

There are several types of grouting materials and additives which can be used to enhance a particular characteristic of the grout, thus increasing the effectiveness of the curtain wall for particular situations (See Appendix B). There are also several chemical grouts available which possess physical capabilities not possible through cement based grouts. (See Appendix C)

8.2.1 Applications of Grout Curtain Walls

Properly installed, the grout curtain wall forms a barrier of reduced permeability which restricts the flow of groundwater and chemicals across the wall alignment. The traditional use of grout curtain walls is to control groundwater movement beneath water retention structures such as dams where uncontrolled groundwater movement promoted by the head differential between the stored water and downgradient conditions could lead to structural failure of the containment facility.

There are a few applications in which grout curtain walls could be used as a part of the containment scheme for the Hyde Park site. These include:

1) As a barrier wall completely encircling the landfill or one or more plumes thus acting as a containment system restricting chemical migration.

2) As a downgradient barrier to block the further advance of chemical migration along selected paths.

3) As an upgradient barrier to reduce the volume of incoming groundwater to the containment area.

4) As a barrier to seal specific fracture zones in the area to contain/eliminate groundwater flow.

8.2.2 Methods of Grout Curtain Construction

The grout curtain is constructed by injecting grout into boreholes in the bedrock (usually drilled vertically). The three most common drilling methods are percussion, rotary roller and rotary diamond, in ascending order of cost. Following the drilling operation, the borehole is flushed to remove cuttings and fine particles in the fracture apertures crossed during drilling. This is particularly important for percussion drilling which is not as effective as the rotary methods in maintaining a relatively clean borehole.

The boreholes are generally drilled in a pattern of 2 to 3 staggered rows. The purpose of staggering the rows is to increase the effectiveness of the grout placed. The number of rows and the spacing of the primary row is dependent on such factors as the gradient of head that the curtain wall is intended to restrain and "experience" related to predetermined and/or expected subsurface conditions.

Subsequent spacings and grouting pressures are refined according to tests completed following the start of the grouting procedure.

One common drilling pattern consists of three rows of boreholes. Every other borehole in each of the outside rows is grouted on the first pass, alternating from row to row. Each of the remaining outside boreholes are then grouted according to the same pattern. The center row boreholes are located in the middle of a group of four boreholes (two from each row) to effectively fill any gaps. Again, the grouting every other borehole on the first pass and the remainder on the second pass.

At the Hyde Park Site, a double row curtain wall could provide sufficient groundwater impedance since the expected groundwater head that the wall would be required to restrain would be minimal.

The injection of the grout is commonly placed by the penetration method. At first the grout travels rapidly as it moves radially from the grout hole. The take is reduced gradually as eventually the applied pressure cannot move the outer fringes. The final step is refusal of the grout take. The pressure should be held for approximately 15 minutes (dependent on grout type used) to ensure total refusal (Houlsby, 1982). Multiple applications may be required if large voids (> 1/4") or long distances (> 50 feet) of grout travel occur.

One of the major drawbacks of refusal grouting is that there is no control over the volume of material injected. This typically results in an excessive waste of grout material. Based on the fact that excessive grout takes are generally the result of extended grout travel into areas beyond the limits of the curtain wall's designed effective width, the need to grout to refusal is not warranted. To prevent such a waste of grout materials, it is recommended that the bedrock formation be evaluated for grout take. This can be accomplished through estimation of a maximum effective porosity for the bedrock formation or for a particular grouting stage. The maximum effective porosity can be increased by a factor to account for nonuniform grout distribution to determine the maximum theoretical grout take for the designed curtain wall width for each grouting stage. Once this volume of grout has been injected, it can be assumed that the effective void space within the designed curtain wall width has been filled. Injection of additional grout simply penetrates further into the bedrock formation and is of no significant value to the effectiveness of the wall.

In cases of grouting over extended depths, the injections are completed in stages which may proceed either upwards or downwards. Generally a packer is used to facilitate grout placement and is a requirement when moving upwards.

The placement of grout in stages permits grouting to proceed within a controlled pressure range which forces the grout out into the fractures without applying pressures capable of causing additional fracturing of the bedrock regime.

As an alternative to injecting grout through a grout plot pattern (conventional) to achieve a barrier wall, it is also possible to construct a continuous grout wall or barrier wall in the bedrock. This can be accomplished through the use of a megadrill to drill large diameter (i.e. 30") boreholes into the bedrock in such a pattern that the edges of each borehole touches the next one. This will result in a continuous void alignment which can be backfilled with grout or a designed low permeability backfill material. The major drawback to the construction of a continuous wall is the excessive quantity of waste rock generated which results in the need for significantly greater backfill material volumes. As a result, an injected grout curtain wall grout curtain wall can achieve similar results with considerably less waste handling and backfill material.

A discussion of the cost comparison of a conventional grout curtain wall and continuous grout curtain wall are presented in Appendix D.

8.2.3 General Specifications for Grout Curtains

The grout take for a particular stage interval will vary with the bedrock characteristics and lithology. Typically primary borehole grout take is considerably greater than secondary borehole grout take as expected. Chemical grouts generally have higher takes than cement grouts due to their physical properties.

Pumping pressures generally vary between one half and one p.s.i. per foot depth of borehole. Too high a pressure may cause rock fracturing, leading to larger fractures and higher grout take. The limiting factor with respect to grouting pressure is therefore the pressure at which hydraulic fracturing begins for that particular rock zone.

The type and mixture of the grout is most important as it will ultimately determine the permeability of the resulting curtain wall. Cement grout usually contains cement, sand, water, bentonite and sometimes fillers. The cement is usually Type I or II Portland cement while the bentonite is a western or "premium grade" bentonite (Geocon, 1980). The water should be fresh and clean. The sand should be well graded with a range of 95-100% passing through the #16 sieve and 10-30% passing through the #100 sieve (Bowen, 1981). Sand is not being considered for use in the Hyde Park site due to the relatively fine fracture apertures.

There are many fillers which may be considered. Flyash is often used and can replace up to 50% of the cement. This reduces costs while increasing water tightness. Pulverized Fuel Ash (PFA) is both inexpensive and durable with a specific gravity of 1.9 to 2.4. Other fillers include expanding agents to counteract shrinkage and accelerators to speed up set or gel time (Littlejohn, 1982).

Special grouts which have been formulated include Embeco non-shrink grout (Bowen, 1981) and super sulphated cement grout (Littlejohn, 1982).

The chemical grouts are typically applied to more specific grouting situations. The most important factor in chemical grout placement is the gel time which determines the extent of penetration for the grout. Generally the gel time should be kept to between 50 and 100 percent of the complete pumping time of one stage.

8.2.4 Grout Curtain Walls: Equipment

A drilling unit is commonly used to drill the grout holes. The type of drilling was previously discussed in the section on methods.

The equipment used for grout injection includes a mixer, agitator, pump, circulation line and control fittings.

A high speed mixer is used to separate the cement grains. This is done to achieve thorough wetting of the grains and therefore, hydration (Houlsby, 1982).

The agitator allows for constant stirring of the grout in order to maintain the fluid state needed for pumping.

The pump can be a constant speed pump since a recirculating line is usually used. The recirculating line transfers grout from the pump to the hole and back to the agitator.

The control fittings control the rate of injection, the pressure and the bleeding of water and thin grout.

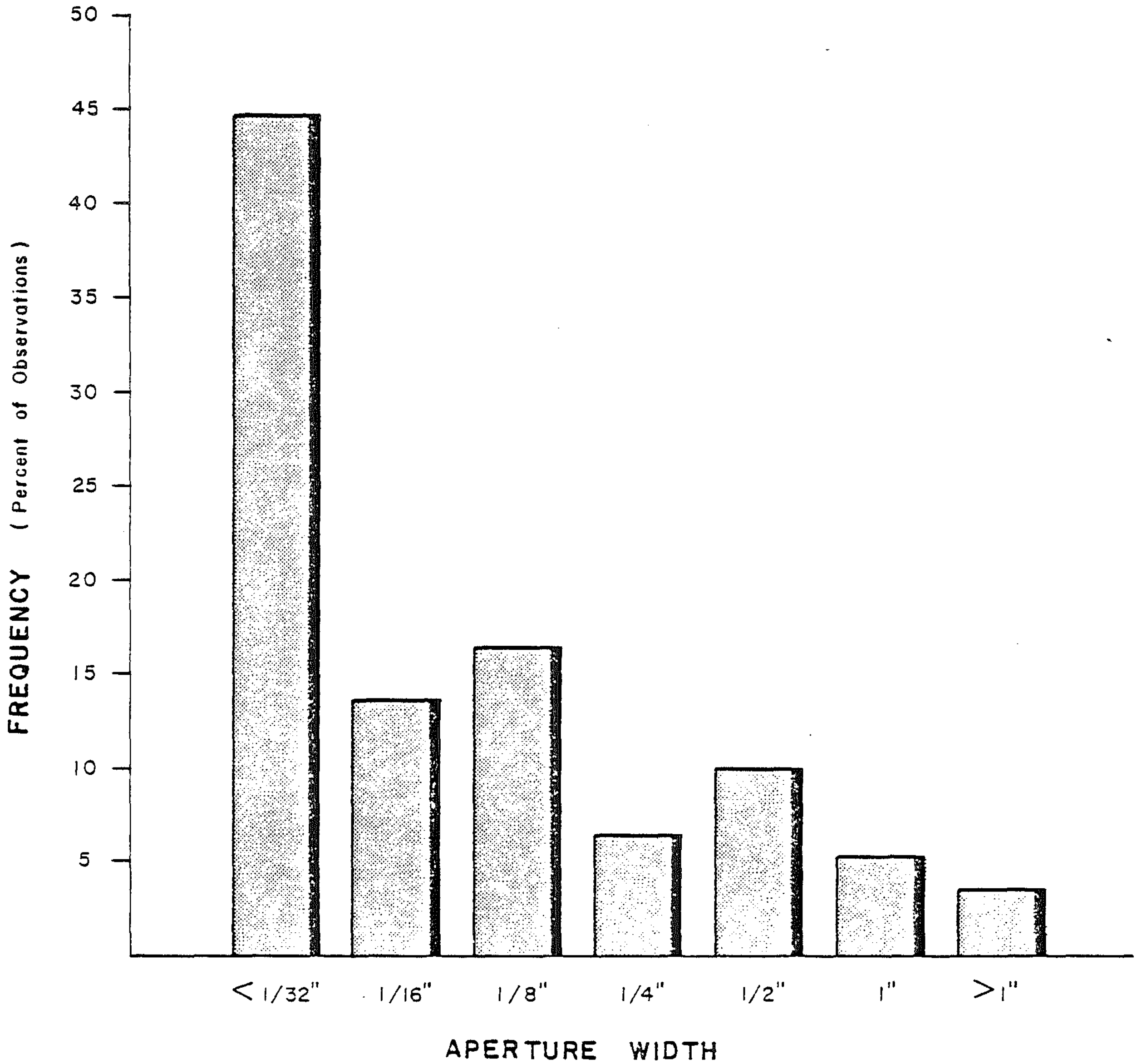
A packer is used for stage grouting. The packer is used to isolate sections of the borehole for grouting in progressive stages.

Chemical grouting requires different instrumentation because of the need to control the mixture of chemicals. Batch feed systems often cannot be used with chemical grouts due to the short gel times. Two tanks must be used to keep the components separate. Each tank must be fitted with a separate pump. For chemical grouting, the mixing typically is performed in place. Chemical grouts are now available which react with water to activate the gelling process. This greatly reduces the logistical problems associated with mixing and quality control.

8.2.5 Effectiveness of Grout Curtain Walls

Grout curtain walls are used extensively in the construction of water reservoirs where curtain permeabilities on the order of 1 x 10-5 cm/sec are generally accepted. The grout used for such an application is generally cement-based and capable of filling 90 to 95 per cent of the voids under certain geologic conditions. Due to the size of the cement particles, there is a physical limitation to the size of the fractures that can effectively be sealed with cement-based grout. The greater the proportion of large fracture apertures in the bedrock, the more effective a cement-based grout will be. Conversely, if the fracture apertures are relatively small, the effectiveness of the cement-based grout will be considerably reduced.

During the Hyde Park Aquifer Survey, a mapping of the Lockport Formation's physical characteristics was completed from the exposed gorge face along the access road to the Robert Moses Generating Station. Based on this investigation, a breakdown of the measured fracture apertures has been generated. This breakdown is presented in Figure 53. As can be seen from Figure 53, approximately 45 per cent of the measured fracture apertures were less than



NOTE:

BASED ON ENTIRE LOCKPORT FORMATION AS MEASURED ALONG LOWER ACCESS ROAD TO ROBERT MOSES GENERATING STATION.

figure 53

FREQUENCY : VS : WIDTH OF FRACTURE APERTURES
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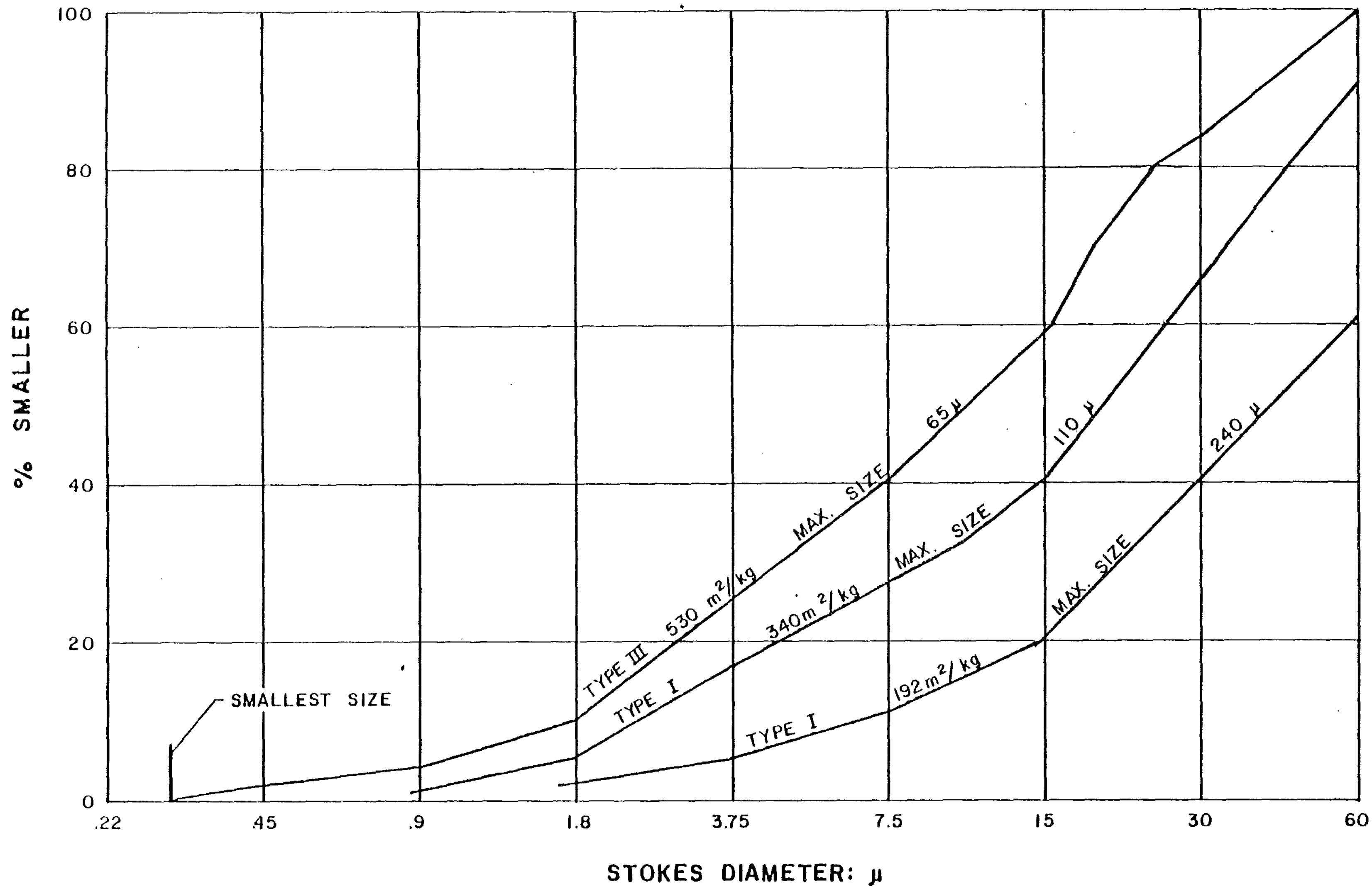
1/32 inch (approximately 800 microns). According to the EPA Handbook on Remedial Action at Waste Disposal Sites "the smallest voids that can be effectively grouted are no smaller than three times the cement grain size." Typical average cement grain sizes (50% passing) range from 22 micron for Type 1 Portland Cement, 15 micron for Type 2 and 10 micron for Type 3. Type 3 Portland cement is finer grained and therefore can be more effective in reducing the permeability of the curtain walls. A microfine cement recently developed in Japan has an average grain size of 4 microns. Figure 54 presents typical grain size curves for cement. The introduction of other fine grained filler materials such as bentonite, flyash, pozzolans and pulverized fuel ash serve to further increase the effectiveness of the curtain wall. A list of additives for pressure grouting are presented in Appendix B.

In order to attain lower permeabilities, chemical grouts are sometimes used. The fact that chemical grouts are liquid allows them to penetrate finer fracture apertures thus making the curtain wall more effective as a groundwater barrier. It is to be noted that chemical grouts are typically more expensive to use than cement grouts and since they are liquid and pass through the fractures more



SOURCE: "GROUTING IN GEOTECHNICAL ENGINEERING"

figure 54

GRAIN SIZE ANALYSIS CURVES FOR CEMENT
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ecology and environment

easily, higher grout takes are generally experienced with the chemical grouts, although this can be controlled to some degree through control of the set time permitted. A list of chemical grouts is presented in Appendix C.

In some cases, cement and chemical grouts are used in combination. The cement grout being used to fill the larger fracture apertures and the chemical grout subsequently used to fill the minor fracture apertures untouched by the cement grout. This method of primary-secondary-tertiary grouting with changing grout materials significantly reduces the volume of chemical grout consumed and thereby reduces cost.

The construction of a grout curtain wall is site specific, being dependant on the rock formation and localized conditions, especially the fracture characteristics. The use of a grout curtain for containment of chemicals at the Hyde Park Landfill is a viable alternative but will require an intensive program of on-site testing. From the testing, the optimum design including grout selection, borehole spacing, drilling pattern and grouting methodology will be determined from which an effective curtain wall can be designed.

Use of pressure grouting technology to seal specific fracture zones in the Lockport Formation is a possible alternative to the construction of an entire grout curtain wall. This technology could be used as a downgradient or upgradient groundwater control mechanism in the APL plume. The use of pressure grouting within the NAPL plume would require testing to determine whether the grout can effectively displace the NAPL from the fracture apertures. Even if the NAPL can be pushed from the center of the fracture apertures, the coating of NAPL on the fracture surfaces may not permit the grout to effectively seal against the bedrock. As a result, the fractures may not be completely sealed.

The use of pressure grouting in the Lockport Formation has been employed by the hydroelectric generators on both sides of the Canada - USA border local to the Hyde Park Site. The uses of pressure grouting at both the Sir Adam Beck and Robert Moses Generating Stations have included reservoir construction, dewatering projects and protection from hydrostatic groundwater buildup.

8.3 HYDROFRACTURING

Hydrofracturing, also referred to as hydraulic fracturing is conventionally used by oil well drillers and water well drillers to stimulate production of recovery wells. Hydrofracturing is the initiation and propagation of fractures in the bedrock through the use of applied pressure. This fracturing of the bedrock increases the local permeability of the formation through increased interconnection of fractures. Hydrofracturing is one of various methods of well stimulation including acid treatment, surging, jetting, and blasting.

8.3.1 Applications of Hydrofracturing

Hydrofracturing can be used to increase the communication between various layers within the bedrock. Fracturing of the bedrock through hydraulic pressure may also form cracks which could connect a well located in a formation of low primary permeability to a nearby water bearing zone.

Possible applications of hydrofracturing and other fracturing techniques at Hyde Park are:

1) to increase the permeability of the bedrock to improve purge well production

2) controlled fracturing of the dolomite for the purpose of enhancing the effectiveness of a grout curtain wall.

8.3.2 Methods of Hydrofracturing

Hydrofracturing of the rock is initiated by injecting liquid into a well under high pressures (Campbell & Lehr, 1973). Obviously, the fracturing pressures are different for each rock formation.

Gelling agents are occasionally added to the injected liquid to reduce the rate of flow of the liquid into the rock formation fractures thus permitting higher pressures to be attained. To overcome the possible closure of fractures upon pressure release, propping agents, usually sand, are added to the injected liquid.

Investigative research of hydrofracturing suggests that fractures are vertical and extend radially from the well (Bouwer, 1978). This provides for increased vertical connection of various zones within the local stratum. Hydrofracturing with pressure results in propagation of fractures along the lines of least resistance (often along bedding planes) which then exert stresses to the surrounding bedrock causing fracturing normal to the initial fracture (vertical).

Other methods of rock fracturing which have been used as alternatives to hydrofracturing are blasting, or shooting and nuclear fracturing. Blasting involves the filling of a borehole with explosives overlain by sand. The sand above the explosives tends to increase the lateral force of the explosion, while sand interspersed with the explosives is forced into the cracks to assist in keeping these new fractures open. This method has been successfully used by well drillers to increase the connection between wells and vertical water bearing faults (Driscoll, 1978).

Nuclear fracturing involves the use of grouting through controlled fractures. This methodology is described in the following section concerning grouting through controlled fractures.

8.3.3 Hydrofracturing Specifications

Previously it was assumed that the required injection pressures had to be greater than the pressure asserted by the overburden on the bedrock formation (Bouwer, 1978). More recently, tests have suggested that pressures as low as 60 percent of the overburden pressure would initiate fracture propagation.

8.3.4 Hydrofracturing Equipment

Hydrofracturing is generally undertaken using truck mounted high pressure, high injection rate pumping equipment complete with metering and control devices. This equipment could range from standard rock grouting equipment capable of providing shallow formation hydrofracture pressures to highly specialized equipment designed for large scale oil field fracturing.

8.3.5 Effectiveness of Hydrofracturing

Hydrofracturing is frequently used in the oil industry with a great deal of success to improve the production of wells (Campbell & Lake, 1973). Hydrofracturing may have little or no effect on the bedrock below Hyde Park due to the present fractured state of the Lockport Dolomite. An additional concern would be the aggravation of the environmental impact. For example, artificially created fractures may extend down into the Rochester Formation creating potential paths for the further migration of chemicals.

8.4 GROUTING THROUGH CONTROLLED FRACTURES

Grouting through controlled fractures is generally described as the combined application of hydrofracturing and pressure grouting to seal a bedrock interval. The practice of grouting through controlled fractures was initially developed by the nuclear industry as a means of safely disposing of nuclear waste. The disposal method involved the use of hydrofracturing or blasting of competent rock formations into which mixtures of grout and nuclear waste could be injected and "safely" disposed.

For the purposes of containment at Hyde Park, this alternative is addressed in the section regarding grout curtain walls.

8.5 PURGE WELL SYSTEM - HYDRAULIC CONTAINMENT

The use of groundwater extraction wells (purge wells) to artifically create or alter groundwater conditions is an accepted means of controlling contaminated groundwater migration. This method of hydraulic containment incorporates the use of pumping wells to remove groundwater in such a manner as to reverse or arrest the natural groundwater gradient and thus contain contaminant plumes. The pumping system can be augmented by various recirculation systems, treatment systems and physical barrier systems to contain or rehabilitate the aquifer.

8.5.1 Applications of Hydraulic Containment

Hydraulic containment incorporates the pumping of water to artificially adjust the groundwater flow. The purposes of this include:

1) to lower a portion of the water table thereby severing hydraulic connection with contaminated or uncontaminated sinks or sources;
2) to contain groundwater plumes and arrest the migration of a contaminant plume;
3) to extract contaminated groundwater for treatment, and

4) to purge or flush contaminants from an aquifer.

8.5.2 Specifications for Hydraulic Containment

Hydraulic containment is site specific and seldom subject to general specifications. Therefore such specifications must be developed as the system is designed and may be arrived at on a performance or other basis. The use of mathematical modelling will assist in the formation of conceptual hydraulic containment systems but the overall acceptance of such a system is typically based on an in place performance evaluation. Specifications for each purge well site must be individually developed to address the requirements of each area because of the differences in conditions.

Specifications will concern such items as:

1) well diameter and depth,
2) screened interval and sand/gravel pack configuration, if required,
3) pump size and pumping rate,
4) location of purge and recirculation wells
5) purge and recirculation system infrastructure
6) number, types and locations of monitoring wells.

8.5.3 Effectiveness of Hydraulic Containment

Hydraulic containment through the use of purge wells can be an effective means of containment in a porous ~~medium such as a~~ fractured bedrock. However, at the Hyde Park Site the aquifer plumes in the bedrock consist of non-aqueous as well as aqueous phase liquids. While the control of an APL plume in bedrock is a predictable phenomena, the movement and control of a NAPL plume in bedrock is not as readily predictable. Since the NAPL with high specific gravities (compared to water) tends to travel under gravitational forces, control through hydraulic containment may be possible by creating a resisting force of groundwater movement in the opposing direction equal to or greater than the gravitational force driving the NAPL.

Since extraction wells may encounter two different groundwater phases, special consideration must be given to the type of pump selected. A pump capable of handling APL may not be suitable for pumping NAPL. As a result, a two pump system may be necessary.

To enhance the effectiveness of a hydraulic containment system, the following technologies should also be considered:

1) the placement of auxiliary wells or barriers in zones of high specific hydraulic activity;

2) injection or recirculation of material to manipulate the cones of influence of the purge wells and assist in the rehabilitation of the aquifer, and;

3) adjustment of the rates of pumping from a series of wells to produce an overall beneficial impact on the groundwater flow pattern.

Due to the presence of NAPL in the bedrock aquifer, it may be possible to increase the effectiveness of the purge well system through the use of pulse pumping. By allowing water conditions to continually revert to prepumping conditions, it may be possible to apply repetitive driving forces on the NAPL slugs to stimulate NAPL mobility. This may result in additional NAPL recovery from the bedrock aquifer.

8.6 INJECTION SYSTEMS

Injection systems are typically used in conjunction with purge wells to enhance the effectiveness of the recovery system. Injection systems can be used in two separate ways to assist in the containment of hazardous wastes. The first method is as part of a hydraulic containment system and the second is as a purging medium. In a hydraulic containment scheme, water is injected at a predetermined location to create a desirable groundwater effect. Through this method of groundwater manipulation, a specific portion of an aquifer can be isolated.

Injected water can also be used to purge contaminants from an aquifer. This is accomplished through injection at an upgradient location. As the water migrates through the aquifer regime it takes the pollutants into solution, and carries them to the purge well where they are pumped to the surface and treated. In some instances, the water is recirculated. It is to be noted that the effect on the NAPL plume of purging with water is unknown. This is due to the fact that Hyde Park NAPL is typically denser than and immiscible with water and therefore its movement is unpredictable and would require testing.

Purging of contaminants from soils or rock can often be enhanced by the use of additives in the injection waters. The injection of air, steam or chemical solutions may also be applicable to aquifer restoration.

8.6.1 Injection Solutions

Injection methods applicable to the APL plume are based on hydraulic containment. Flushing of the area with injected fresh or recirculated water and treatment of the extracted waters over a period of time will lower the chemical concentrations. The addition of air or chemical solutions may serve to accelerate the process.

Through the use of injection, fluids may also be used to reduce the mobility of NAPL constituents and effectively lock them in place. Unfortunately, no such fluids presently exist which can address the wide variety of chemicals in the Hyde Park NAPL. However, this could not be accepted as a permanent solution as it would only slow down the migration of NAPL.

The injection of chemical solutions, such as alcohols or ketones may serve to release the NAPL from pore spaces, thus improving mobility and extraction. The major drawback in the use of chemical solutions to increase NAPL mobility and accelerate aquifer restoration is that the addition of an injection fluid into an aquifer containing the wide range of chemicals and chemical concentrations known to exist at Hyde Park may result in undesirable and uncontrolled chemical reactions.

Reclaiming NAPL may also be improved by implementing enhanced oil recovery techniques. Such techniques as presented by Van Pollen (1980) are listed below and further described in Appendix E. As with fracturing and injection fluid applications, the concern with applying oil recovery technology to the NAPL would be mobility causing an adverse impact.

A. Thermal Processes

Thermal processes account for approximately 70% of enhanced oil recovery operations. These processes usually apply to reservoirs of low gravity, high viscocity and high porosity. The three basic types of thermal processes are described as follows:

1) Steam Injection

An effective method whereby increased mobility of the oil is obtained through heating by steam injection. The formation is heated and the oil recovered through the same well.

2) Steam Flooding

This method uses injected steam to force the oil to other wells for collection.

3) In Situ Combustion

This method uses injected air, which sometimes includes pulses of water, to feed combustion within the formation, heating the geologic formation and oil to increase mobility and therefore collection.

B. Chemical Processes

Due to the complicated technology, high risk and high costs, less than 1% of existing enhanced oil recovery schemes in the U.S. involve chemical processes. The three major enhanced recovery chemical processes are described as follows:

1) Surfactant - Polymer Injection

This method involves the injection of a surfactant slug to lower interfacial tension followed by a polymer mobility buffer to increase production of the oil well.

2) Polymer Flooding

This method involves the addition of a thickening agent to injected water to push oil to recovery wells.

3) Caustic Flooding

This method uses high pH water having 1-5% NaOH added to push oil to recovery wells. Although the costs for this method are relatively low, the effectiveness is limited.

C. Miscible Displacement Processes

Miscible displacement processes involve the introduction of solutions into the oil field to reduce the forces holding the oil. Once the forces have been broken, the oil is driven to the recovery well. The three major miscible displacement processes are described as follows:

1) Miscible Hydrocarbon Displacement

There are three methods of miscible hydrocarbon displacement; namely miscible slug process, enriched gas process and high pressure gas process.

2) Carbon Dioxide Injection

Limited testing has been conducted using injected CO_2 or CO_2 plus water to increase the mobility of oil.

3) Inert Gas Injection (N_2)

Inert gas injection is generally a lower cost miscible displacement method, however, the number of cases where the right conditions exist to make use of this process is limited.

It is to be noted that enhanced oil recovery projects differ from the Hyde Park situation in that oil is lighter than water and the majority of the NAPL is heavier.

8.6.2 Effectiveness of Injection Systems

The use of injected foreign solutions to enhance the removal of NAPL or to accelerate the restoration of the aquifer are generally deemed to be of limited value for Hyde Park. The major factors influencing the effectiveness of injection solutions are:

a) the uncertainty of the potential chemical reactions resulting from mixture with chemicals in the aquifer,

b) the uncertainty of the effect of NAPL plugs on the recollection of the injected fluids,

c) the uncertainty of the effect of increased NAPL mobility on the bedrock regime,

d) the environmental impact of the injected solutions themselves, and

e) the associated costs of injection and injection materials.

The overall consensus is that injection with foreign materials is not a suitable technology for use at the Hyde Park Site.

However, the use of recirculation water to enhance NAPL recovery may be a suitable technology and will be evaluated.

8.7 EXTRACTION OF NON-AQUEOUS PHASE LIQUIDS

There are two NAPL extraction scenarios which could be considered for the Hyde Park site. The first approach is to remove only the available NAPL within the landfill above the bedrock. This would result in pumping depths of 30 feet or less. The second option is to attempt to remove the NAPL identified in the Lockport Formation. In this case, pumping depths could be as great as 150 feet.

Depending on the final RRT plan selected for the Hyde Park Site, removal of the NAPL may be effective technology to reduce the potential for future contamination.

8.7.1 Extraction from Landfill

Leachate is currently being collected in the Existing Barrier Collection System on the Hyde Park Landfill. The leachate collected includes NAPL as well as APL. Although this system prevents the discharge of leachate from the toe of the landfill slopes, it does not, nor was it designed to address leachate at depth in the overburden, the downward migration of leachate or the horizontal migration of leachate at depth. In order to assist in the removal of leachate, french drain laterals could be cut into the landfill and tied into one of the collection systems. Although such a system would be capable of lowering the leachate level and thus the driving force which is pushing the leachate into the bedrock, it would only address the problem to the elevation of the tile. In addition, open cut excavations into the waste materials are not desirable because of health and environmental concerns.

An alternative would be to install a system of purge wells into the on-site waste material and pump the leachate from the landfill. This may permit the extraction of NAPL to the installed depth at each pump site. It is to be noted that the Hyde Park landfill hydraulic characteristics are not ammenable to the installation of an effective purge well network.

8.7.2 Extraction from Bedrock

In the bedrock, it may be possible to collect NAPL from extraction wells strategically placed in the NAPL plume. These wells could be installed as part of a hydraulic containment system for the entire plume control program or as separate wells installed specifically to collect NAPL. If such a system was deemed effective, the installation of an on-site NAPL collection system would be redundant.

Physical removal of NAPL in the bedrock by excavation methods is discussed in a subsequent section of this report.

8.7.3 Extraction Methodology

Through the pump tests performed to date it is apparent that any hydraulic purge well system designed must be expected to handle minor volumes of NAPL. The major hydraulic containment pumps can be selected on their ability to handle these minor volumes of NAPL. However, in order to effectively remove the NAPL from the landfill or the bedrock, the pumps would necessarily have to be placed in areas of major NAPL volumes. As a result, it may be necessary or desirable to install dual pump systems; one of which pumps the NAPL and the other which would pump APL.

The oil recovery industry proved to be the best source of applicable technology for

NAPL extraction, however most of these systems were geared to high production recoveries of large product reservoirs which are not always applicable or economically viable for smaller situations.

In order to effectively remove NAPL from either the landfill or bedrock regime, a series of tests would be required to determine the suitability of extraction methodologies. The results of the testing and mathematical modelling would be used to determine the pumping system parameters. This would include well locations, extraction intervals, recirculation intervals, pumping rates, well spacing and phreatic surface conditions. The final evaluation of effectiveness of such a system would be based on an in place performance criteria.

8.7.4 Extraction Effectiveness

8.7.4.1 Landfill

The removal of NAPL from the landfill could be effective in reducing the potential for chemical migration in the bedrock aquifer. The suitability of such a remedial effort would depend on the remedial works selected for the existing condition in the bedrock. Where a physical barrier (i.e. grout curtain wall) is constructed in the bedrock around the site, the addition of the landfill NAPL into the bedrock regime already containing NAPL will not significantly change the current condition. However, if the existing NAPL is to be handled only by purge wells, the elimination of a continuing NAPL source may be a suitable remedial effort.

Based on the fact that the disposal of the waste materials at the Hyde Park site did not follow a disposal plan, it is unlikely that an effective purge well system could be constructed on the landfill site. It is surmised that the pockets of NAPL will be scattered across the site and that the effective cones of influence that could be created by a purge well system will be severely limited by the heterogeneity of the waste materials disposed at the site. The installation of purge wells could therefore be a "hit or miss" operation. Installation of an on-site purge well system would require large diameter boreholes into the waste materials which is not a desirable undertaking from an environmental point of view.

Furthermore, it is possible that the NAPL within the landfill has reached an equilibrium condition and that NAPL is no longer being driven into the bedrock formation. It is also conceivable that the volume of NAPL which migrated into the bedrock was a function of the site operation during the active disposal years of the site and therefore further migration into the bedrock formation has since ceased. Therefore, it is concluded that the installation of NAPL purge wells into the landfill is not an effective technology.

absolutely no evidence
support this

8.7.4.2 Bedrock

While it is true that the removal of mobile NAPL from the bedrock would eliminate a possible source of chemicals, the removal of NAPL from the bedrock does not necessarily remove the potential for chemical migration. Although it may be possible to remove the NAPL from the void space within a bedrock fracture (mobile NAPL), it will not be possible to remove the coating of NAPL from the fracture surfaces or NAPL trapped by capillary forces (non-mobile NAPL). The net effect of this is that the long term potential for future chemical migration has been reduced. However, in the short term, the potential for environmental impact may have increased since the evacuated fracture is now susceptible to APL transfer.

The pump tests conducted during the bedrock aquifer survey showed that the pumping of the groundwater and NAPL could increase the permeability of the aquifer. Pump Well C-5A is an example of this. It was observed that during pumping, the removal of NAPL from fractures resulted in an increased transmissivity. In consideration of the entire bedrock aquifer environment, it is possible that the presence of NAPL in the fractures beneath the site has reduced the flow of groundwater through the area. As a result, the presence of NAPL is in itself somewhat of a limiting factor for groundwater movement. The only generation of chemicals in the NAPL filled fractures is through groundwater contact with the leading edges of the NAPL plume. Thus historic chemical migration near the NAPL plume was limited as a result of reduced transmissivities from plugging of the pore space by NAPL.

It is recommended that NAPL extraction be limited to the capability of wells installed to arrest NAPL movement and that augmentation with recirculation wells be evaluated. N6

8.8 FIXATION/STABILIZATION OF WASTE - IN PLACE

The fixation of waste in place is a method of waste containment in which a fixing agent is pumped either into or around the waste. The injected agent is designed to fix the waste in an encapsulating structure. Once cured, the capsule is inert and impermeable and thus the waste is no longer a threat to the groundwater environment. Stabilization is a similar technology in which material is added to solidify/stabilize the waste thus reducing the potential for migration.

Other in-situ treatments are capable of converting the waste into a less harmful compound which is more acceptable to the environment. However, these treatments are capable of treating only very specific waste materials.

The fixation and stabilization of waste in place is presently not a proven technology. Although successful fixation projects have been undertaken for one specific contaminant or common group of contaminants, there is not a proven technology that can be applied to the chemical mixture contained at Hyde Park.

8.9 SLURRY WALLS

Slurry wall construction is a technology which is generally oriented to groundwater control in unconsolidated materials. The purpose of the slurry wall is to reduce or redirect the flow of groundwater. For an overburden slurry wall, a trench is excavated and backfilled with a designed low permeable material such as concrete or a soil and bentonite mixture. During the excavation operation, the trench is kept open by use of a slurry consisting of a bentonite/water mixture.

As an alternative to slurry wall construction, the use of vibratory beam injection methods are sometimes employed to install positive cut off walls. These thin walled members are constructed through installation of piles or sheets into the overburden regime. As the piles or sheets are extracted, the void created is grouted under pressure. Typical injection grouts include, bentonite, cement, asphalt or combination mixtures to create the desired effect.

As discussed in Section 5.0 chemical plumes in the overburden are contained hydraulically. Therefore slurry wall construction would be redundant.

The application of slurry wall technology to a containment system in the bedrock is not appropriate. In the bedrock, a continuous grout curtain wall (as described in Section 8.2) was considered and it was concluded that the appropriate technology for Hyde Park would be a conventional grout curtain wall in the bedrock.

8.10 WASTE RELOCATION SCHEME

One alternative evaluated as a remedial solution for the Hyde Park Landfill Site is the removal of the waste material through excavation and redisposal in a secure landfill site. There are many prohibitive factors which would have to be addressed in such an undertaking. The combination of these factors and the physical constraints regarding removal tend to cause the cost to be prohibitive. An estimate of the cost of waste relocation is presented in Appendix D.

The most important factor to be considered in a waste relocation plan is that in order to be effective, it would be necessary to remove not only the landfill wastes but all material (surface, overburden, bedrock etc.) contaminated with the waste. This would include those portions of the overburden and bedrock regimes which contain NAPL. The presence of NAPL in the geologic environment must be considered as a potential source of contamination. Any groundwater contacting the NAPL plume has the potential of becoming contaminated. As a result, in order to fully eliminate this potential, all NAPL bearing soil and bedrock must also be removed.

8.10.1 Factors For Consideration

There are several factors to be addressed in the proposed waste relocation scheme. Each of these factors in some way either enlarge the scope or place restrictions on the methodologies employed for the relocation program. The net effect of these factors is that they ultimately impact the final cost of the relocation program.

Some of the major factors requiring consideration are listed as follows:

1. Siting and approval of a new hazardous waste disposal site for the Hyde Park wastes at a suitable nearby location,

2. Monitoring programs for the newly approved site,

3. Waste transport to approved site
   - waste enclosure during transport
   - use of public streets for transport
   - decontamination of vehicles for off-site travel,

4. Groundwater control during excavation
   - overburden
   - bedrock,

5. Treatment of collected groundwater prior to discharge,

6. Treated groundwater discharge system,

7. Surface water control program
   - within excavation
   - around perimeter,

8. Relocation of high voltage electric transmission lines,

9. Property purchases
   - land
   - residences
   - industrial facilities
   - commercial facilities,

10. Building demolitions - salvage value,

11. Effect of repeated bedrock blasting on surrounding residences and facilities (seismic shock wave study),

12. Air vapor control measures,

13. Air particulate control measures,

14. Safety program
    - on-site workers
    - surrounding neighborhoods
    - safety equipment
    - supervision
    - worker facilities,

15. Site security,

16. Access road construction,

17. Slope stabilization
    - overburden
    - bedrock,

18. Location of backfill material source,

19. Rehabilitation of site,

20. Permits and licenses,

21. Legal participation.

8.10.2 Waste Removal Quantification

In order to quantify the amount of overburden and bedrock to be removed, the following assumptions have been made:

1. All bedrock identified to contain NAPL (as defined by the Hyde Park Aquifer Survey) will be removed.
2. All overburden overlying the above identified bedrock intervals will also be removed.
3. All bedrock will be excavated to a depth equal to the lowest elevation of identified NAPL presence for each individual area.

Based on the above assumptions, the volume of overburden and bedrock removal are estimated to be as follows:

Overburden	2,800,000 C.Y.
Bedrock	10,200,000 C.Y.

8.10.3 Effectiveness

At the Hyde Park approval hearings, Mr. B. Mason, using smaller excavation quantities concluded that excavation was not a viable remedial alternative. The above discussion supports this conclusion. Therefore it is concluded that excavation is not a "requisite" technology.

8.11 SUMMARY

The evaluation of alternative technologies to contain the NAPL plumes in the bedrock aquifer resulted in the identification of two promising technologies. The first being hydraulic containment (purge well system) and the second being physical containment (pressure grouting).

Since the cost of hydraulic containment is in general substantially lower than the cost of pressure grouting, the use of hydraulic containment should be used wherever possible.

9.0 PROPOSED REMEDIAL PLAN - BEDROCK AQUIFER - PHASE 1

Based on the results of the bedrock aquifer survey and the alternative technology evaluations, it is concluded that the Requisite Remedial Technology for the bedrock aquifer would consist of one or a combination of hydraulic and physical containment.

To permit refinement of the design of these technologies, however, it is recognized that further field data is needed. In this regard the following testing programs would be carried out to collect the data needed for this purpose:

1) A program to determine the effectiveness of a hydraulic containment system to control NAPL and APL migration through implementation of a prototype purge well system.

2) A program to determine the effectiveness of a grout curtain wall as a means of controlling inflowing groundwater and further NAPL migration. (A test section is proposed).

Furthermore, a study to determine the effectiveness of the Rochester Formation as an aquiclude thus preventing potential vertical chemical migration will be conducted.

The proposed testing programs described in this section of the report will be the Phase I Remedial Action Plan for the Hyde Park Bedrock Aquifer.

9.1 HYDRAULIC CONTAINMENT INVESTIGATION PROGRAM

The practice of controlling groundwater movement through the use of hydraulic barriers is a well tested and proven remedial technology. Such a plan typically relies on the ability of pumping systems to manipulate the groundwater flow pattern into a desirable hydraulic trap from which the groundwater is extracted.

Hydraulic containment is a proven technology ~~for handling APL.~~ However, at Hyde Park the bedrock aquifer also contains NAPL. The Hyde Park NAPL is a second phase liquid which is typically heavier than water (some lighter than water NAPL also exists). NAPL samples collected indicate a specific gravity on the order of 1.2. As a result, the migration of NAPL in the bedrock groundwater regime is controlled to some degree by gravitational forces rather than hydraulic conditions as is typically the case with APL. A general lack of information regarding the NAPL's reaction to gravitational vs hydraulic influences in the bedrock has resulted in difficulties in establishing a final remedial program for the Hyde Park Site.

This situation is further complicated by the fact that the bedrock aquifer is a nonhomogeneous fractured media which results in irregular aquifer responses.

As a result of these uncertainties, the proposed remedial plan is composed of a series of prototype pumping stations which will be monitored and tested.

9.1.1 Well Placement

Since one purpose of the initial phase of the remedial program is to collect information regarding the flow characteristics and containment of the NAPL plume, all of the proposed wells will be set within the defined limits of the NAPL plume. Placement of wells in the NAPL plume has two other advantages:

1) Mobile NAPL collection.
2) The hydraulic gradient created by pumping in the NAPL plume will reverse the gradient of the APL

plume in adjacent downgradient areas near the well locations and will therefore hydraulically contain portions of the APL plume.

The groundwater modelling conducted to date, suggests that the most effective purge (extraction) well locations would be along the western and southern edges of the NAPL plume. This would permit the natural groundwater gradient to assist in the containment of the APL plume to the maximum extent possible. However, it is to be noted that placement of wells at the extreme western and southern edges of the NAPL plume may have an adverse effect in that the wells may draw significant quantities of NAPL into areas that previously contained little NAPL and would have remained so had the gradients not been adjusted.

Through evaluation of the above factors, the following three locations have been selected for purge well installations:

PPW-1 West of existing pump well C-5A on OCC property at the Field Office

PPW-2 Well D-7 area (Hyde Park Boulevard at Lafayette Avenue)

PPW-3 Between wells F-4 and H-3

A fourth purge well (PPW-4) may be installed following evaluation of pumping results from PPW-1, PPW-2 and PPW-3.

The proposed prototype purge well locations are presented on Figure 55.

9.1.2 Data Collection

As previously mentioned, the prototype purge wells are designed to collect aquifer information. The list of data to be collected will include:

1) general aquifer characteristics - transmissivity and storativity
2) groundwater volume extracted
3) areal extent of hydraulic effectiveness
4) variation of vertical groundwater flow
5) maximum extractable groundwater volume
6) characteristics and volume of NAPL collected
7) combined pumping effect of groups of wells on aquifer
8) equipment suitability
9) NAPL handling techniques

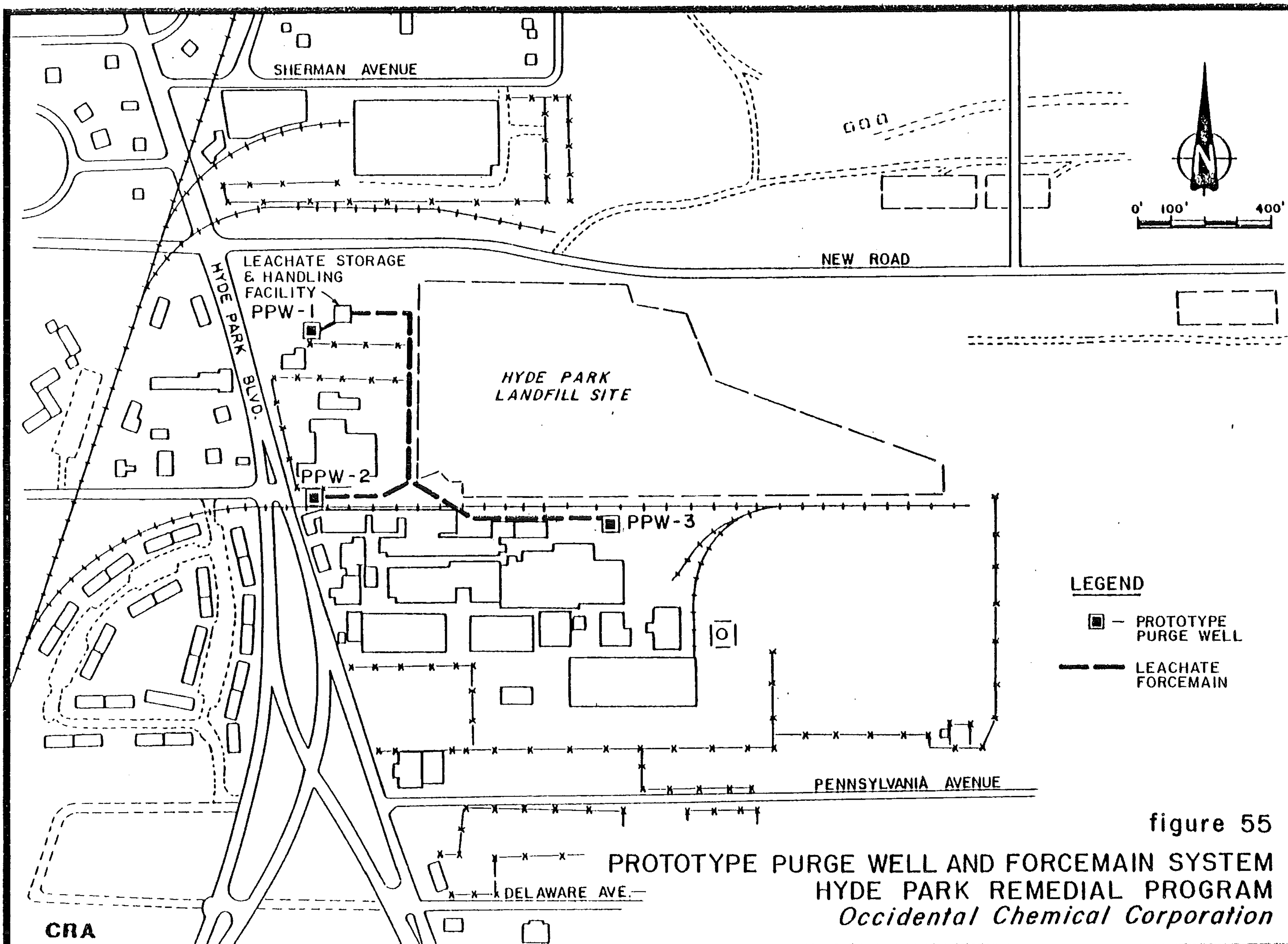

figure 55

PROTOTYPE PURGE WELL AND FORCEMAIN SYSTEM
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

10) treatment techniques for wastewater

11) effect of pulse pumping on removal of NAPL

12) effects of recirculation

9.1.2.1 Wastewater Volumes

All leachate extracted will be routed through a flow recorder so that leachate groundwater volume records can be monitored.

The percent of NAPL in the recovered groundwater will be determined periodically.

It is to be noted that the anticipated groundwater volume to be handled, will require an on-site carbon adsorption unit for treatment of the collected leachate. For this reason, an application for a permit to install and operate a carbon adsorption unit and to discharge the treated leachate to the City of Niagara Falls sewer system will be submitted. Various pretreatment methods are being evaluated by OCC at the Niagara Plant at the present time.

9.1.2.2 Groundwater Quality

During initial pumping, APL samples will be periodically collected and analyzed for TOC, TOH, pH, conductivity and phenols.

If sufficient NAPL volumes are available, measurements of specific gravity and viscosity will be taken. In addition, observations made during the visual inspection of the NAPL will be recorded including color and unique features. These measurements and observations will be conducted weekly.

9.1.2.3 Cone of Influence

Each well will initially be pumped at an approximate rate of 20 GPM.

Based on the observed drawdown at the purge well and surrounding monitoring wells, two other pumping rates will be investigated through extended testing (approximate duration 1 month).

Evaluation of the data at each well will provide a pumping optimization plan.

Combined cones of influence will be investigated. Various experiments of pumping combinations may be undertaken as aquifer conditions suggest.

It is to be noted that in order to properly evaluate the aquifer conditions, it may be necessary to discontinue pumping from particular wells for extended periods or to pump all three wells at once.

At some point in time during the testing, the pumping rate will be increased to determine the maximum possible extraction rate from each well.

Sixteen (16) monitoring wells will be installed around the prototype well locations using existing monitoring wells where possible. Of these, eight (8) wells will penetrate the entire Lockport Formation and will be equipped with multi level piezometers to monitor hydraulic heads throughout the aquifer. The intervals selected for monitoring will be based on the historical information available. The remaining eight (8) wells will monitor the upper Lockport regime.

The proposed monitoring well locations are presented on Figure 56.

9.1.2.4 Equipment

Maintenance records of all mechanical equipment will be kept to monitor equipment suitability and reliability.

During regular maintenance inspections, the equipment will be examined for wear and adverse affects of leachate handling.

9.1.2.5 Pulse Pumping

As an alternate means of promoting NAPL extraction from the bedrock aquifer, a program of pulse pumping will be undertaken. The program will involve the monitoring of NAPL volume that can be extracted through repeated pulsing. This is accomplished through repeated cycling of the pump. (During the on cycle, groundwater and NAPL move toward the pump; during the off cycle, the aquifer refills with water around the well so that when the pump is turned back on, the groundwater will create a surge of

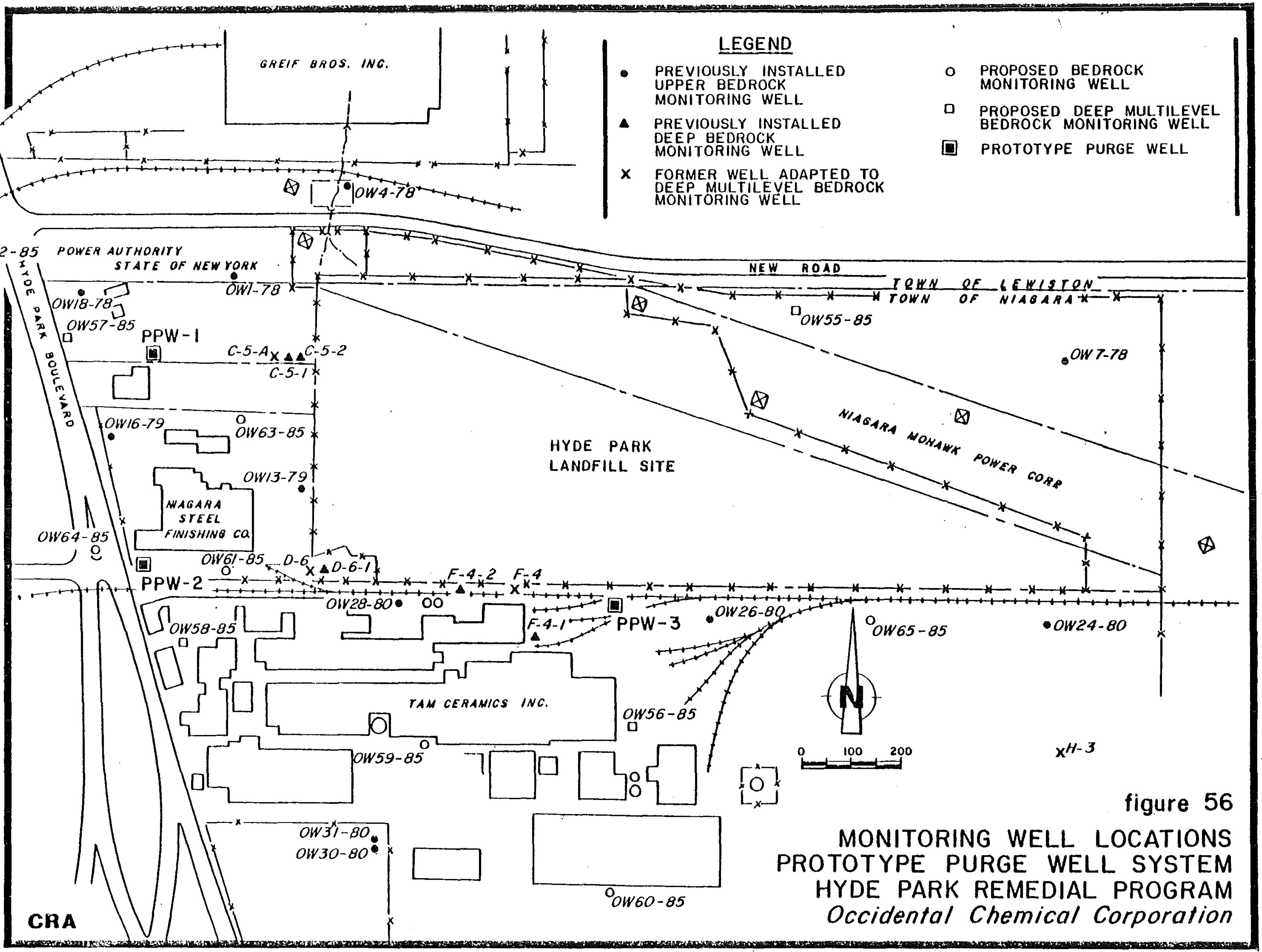

figure 56

MONITORING WELL LOCATIONS
PROTOTYPE PURGE WELL SYSTEM
HYDE PARK REMEDIAL PROGRAM
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groundwater and NAPL movement towards the pump again.) It is postulated that the surging motion of the groundwater may enhance the collectability of the NAPL.

9.1.3 Prototype Purge Well Installation

All new purge well installations will be constructed as follows:

1) auger through overburden to top of bedrock
2) insert a permanent 14" diameter steel casing and seal annular space between borehole wall and casing with cement grout
3) drill to top of Rochester Formation using an NX diamond drill bit
4) enlarge borehole to 12" diameter

Once complete, each well will be equipped with a pump and pumphouse. All metering equipment will be housed inside the pumphouse. The well will be fitted with a securable cover.

9.1.4 Monitoring Well Construction

Two groups of new monitoring wells will be installed in conjunction with the proposed prototype purge wells. One group will monitor the upper 15 to 45 feet of the Lockport Formation while the other group will monitor a series of intervals over the entire Lockport Formation.

The upper formation monitoring wells will be constructed as follows:

1) auger through overburden to the top of bedrock
2) insert a permanent casing and seal the annular space between the borehole wall and casing with cement grout
3) core to the desired depth with an NX diamond drill bit

The deep wells monitoring the entire Lockport Formation will be constructed as follows:

1) auger through overburden to the top of bedrock
2) insert a permanent casing and seal the annular space between the borehole wall and casing with cement grout
3) core to the top of the Rochester Formation with an NX diamond drill bit
4) based on historical hydrogeologic information, install water level recording instruments in selected water bearing intervals. The selected intervals will be backfilled with sand and the

monitored intervals separated with bentonite plugs

9.1.5 Wastewater Handling

Each prototype well will be serviced with a forcemain system connected to the proposed leachate storage and handling facility. All forcemains will be of welded construction.

At the leachate storage and handling facility, the leachate will be discharged into a decant tank to promote phase separation and sediment removal. The effluent from the storage tank system will be discharged to a carbon adsorption system for treatment.

9.1.6 Prototype Purge System Appurtanances

Through an evaluation of collected data, the suitability and shortcomings of the hydraulic containment system will be determined. As previously mentioned, there are supplementary alternatives which may be considered for enhancing the effectiveness of the prototype purge well system.

These alternatives include:

1) hydrofracturing
2) injection and purging with foreign materials
3) recirculation of groundwater

9.1.6.1 Hydrofracturing

During the pressure grouting investigation, the potential use of hydrofracturing to enhance grouting and the permeability of the bedrock formation, will be considered. This alternative will be evaluated following the preliminary tests for the grout curtain wall.

9.1.6.2 Injection and Purging With Foreign Materials

At the present time, the benefits of injecting foreign materials into the bedrock formation to accelerate the cleansing of the aquifer cannot be justified due to the risks involved. The primary risks are:

1) the toxicity of the injected material
2) the unknown affect of chemical composition changes due to the injected material
3) the unknown affect of increased mobility of NAPL on the bedrock aquifer environment

As a result, this alternative is not being actively pursued, however, development of an innovative injection material in the future would be considered.

9.1.6.3 Recirculation of Groundwater

The recirculation of decanted leachate back into the bedrock aquifer through injection wells has several environmental benefits:

1) Recirculation reduces the volume of leachate requiring treatment.
2) Recirculation can accelerate the restoration of the aquifer in the NAPL plume through additional flushing.
3) Recirculation would allow for the most cost effective use of carbon.
4) Recirculation may provide the final containment system with an added dimension of flexibility to assist in the manipulation of the groundwater flow regime.

Injection well(s) will be installed in appropriate locations when the cones of influence of the proposed prototype purge wells are identified. This is necessary to insure that the flow patterns of the recirculated wastewater are contained within the hydraulic barrier system created by the purge wells.

9.2 PHYSICAL CONTAINMENT INVESTIGATION PROGRAM

The construction of a physical containment barrier at the Hyde Park site is the second major technology to be investigated. The investigation of the physical containment technology will determine:

1) whether a barrier wall can be installed.
2) whether a barrier wall can be effective in waste containment if a hydraulic system is not capable of providing the required containment.
3) the effectiveness of a barrier wall in reducing the volume of groundwater handled by a purge well system.
4) the materials and specifications for a barrier wall.

For the purpose of this investigation, the physical barrier under consideration is the construction of a grout curtain wall section. The curtain wall is constructed through the injection of grout materials, under pressure, into a predrilled arrangement of boreholes in the bedrock formation. The injected grout penetrates the available fracture apertures in the bedrock. Once the grout is set, the permeability of the bedrock formation is reduced due to the presence of the grout in the previously open fractures.

The possible uses of pressure grouting techniques at the Hyde Park site include:

1) grouting to reduce the volume of groundwater collected by the purge well system.
2) grouting to manipulate the zone of effectiveness of a purge well system.
3) grouting to control the further migration of NAPL.
4) grouting to control the further migration of APL.

It is to be noted that with each of the above alternatives, the grout could be injected into specific bedrock fractures, specific areas or into the entire Lockport Formation to create the desired effect.

There are several factors to be investigated with regards to pressure grouting at the Hyde Park landfill to ensure that any proposed grouting plan is both physically and economically viable. Due to the uncertainties surrounding the use of pressure grouting in a chemically contaminated bedrock formation, it is recommended that the proposed physical barrier investigation be developed in iterative stages. The primary steps include:

1) A laboratory study of the suitability of grout materials.
2) An investigation of the groutability of the Lockport Formation including enhancement with hydrofracturing.
3) A field determination of grouting parameters in the Lockport Formation.
4) A field test of the grout curtain wall section.

Failure of the grout to pass any of the above tests will result in the need to reevaluate continuation of the physical barrier study.

9.2.1 STEP 1 - LABORATORY STUDY

The initial testing required for the pressure grouting program involves the selection of a suitable grouting material. Due to the economical advantage of cement grouts, chemical grouts are not currently being considered. The grouts under consideration are stable grouts. The combinations of grout compositions to be evaluated are as follows:

Grout #	Water	Bentonite	Cement	Flyash	Sodium Silicate
1	X	X	-	X(2)	-
2	X	X	X	X(2)	-
3	X	X	-	-	X + Reagent
4	X	X	X	X(1)	-
5	X	X	X	-	X
6	X	X	X	-	-

(1) No hydration properties by itself

(2) Will hydrate and set by itself

Each grout will be tested under variable proportioning. Grouts will be tested in ascending order of costs. (The above grouts are generally listed in order of ascending cost although the actual order is subject to the proportioning of components). If a grout mix at the bottom of the cost scale is found to be satisfactory, testing of further mixes may be cancelled.

The grout evaluation will be based on a series of seven tests:

1) final strength
2) setting time
3) bleeding characteristics (water separation during set time and filtration test - ability of grout to release water under pressure)
4) rheological characteristics - physical characteristics prior to set - viscocity
   - gel strength with time
   - shear strength with time
5) permeability to water
6) permeability to APL
7) permeability to NAPL (or an appropriate placebo)

The selected grout material will be retested for the above parameters using APL instead of water as the hydration agent. The purpose of this test is to simulate possible field mixing which could occur during grout placement.

As a part of the compatibility evaluation, it is important to assess whether the leachate mass behaves as a uniform substance or exhibits separation properties. This is of concern in cases where the isolation of leachate components result in extreme pH values or different rheological characteristics which may exert unusual stresses on the grout material. For this reason, centrifugation tests of NAPL and APL will be performed in which periodic extruded samples will be collected for observation of physical differences. The samples will be subsequently tested for pH, viscosity and specific gravity and compared to pre-test conditions. One additional set of NAPL samples will be collected from Wells C-5A, F-4 and D-6-1. These samples will be tested to determine variation with temperature.

9.2.2 STEP 2 - LOCKPORT FORMATION GROUTABILITY

In order to determine the groutability of the Lockport Formation, a field study of the bedrock will be undertaken. The study will involve the drilling and testing of four boreholes through the entire Lockport Formation. The test will be performed as follows:

1) An NX borehole will be drilled to the top of the Rochester Formation. (An accurate log of drilling information will be recorded.)
2) A remote control camera will be sent into the borehole to record the fractures and bedrock structures.
3) The Lockport Formation will be pressure tested with water in appropriate intervals consistent with the observed fracture spacing using the

Lugeon type testing format. A minimum of five pressure stages will be required from each test interval. Flow rates will be measured 10 minutes after flow stabilization.

4) The borehole will be grouted in the identical intervals used in the pressure testing program. Grout viscosities and setting time will be varied from one hole to another to compare grout penetration in each typical horizon. The grout material will be selected during the laboratory testing.

5) The water pressure tests, drilling logs, photography and grouting records will be correlated and evaluated to determine groutability characteristics.

9.2.3 STEP 3 - FIELD DETERMINATION OF GROUTING PARAMETERS

The laboratory testing program provides a grouting mixture which has been deemed suitable for the anticipated conditions. The purpose of the field testing program is to determine the in situ practicality of the selected grout and to determine the grouting parameters.

The objectives of the field testing program are:

1) Evaluate the groutability of the selected mixture.
2) Refine the grout mixture proportioning.
3) Determine an efficient grout hole pattern on a technical and economical basis.
4) Evaluate drilling methods and hole deviation. Correlate blind hole percussion drilling parameters with diamond coring logs.
5) Determine the practical grouting method; downward or upward, or a combination of both.
6) Measure in situ permeability.
7) Develop quality control procedures and performance tests for grout curtain evaluation.

9.2.3.1 Selected Mixture

Based on the results of the field grout tests, the grout mixture proportioning most suitable for the Hyde Park area will be determined. The evaluation process will be directed by an experienced grouting engineer.

9.2.3.2 Grout Hole Pattern

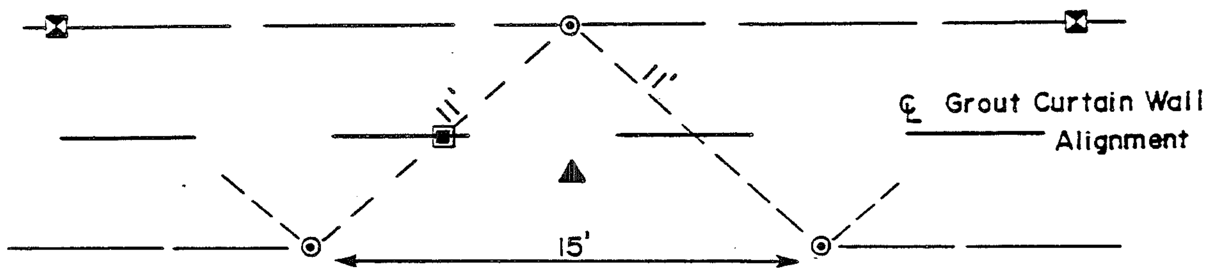
One of the prime objectives of the grout testing program is to develop an effective grout hole pattern. Since the intent of the grouting program is not to grout to refusal, the goal of the proposed grouting plan is to distribute the grout as efficiently as possible. This can be achieved by a double row pattern along the proposed curtain wall alignment. A double row pattern is more efficient than a single line pattern due to the variability of the bedrock formation.

The proposed testing program will consist of the installation of six test plots. Two distinct test plot spacings will be tested three times each. The two proposed test plot patterns are presented in Figure 57.

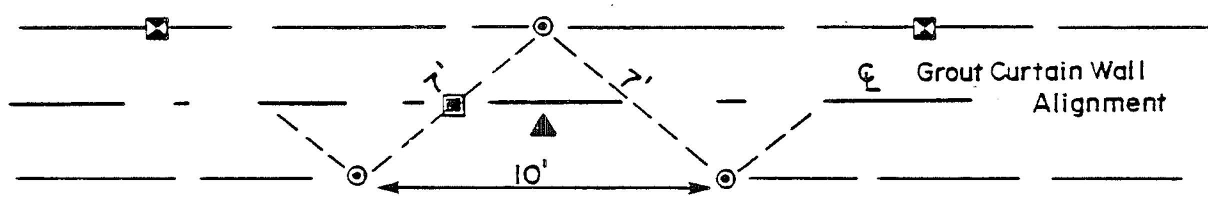
9.2.3.3 Drilling

On each plot, one hole will be cored and all others will be percussion drilled. Holes drilled with air will be flushed with water to wash out the rock flour. Deviation measurements of the verticality of each hole will be made to evaluate the accuracy of each drilling method.

Alternative A



Alternative B



Legend

- Test Plot Grout Hole
- Future Curtain Wall Grout Holes
- Control Borehole
- Auxiliary Grout Hole

figure 57

PROPOSED GROUT TEST PLOTS
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Records of drilling will be completed including time versus depth. Bit wear of the cutting instruments shall also be recorded. A study of the different drilling data will be made in order to try to correlate the percussion drilling with the cored samples.

At least one hole will be drilled with the down the hole method and one hole with the top hammer method for comparison.

Each borehole will extend through the entire Lockport Formation and penetrate approximately 5 feet into the Rochester Formation.

All groundwater from the drilling program will be discharged to the Hyde Park Leachate Storage and Handling Facility. During diamond drill coring, the return water may be recirculated.

All overburden cuttings will be evenly distributed at the drill site except in areas where the cuttings would interfere with restoration. Any overburden soils collected within the overburden plumes will be collected and transported to Hyde Park Landfill for disposal.

All rock cuttings will be collected and placed in sealed containers and transported to the Hyde Park Drum Storage Facility for future disposal.

9.2.3.4 Water Tests

Each hole will be water pressure tested using previous information obtained during the Groutability Testing Program to determine the interval length and pressure staging. The purpose of the water pressure testing is to correlate them with grouting pressures.

9.2.3.5 Drilling and Grouting

Where possible, the bedrock will be drilled full depth and the grouting will commence at the bottom and progress upwards. In problem areas, the drilling and grouting will progress downwards through the highly fractured rock and continue into the lightly fractured bedrock by drilling full depth and grouting from bottom up.

9.2.3.6 Grout Placement

As previously mentioned, the intent of the grouting program is not to grout to refusal, but to grout to provide an effective barrier. The proposed barrier width will be on the order of 20 feet. In order to determine the volume of grout to be injected, the following assumptions have been made:

a) Very fractured rock contains 10% voids maximum.

b) Lightly fractured rock contains 3% voids maximum.

* Note - These assumed values are subject to change based on the field testing results.

Based on the two test plot patterns, the amount of grout required per 10 foot stage is as follows:

Test Plot Pattern	Very Fractured (10% void)	Lightly Fractured (3% void)
A	150 cu. ft.	45 cu. ft.
B	100 cu. ft.	30 cu. ft.

A coefficient of filtration will be added to these quantities once the laboratory tests of the grout are completed. The above grout quantities represent the maximum quantity of grout which will be injected into a particular stage as any additional grout would theoretically be grouting beyond the desired limits of the 20 foot barrier width.

9.2.3.7 Performance Tests

Within each test plot, a control hole will be drilled with an NX diamond drill bit. The grout will be allowed to set for a minimum of 7 days (subject to laboratory determined set time) prior to drilling of the control hole. The permeability of the grout curtain wall will be tested by means of slug or recovery testing. Permeability slug testing may also be conducted using a placebo to compare to water testing. The permeability information will be evaluated to determine the most effective grout plot pattern. Additional testing may be undertaken.

If the observed curtain wall performance is not satisfactory, auxiliary grout holes may be added as shown on Figure 57. The auxiliary grout holes may be full depth or penetrate only a portion of the Lockport Formation depending on identified requirements.

9.2.3.8 Quality Control

An experienced grouting engineer would be employed at the site to oversee the grouting operations. An experienced drilling supervisor would oversee the drilling operations. The major areas of concerns are:

1) accuracy of drilling logs
2) verticality of grout holes
3) washing and cleanliness of grout holes
4) adequate proportioning and grout field friality control (viscosity, bleed, set time)
5) correct hydration and mixing of various products
6) proper grouting reports, flow/pressure charts, correct location of stages

9.2.3.9 Grout Equipment

Turbo-mixers will be used to mix the grout. Remixing will be required every 10 minutes.

Proportioning of the grout mixture will be by weight.

Metering of the grout will be through individual grout meters with paper printout recorders.

Grout pump capacity will be 15 gpm at 250 psi.

9.2.4 TEST SECTION - GROUT CURTAIN WALL

Based on the evaluation of test results collected during the preceeding testing programs, the suitability of a test section of grout curtain wall will be determined. The purpose of such a test section would be to determine the effect of the barrier on the groundwater flow patterns induced by the pumping of purge well PPW3.

9.2.4.1 Test Section Location

During the 72 hour pump tests conducted for the Hyde Park Bedrock Aquifer Survey, it became apparent that an elongated cone of influence was created by the pumping of well F-4A. This trough was noted to extend preferentially in the direction of observation well H-3. Based on this information, it is proposed that the alignment of the grout curtain test section straddle this trough as shown in Figure 58 (perpendicular to the observed trough and parallel to but outside of the defined extent of the NAPL plume).

Placement of the barrier across the trough will permit monitoring of the wall's effectiveness under potentially significant head differential conditions. Through comparison of the cones of influence of well PPW-3 prior to and after construction of the wall, an evaluation of the effectiveness of the wall as a groundwater manipulation tool can be made. Similar comparisons of groundwater volumes handled at well PPW-3 would also be evaluated. If necessary, limited testing will also be undertaken at existing well H-3.

9.2.4.2 Test Section Construction

The construction methodology and grouting materials employed during the test section installation would be based on the recommended

was created by the pumping of well F-4A. This trough was noted to extend preferentially in the direction of observation well H-3. Based on this information, it is proposed that the alignment of the grout curtain test section straddle this trough as shown in Figure 58 (perpendicular to the observed trough and parallel to but outside of the defined extent of the NAPL plume).

Placement of the barrier across the trough will permit monitoring of the wall's effectiveness under potentially significant head differential conditions. Through comparison of the cones of influence of well PPW-3 prior to and after construction of the wall, an evaluation of the effectiveness of the wall as a groundwater manipulation tool can be made. Similar comparisons of groundwater volumes handled at well PPW-3 would also be evaluated. If necessary, limited testing will also be undertaken at existing well H-3.

9.2.4.2 Test Section Construction

The construction methodology and grouting materials employed during the test section installation would be based on the recommended

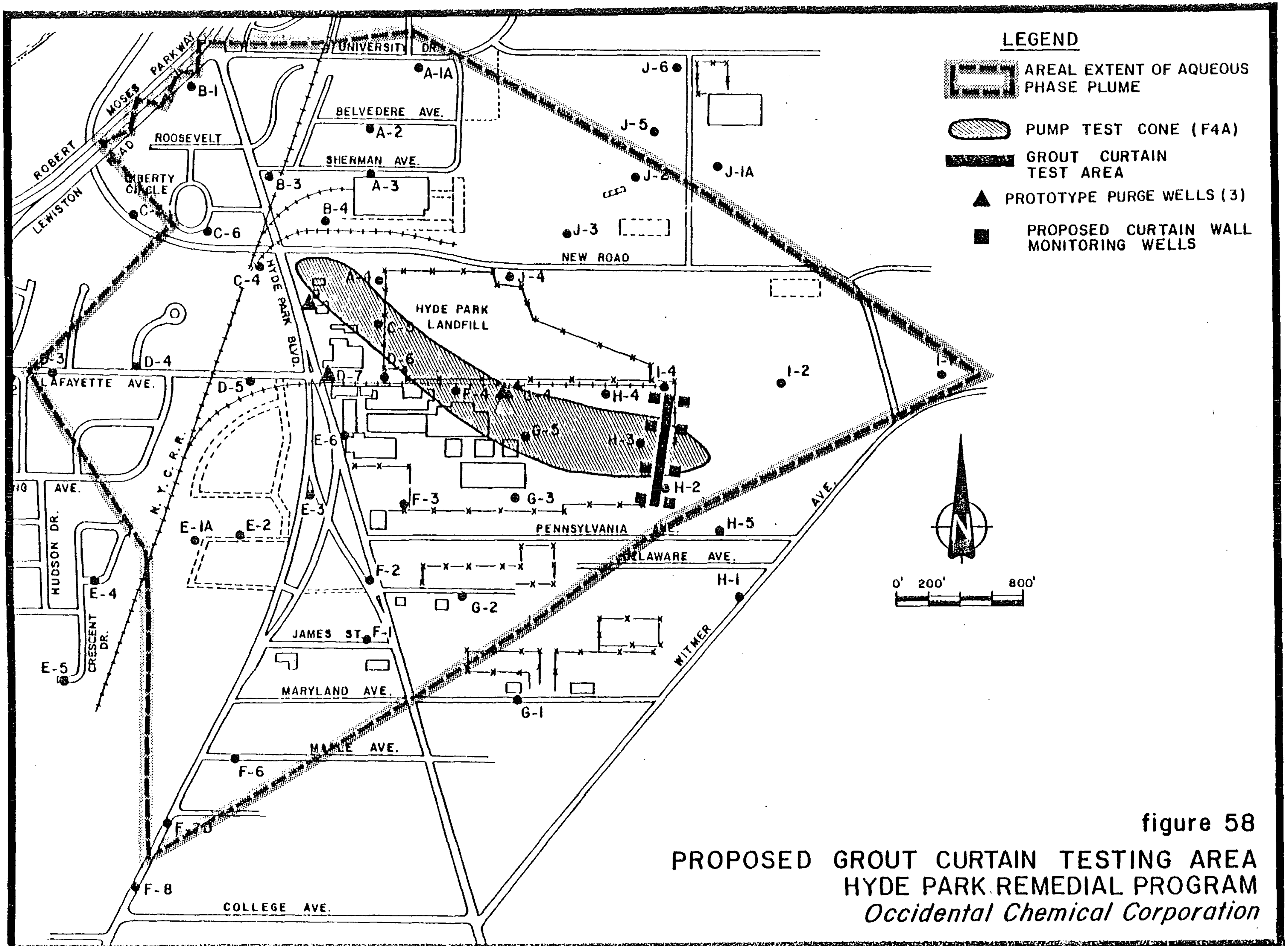

figure 58

PROPOSED GROUT CURTAIN TESTING AREA
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procedures outlined during the testing program. The length of the proposed test section may be as long as 700 feet although a reevaluation will be made at the conclusion of the preconstruction testing programs.

It is to be noted that all of the test grouting completed in the initial phases of the physical barrier study will be incorporated into the test section alignment.

9.2.4.3 Performance Tests

Every second test plot will include the installation of a diamond drilled control hole identical to those constructed for the initial grout test plots. These control holes will be slug or recovery tested monthly during the testing program to monitor variations with time.

9.2.4.4 Monitoring

With any test section, monitoring wells will be installed. In the case of a 700 foot test section, eight monitoring wells would be installed into the upper Lockport Formation. An estimated depth of 15 to 45 feet into the Lockport Formation is proposed. The monitoring well placement would be as shown on Figure 58. Two pairs of wells will be installed on either side of the wall to measure the gradient across the wall. The remaining four wells will monitor the end effect of the wall.

Each monitoring well will be constructed as follows:

1) Auger through the overburden to the top of bedrock.
2) Install a permanent casing and seal the annular space between the borehole wall and casing with grout.
3) Use an air percussion drill to penetrate to the required depth.

Each of the installed monitoring wells will be monitored according to the following schedule during pump testing startups conducted at well PPW-3:

- Hourly - for initial 6 hours of test.
- Daily - thereafter until stabilization.
- Weekly - thereafter for 4 weeks.
- Monthly - thereafter.

9.2.4.5 Waste Material Handling

All wastewater, exhaust air, overburden soils and bedrock fragments will be handled in the same manner as used during the field test program.

9.3 ROCHESTER FORMATION INVESTIGATION

The Hyde Park Bedrock Aquifer Survey Program identified the existence of chemicals from the Hyde Park Landfill throughout the entire Lockport Formation to the top of the Rochester Formation. In contrast to the dolomitic structure of the Lockport Formation, the Rochester Formation is a shale. The massive structure of the Rochester Formation determined from previous studies is expected to act as an aquiclude thus restricting further vertical migration of chemicals from the Hyde Park Landfill.

In order to investigate the effectiveness of the Rochester Formation as an aquiclude, wells will be installed into the Irondequoit/Reynales Formation and monitored. The wells will be located outside the perimeter of the plumes to preclude the possibility of vertical cross contamination.

It has been determined that groundwater quality monitoring of the Irondequoit/Reynales Formation is the only effective testing program of the Rochester Shale to evaluate its restriction to chemical migration.

The EPA/STATE concluded in a letter received by OCC on March 23, 1984 the following:

"In order to be convincing, the Rochester formation testing program must include evidence (that) the data from the single test location is representative of the conditions in the formation throughout the Hyde Park Landfill locale. It must be shown that the physical characteristics of the formation, observed at several locations are so similar that the formation may be called 'laterally uniform' and that any reasonable person would conclude that a single test site could reliably represent the permeability of the formation. Otherwise, the test must be duplicated at other locations to test the range of variability in permeability."

Evidence that a single or several test locations for permeability testing is representative of the condition in the formation throughout the Hyde Park Landfill locale cannot be collected. As a result, core collection and testing (physical and chemical) and permeability testing will not be done in the Rochester Shale.

same could be said about any geologic formation anywh

9.3.1 Irondequoit/Reynales Formation Investigation

At three (3) selected locations outside of the Hyde Park plumes, monitoring wells will be installed into the Limestone Formations underlying the Rochester Shale. The two (2) formations of interest are the Irondequoit and Reynales Limestone Formations. The installed wells will access both formations for the purpose of collecting groundwater samples. The locations of the proposed monitoring wells are presented on Figure 59.

9.3.1.1 Well Construction

Each of the installed wells will be constructed as follows:

1) auger through the overburden to the top of bedrock

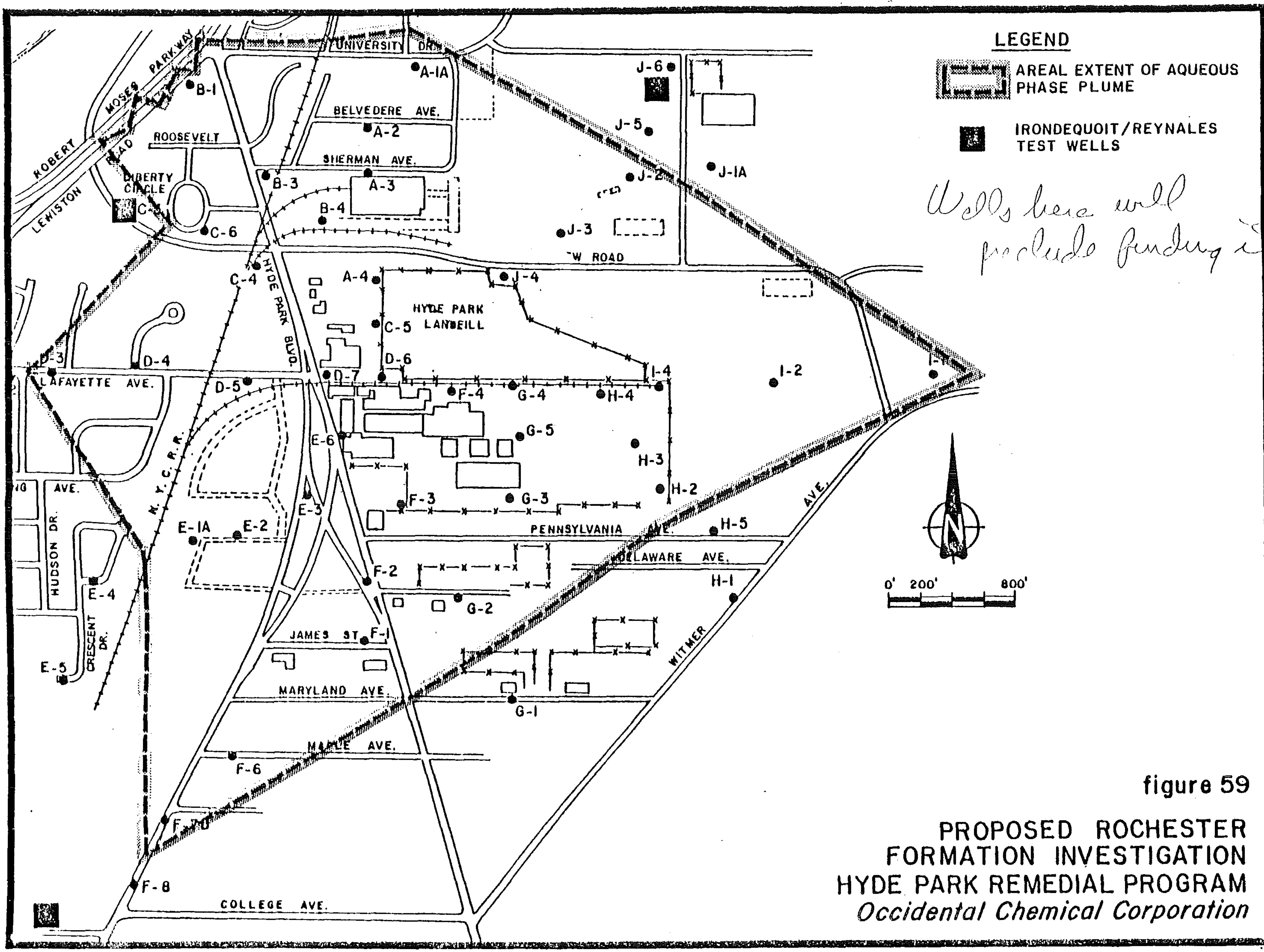

figure 59

PROPOSED ROCHESTER FORMATION INVESTIGATION
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2) install a temporary flush joint casing into the borehole

3) seal the annular space between the casing and borehole wall with bentonite to the ground surface

4) core with an NX diamond drill bit to within 15 feet of the base of the Rochester Formation.

5) enlarge the borehole to 6" diameter.

6) insert a 4" diameter steel casing into the borehole.

7) grout the annular space between the borehole wall and the casing from the base of the casing upwards. (Remove the temporary casing in the overburden as grouting progresses)

8) allow the grout to set for 48 hours

9) continue coring through the 4" casing with the NX diamond drill bit to the base of the Reynales Formation.

9.3.1.2 Sampling Program

Following the installation, the wells will be developed through flushing with injected water. The wells will then be allowed to equilibrate for a minimum of 2 weeks. Prior to sampling, the wells will be purged of 10 volumes of well water or until dry. Samples will then be collected and analyzed for the parameters used in the Hyde Park Bedrock Aquifer Survey.

9.3.2 Result Evaluation

At the conclusion of the sampling program, the collected information will be evaluated and a determination of the status of the Rochester Formation as an effective aquiclude will be presented. - what will happen if it is not an aquiclude

9.4 MONITORING PROGRAM

Final design of a remedial program will include the design of appropriate monitoring procedures. Information gathered as a result of the proposed testing program will therefore be useful in both remedial and monitoring program design. However, it would be premature to propose any specific monitoring at this time until further data has been collected and evaluated.

10.0 MODELLING OF PROTOTYPE WELLS AND GROUT CURTAIN SECTION

The following scenarios have been modelled to obtain an indication of their effect on the aqueous flow regime: 1) Steady state homogenous, isotropic conditions, 2) steady state conditions with the inclusion of defined higher hydraulic conductivity zones, 3) steady state, homogenous, isotropic conditions with purge wells and a 700 foot grout curtain section and 4) steady state conditions with purge wells, the 700 foot grout curtain section and the inclusion of defined higher hydraulic conductivity zones. The areal extent of the higher hydraulic conductivity zones was assumed to coincide with the lineaments as illustrated on Figure 48 of this report.

Figure 60 presents the areal extent of the higher hydraulic conductivity zones, the alignment and location of the proposed 700 foot grout curtain section and the locations of the prototype purge wells. For this analysis, the purge wells will have a purge rate of 20 gpm each.

Scenarios 1 and 2 have been previously discussed in Section 7.6.1.

GROUT CURTAIN SECTION AND HIGHER HYDRAULIC CONDUCTIVITY ZONES AND PURGE WELLS

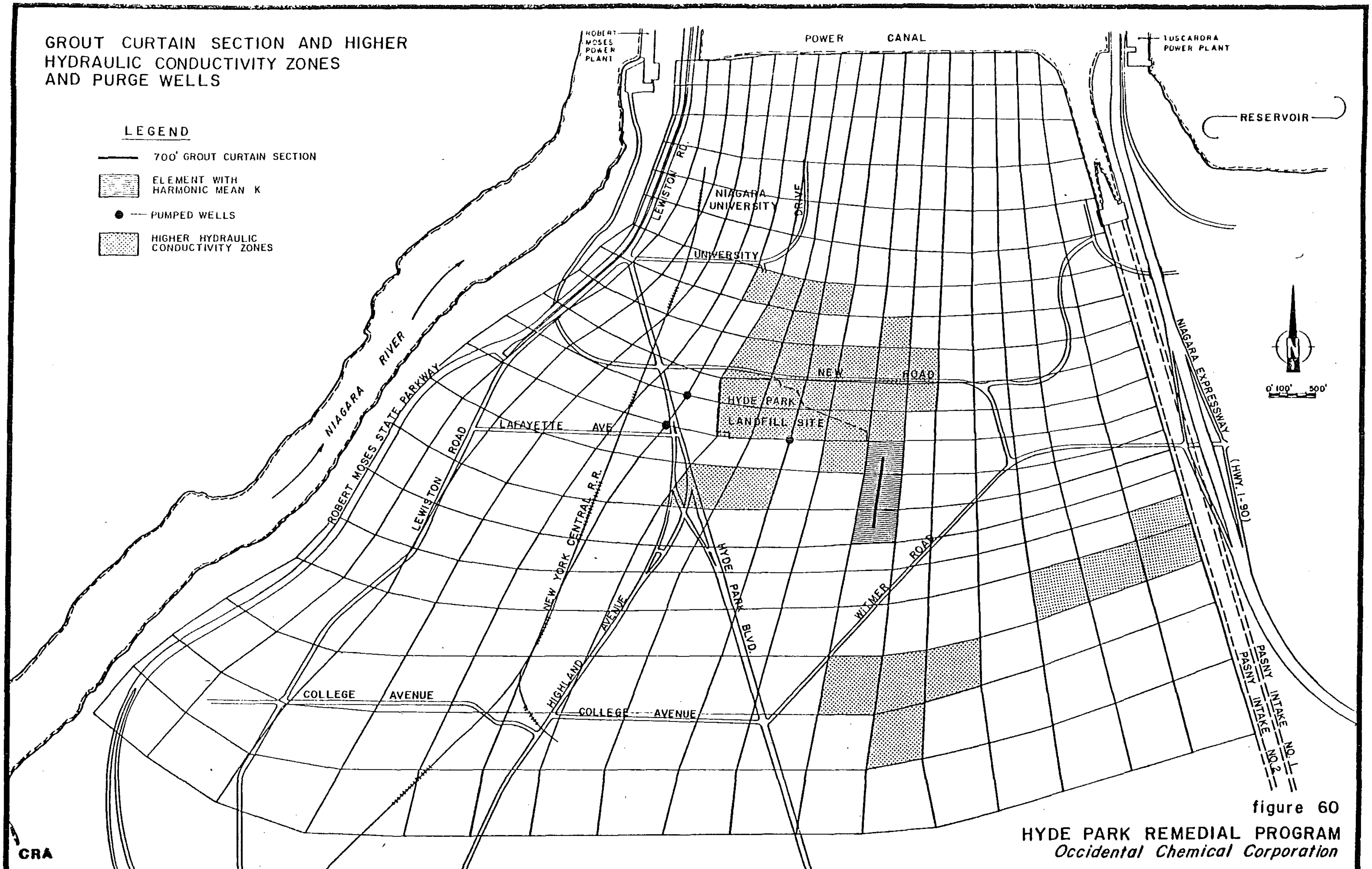

figure 60

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Figures 61 and 62 illustrate the phreatic surface and Darcy velocities for the third scenario respectively, while Figures 63 and 64 illustrate the phreatic surface and Darcy velocities for the fourth scenario. Analysis of the results indicate that with the inclusion of higher hydraulic conductivity zones, the phreatic surface elevations at the purge wells increases while the elevations in the higher hydraulic conductivity zones decrease. Thus, the gradient is decreased. This is the expected result based on Darcy's Law. For the same flow rate an increase in hydraulic conductivity will require a decrease in gradient.

To model the effect of the grout curtain section, a harmonic mean of the hydraulic conductivities was employed for those elements (113, 130 and 147) which contained the grout curtain section. The physical basis of this was to produce the same total head loss through an element with the harmonic mean hydraulic conductivity as the summation of the actual head losses of the individual materials in the element. The principles utilized to obtain the harmonic mean estimate are presented in Appendix F. Results employing an effective grout curtain section width of approximately 20 feet and hydraulic conductivities of 1.0 and 0.0283 feet per day for the aquifer and grout curtain section respectively produced a harmonic mean value of 0.326 feet per day. The elements assigned the harmonic mean values are shaded in Figure 60.

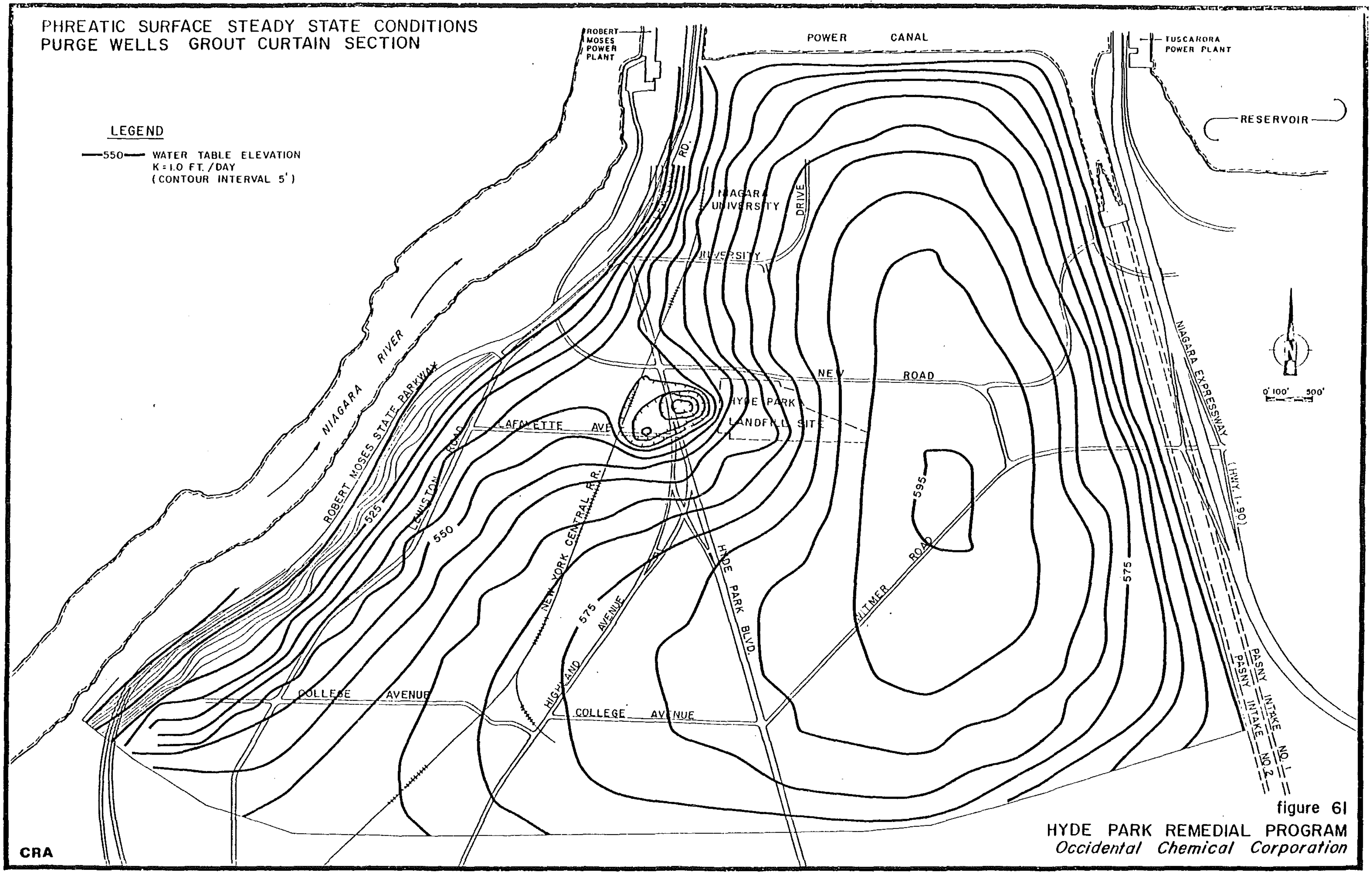

figure 61
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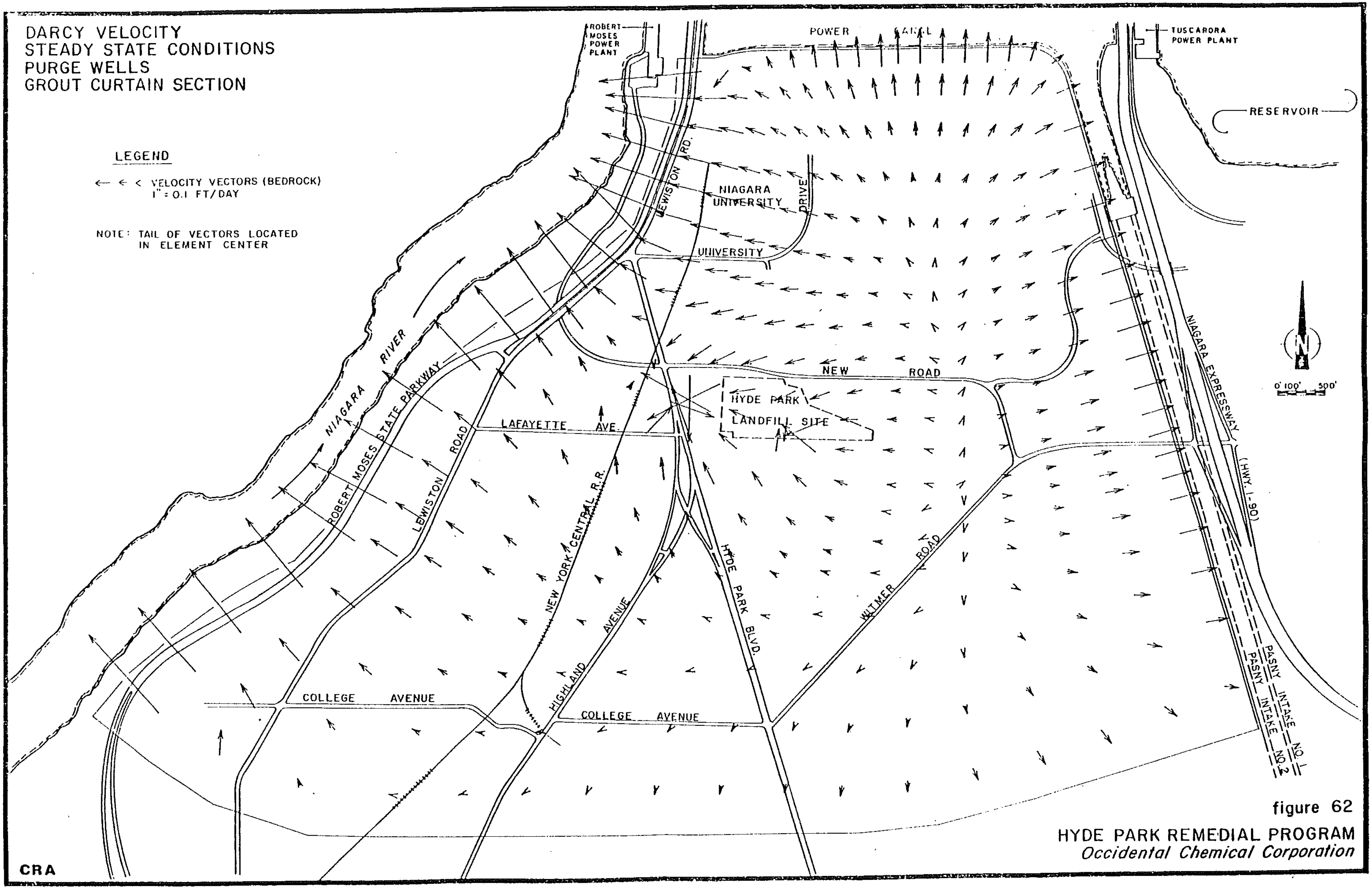

figure 62

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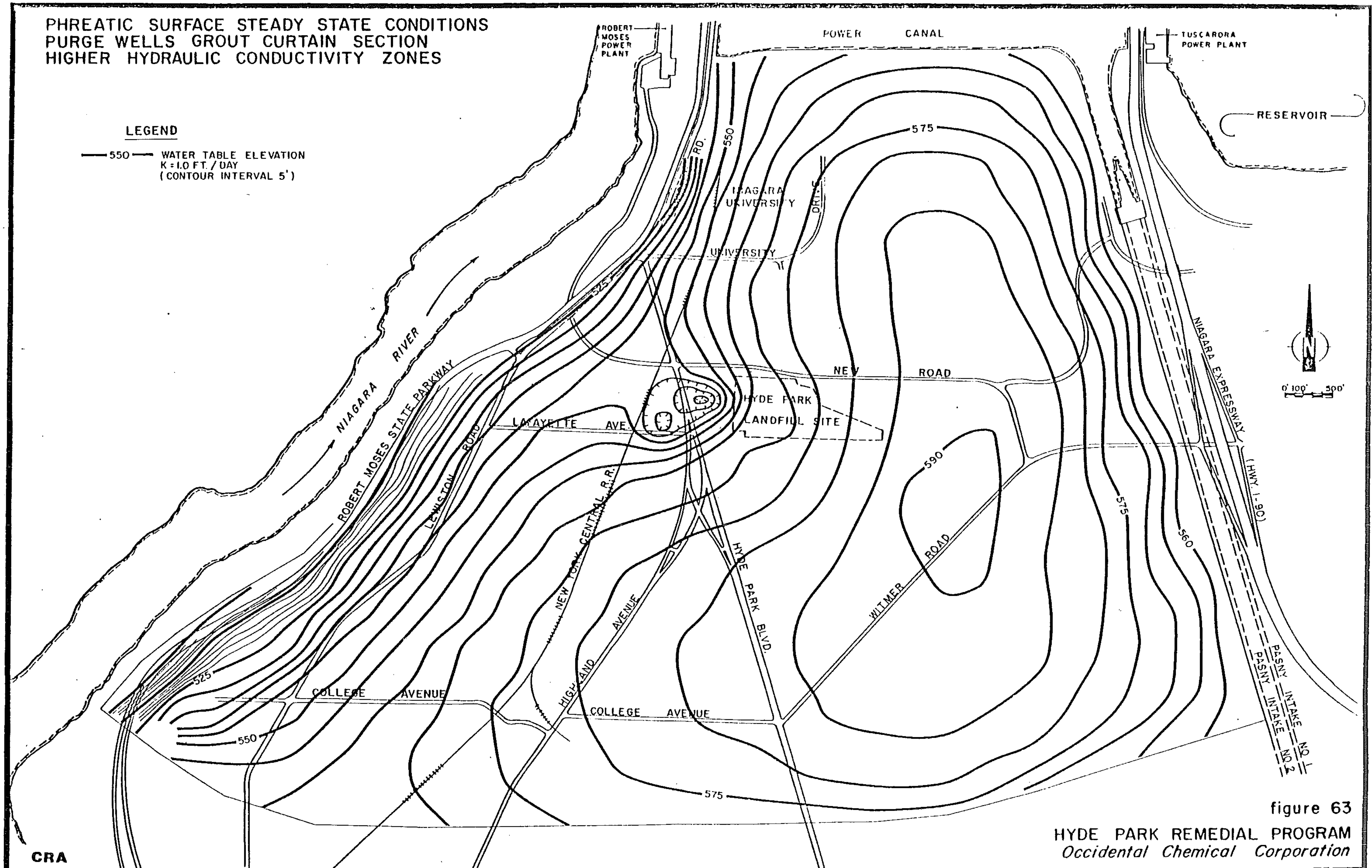

figure 63

HYDE PARK REMEDIAL PROGRAM
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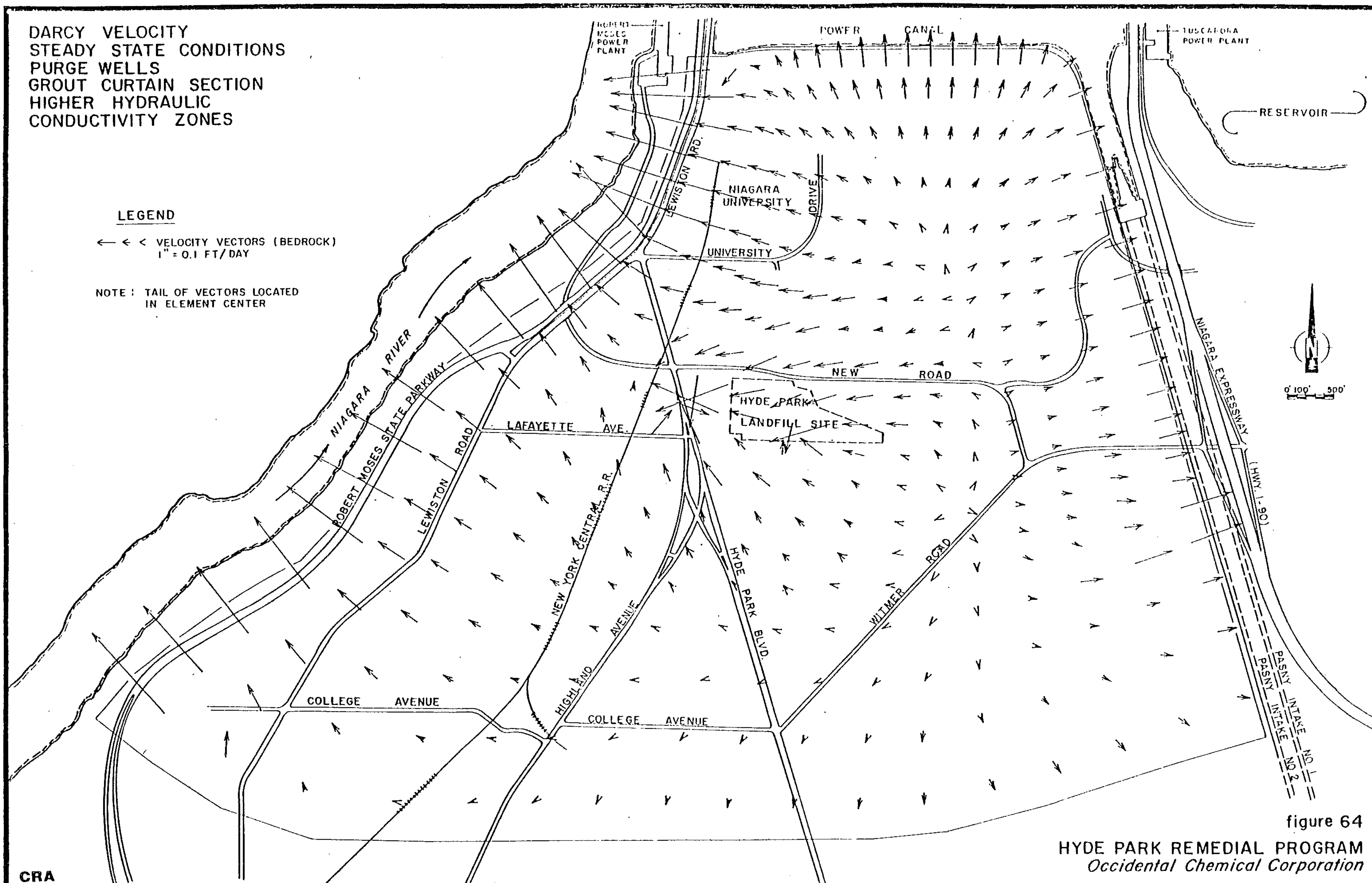

figure 64

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It is to be noted that when the harmonic mean hydraulic conductivity value is used for the entire element containing a grout curtain section, the plotted contours are not as closely spaced as would occur if the level of discretization were increased to allow individual elements to consist of the grout curtain section only. This should not affect the general pattern and magnitude of modelled velocities and water table elevations through the Hyde Park Landfill since the total head loss through the corresponding lengths for the two cases, coarse and fine discretization, would be approximately equal.

The effect of the 700 foot grout curtain section is confined to the nodes and elements immediately surrounding the elements containing the grout curtain section.

The results indicate low velocities through the elements containing the grout curtain section with higher velocities in the elements to the north and south of the grout curtain section. This indicates that the groundwater in this area is principally flowing around the ends of the grout curtain section.

The effect of the grout curtain section in blocking a higher hydraulic conductivity zone is not illustrated in the above results due to the previous assumption that these zones coincide with the lineaments. There are no higher hydraulic conductivity zones east of the proposed grout curtain section.

The effect of the wells is clearly illustrated on Figures 61, 62, 63 and 64. A significant drawdown zone has been created for both scenarios. Without the inclusion of the higher hydraulic conductivity zones, the high phreatic surface elevation exists between nodes 139 and 157 at an elevation of 595.35 feet AMSL. With the inclusion of the higher hydraulic conductivity zones the high phreatic surface elevation is located at node 139 with an elevation of 591.24 feet AMSL. The inclusion of the purge wells has shifted the groundwater high in the Hyde Park area approximately 600 feet to the southeast with respect to the phreatic surface discussed in Section 7.6.1.

The modelling results indicate that the cone of influence created by pumping the prototype wells at 20 gpm and intersecting the higher hydraulic conductivity zone with the grout curtain wall, should encompass a large segment of the NAPL plume.

11.0 SCHEDULE

The three major factors affecting the scheduling of the proposed plan are:

1) Approval and implementation of the Monitoring and Safety Programs for Special Construction Activities.
2) Approval and installation of the permanent storage facilities for leachate and an on-site leachate treatment unit.
3) Approval of the NAPL disposal plan.

At the present time, there are only two activities which could be undertaken without delay once EPA/STATE approves the proposals discussed in Section 9.0. These activities are:

1) Rochester Formation Investigation
2) Laboratory Testing of Grout Materials

There are presently no facilities to store the wastewater which will be generated by the activities proposed in Section 9.0. In fact, there are insufficient facilities to store NAPL presently collected by the existing barrier collection system. Thus, remedial activities at the landfill cannot proceed unless a permit for additional storage or incineration is issued in the near future.

Assuming that the scheduling problems described above can be resolved in the near future, an estimated timetable can be established. Figure 65 presents such a proposed schedule.

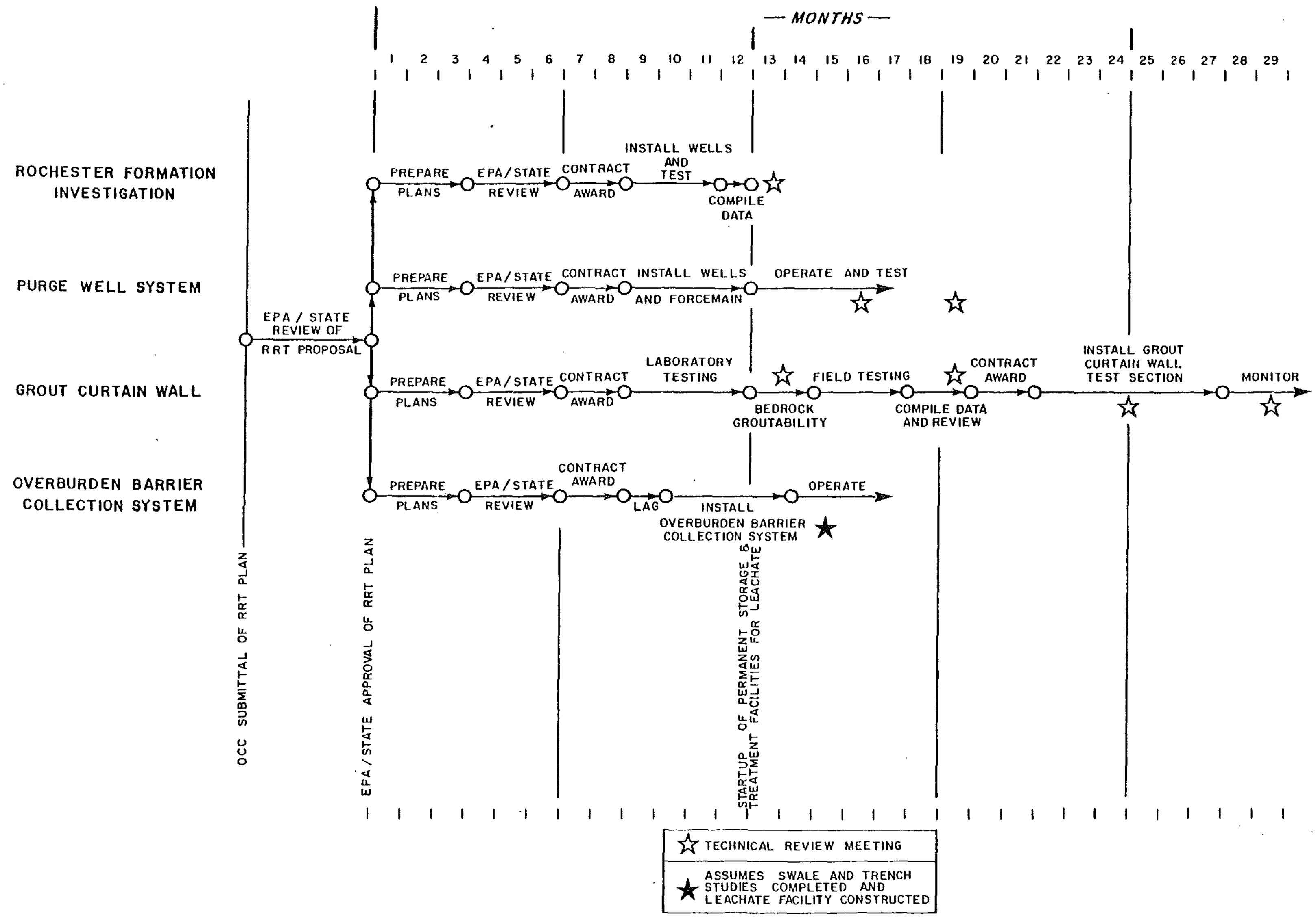

figure 65

ESTIMATED SCHEDULE
PROPOSED PHASE I - REMEDIAL PLAN
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

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All of Which is Respectfully Submitted

CONESTOGA-ROVERS & ASSOCIATES LIMITED

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APPENDIX A

PHASE I RRT PROGRAM

The following items have been evaluated and are recommended to form part of the Hyde Park Remedial Program.

1. <u>Overburden Barrier - Collection System</u> $ 930,000.00

   - consists of a gravity perimeter collection drain tile around three sides of the landfill site, embedded in a filter media bedding material ranging in depth from seven (7) to twenty (20) feet. Manholes are located at direction changes and at various locations for future inspection and maintenance, with a wet well at a strategic location and transfer equipment for pumping to the proposed leachate handling facility. Observation wells will be installed to monitor the performance of the system. All excavated material will be taken to the Hyde Park landfill site.

2. <u>Rochester Formation Investigation</u> $ 42,000.00

   - includes the installation of three (3) monitoring wells into the Irondequoit and Reynales limestone formations underlying the Rochester shale. The wells will be located outside the Hyde Park

Plumes. The wells will be used to monitor the groundwater quality of the Irondequoit/Reynales formation thereby testing the ability of the Rochester formation to restrict vertical chemical migration. Each monitoring well will be equipped with a dedicated pump and riser tubes for sampling purposes.

3. Hydraulic Containment Investigation Program (Prototype Purge Wells) $ 645,000.00

- consists of three prototype purge wells and pumping stations (PPW-1, PPW-2, PPW-3) each having a pump well the full depth of the Lockport Formation, a pump house complete with monitoring and sampling equipment and a double steel wall force-main, with leak detection system within the casing pipe, to transfer pumped groundwater to the proposed leachate handling facility.
- a fourth similar pumping station, prototype purge well No. 4 (PPW-4) may be located and installed following evaluation of pumping results from PPW-1, PPW-2 and PPW-3.

- to augment collection of bedrock aquifer information, sixteen monitoring wells will be utilized. Eight upper wells will supplement monitoring of the upper fifteen to forty-five feet of the Lockport Formation. Eight multi-level monitoring wells over the full depth of the Lockport formation will be installed to distinguish stratified piezometric pressure variances during pump testing.

4. <u>Physical Containment Investigation Program</u> (Grout Curtain Testing) $1,210,000.00

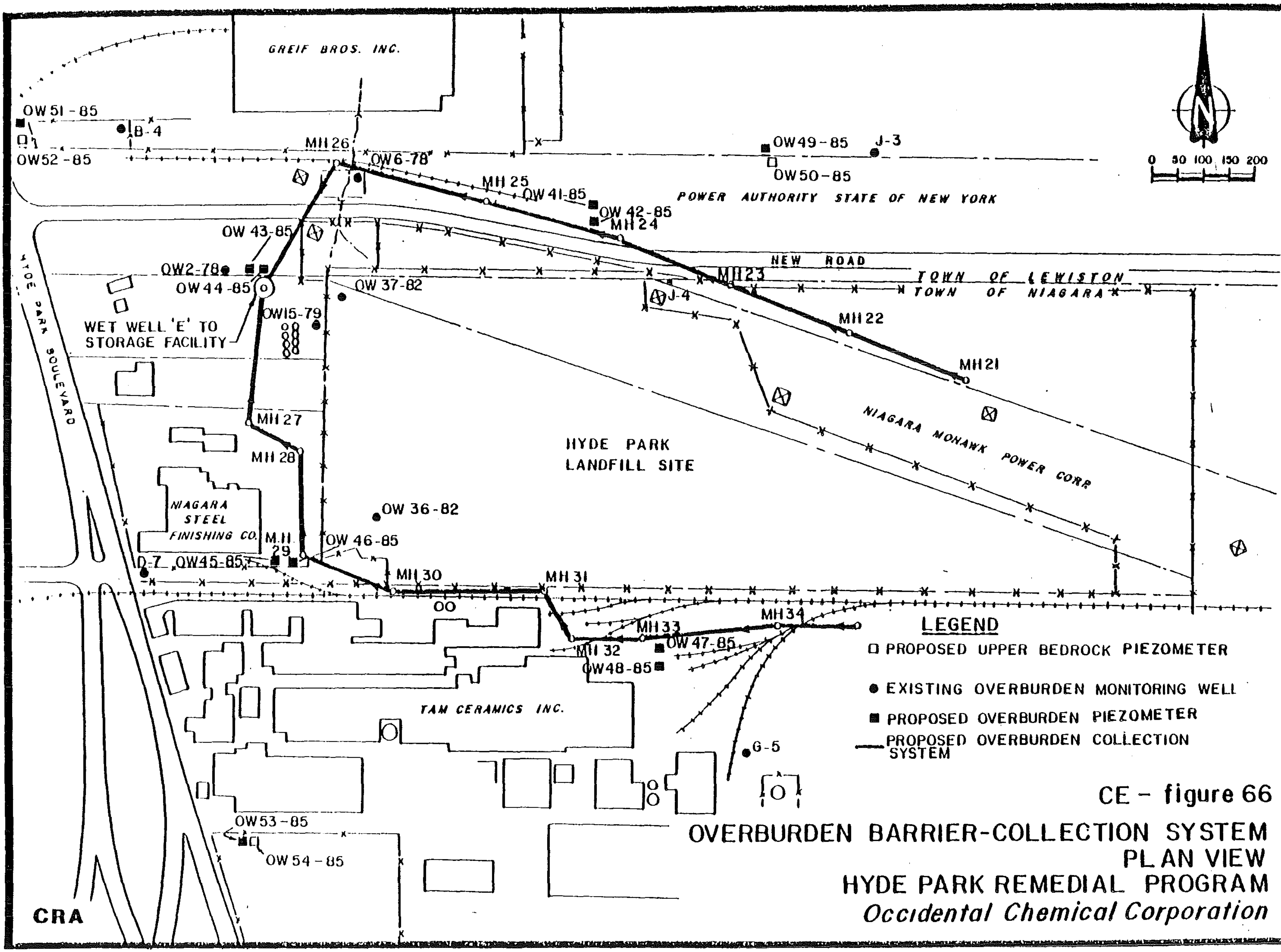
   a) consists of intensive lab testing to determine a suitable grout mixture(s) to be used during field testing. 90,000.00

   b) four NX boreholes drilled through the full depth of the Lockport Formation will be precisely logged, inspected by camera and subject to water pressure testing and subsequent grouting, in stages. The information collected will be correlated and employed to evaluate bedrock groutability. 63,000.00

   c) in situ drilling and grouting of six test plot groups consisting of three grout 320,000.00

holes and one control borehole. Various borehole patterns, drilling methods, grouting procedures and other parameters will be tested to determine the optimum grout curtain wall design.

d)	a 700-foot pilot curtain wall incorporating the preliminary test sections and test recommendations will be installed. Control boreholes will be installed to monitor performance of the wall with time.	710,000.00
e)	eight monitoring wells will be installed into the upper Lockport formation, two pairs on either side of the wall to measure the gradient across the wall with the remaining wells to monitor the end effects of the grout curtain test section.	32,000.00



CE - figure 66

OVERBURDEN BARRIER-COLLECTION SYSTEM
PLAN VIEW
HYDE PARK REMEDIAL PROGRAM
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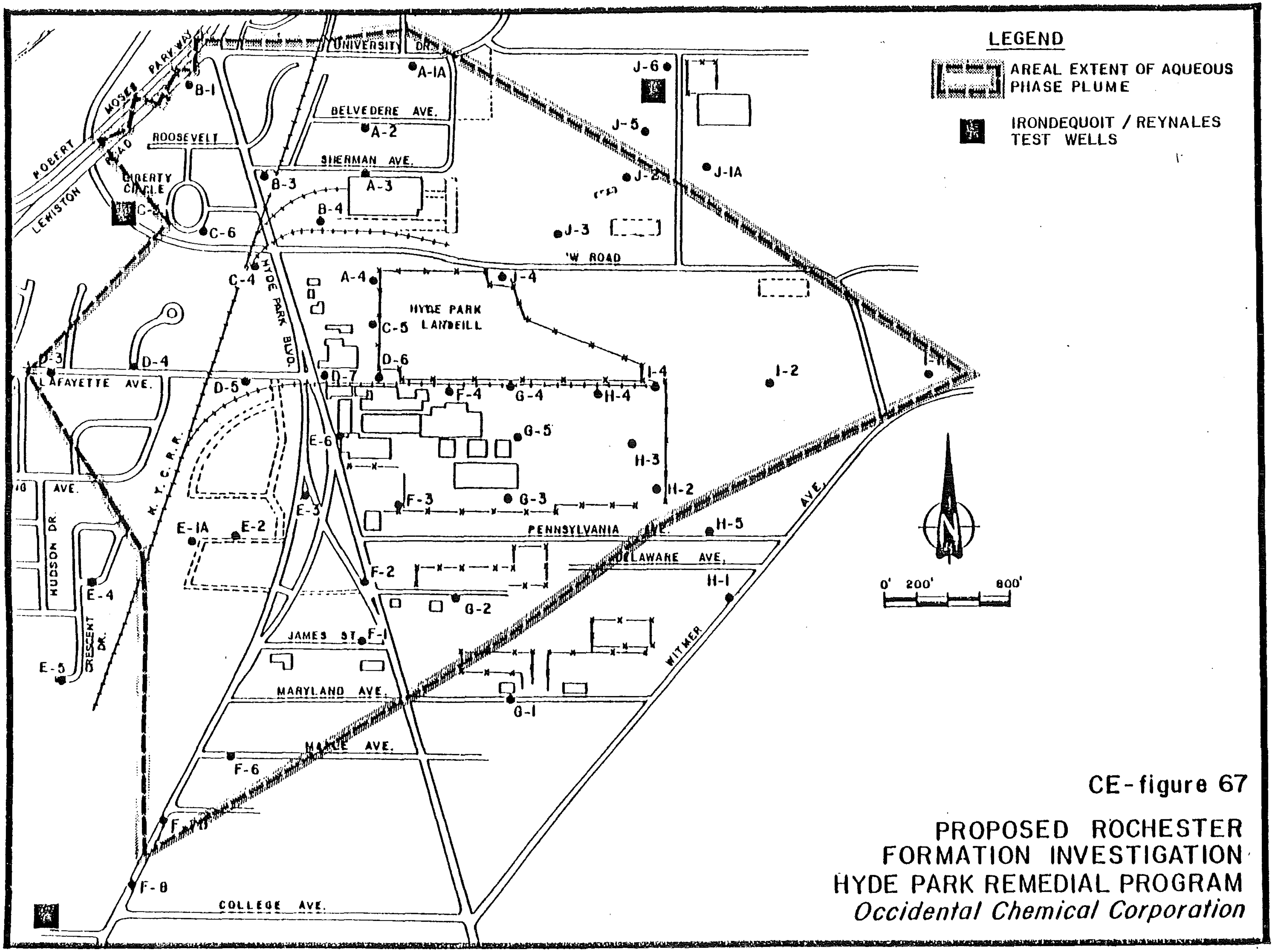

CE-figure 67

PROPOSED ROCHESTER FORMATION INVESTIGATION
HYDE PARK REMEDIAL PROGRAM
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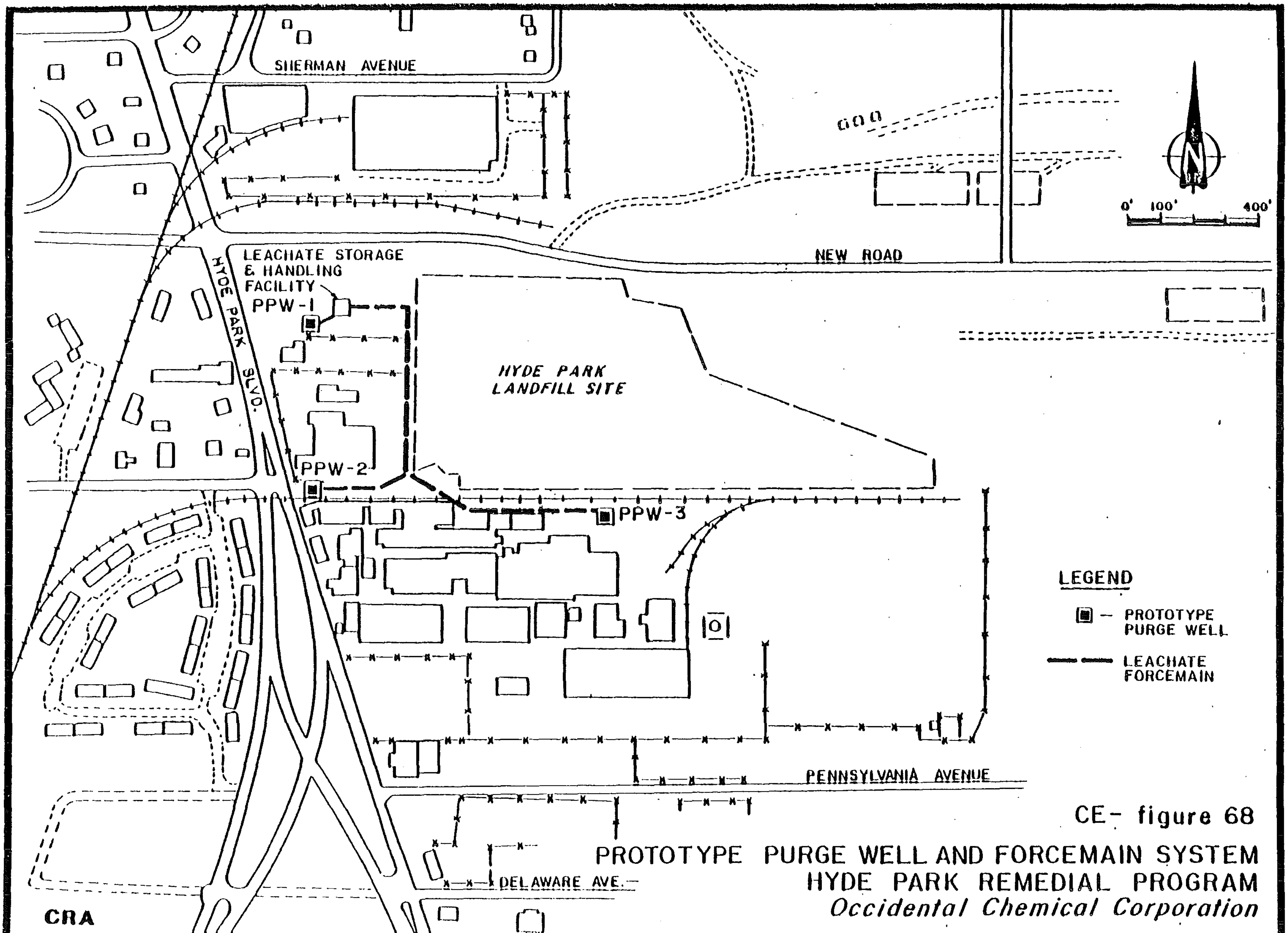

CE- figure 68

PROTOTYPE PURGE WELL AND FORCEMAIN SYSTEM
HYDE PARK REMEDIAL PROGRAM
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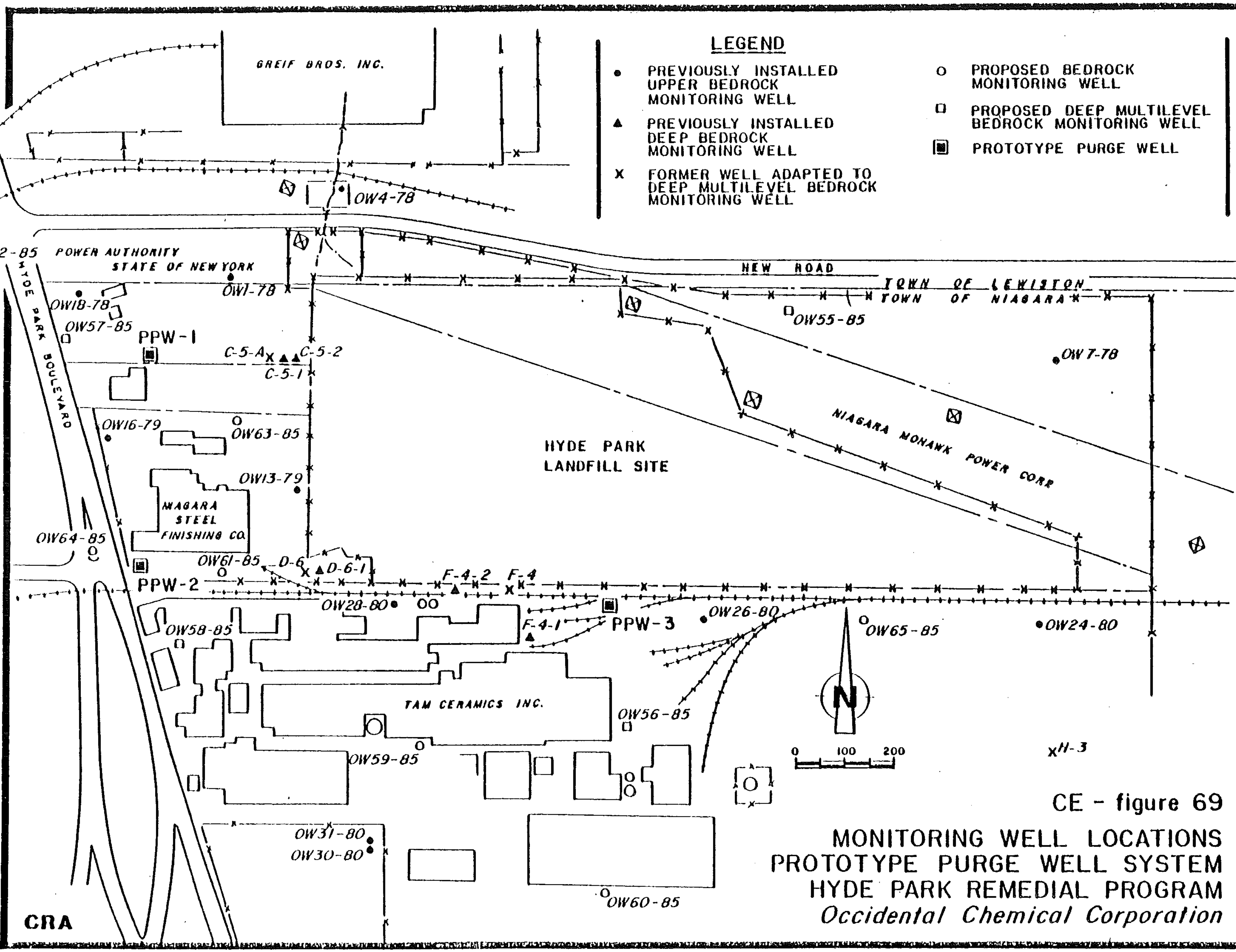

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MONITORING WELL LOCATIONS
PROTOTYPE PURGE WELL SYSTEM
HYDE PARK REMEDIAL PROGRAM
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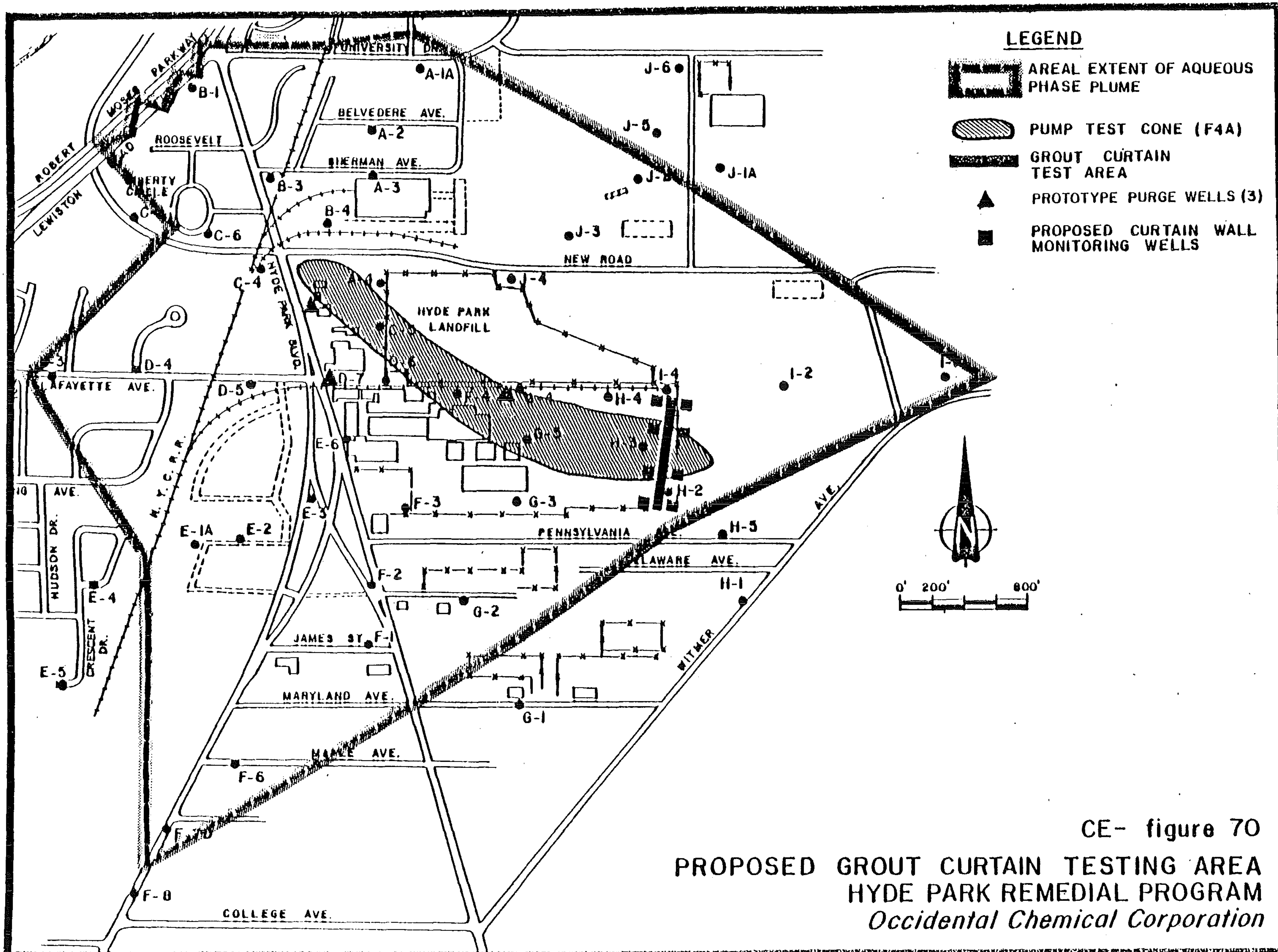

CE- figure 70

PROPOSED GROUT CURTAIN TESTING AREA
HYDE PARK REMEDIAL PROGRAM
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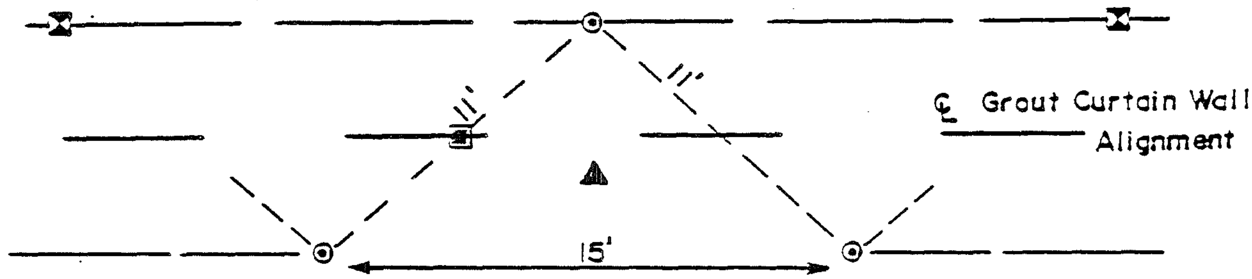
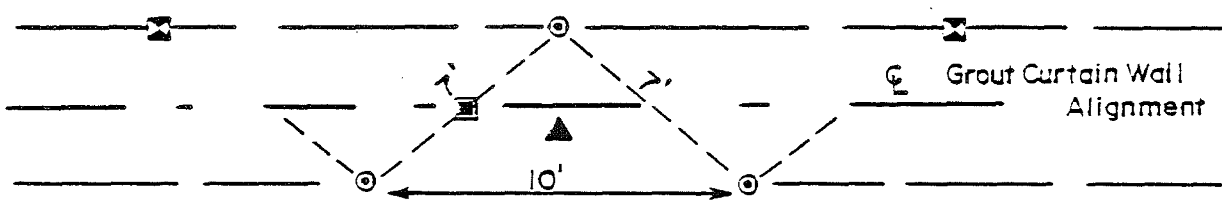
Alternative A



Alternative B



Legend

- Test Plot Grout Hole
- Future Curtain Wall Grout Holes
- Control Borehole
- Auxiliary Grout Hole

CE - figure 71

PROPOSED GROUT TEST PLOTS
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APPENDIX B

PRESSURE GROUT FILLERS & ADMIXTURES

APPENDIX B

PRESSURE GROUT FILLERS & ADMIXTURES

Fillers	Remarks
Clay	- absorbs water - stabilizes cement - not for strength - increases volume - use bentonite for strength
Fine Sands	- durable - does not shrink - evenly graded

Pozzolans	- react with free lime cement
	- natural - finely ground shale
	    - pumicite
	    - diatomite
	- artificial - flyash
	    - pulverized fuel ash
	    - blast-furnace slag
	- cheap bulk fillers

APPENDIX B (Cont'd)

PRESSURE GROUT FILLERS & ADMIXTURES

Fillers	Remarks
Admixtures	modify grout properties many are incompatible with High Alumina and Super sulphated cements (see next table)

APPENDIX B (Cont'd)

PRESSURE GROUT FILLERS & ADMIXTURES

Admixture	Chemical	Remarks
Accelerator	calcium chloride	accelerates set
	sodium silicate	better water cutoff
	sodium aluminate	more costly
Retarder	calcium ligno-sulphonate	increases fluidity
	tartartic acid	slows set time
	sugar	
Air Entrainer	vinsol resin	entrains air
Expander	aluminium powder	expands
	saturated brine	combats shrinkage
pH buffer	disodium phosphate	raises pH
	heptahydrate	should be 7 to 11
Inhibitor	oxygen	slows gel time
	hydroquinone	gives weak gel
	sodium nitrate	
Flexibilizer		increases ability to accept movement
Dye traces	dyes and colors	traces grouting
Anti-bleed	cellulose ether	prevents water bleeding
		entrains air

APPENDIX C

CHEMICAL GROUTS

APPENDIX C

CHEMICAL GROUTS

Chemical Grouts	Components	Remarks
Silicates	sodium silicate formamide accelerator (chemicals or cement)	used in dewatering and stabilization shrinks when dry permeability = 10^{-5}(dry) to 10^{-9}(wet)
Acrylamides	DMAPN ammonium persulfate potassium ferricyanide (inhibitor)	shrinks when dry but will reswell permeability = 10-8 to 10-10 considered ideal grout except neurotoxic
Lignins	lignosulforate hexavalent chromium	fine sands, coarse silts won't mix with cement permeability = 10-7 chromium is toxic
Epoxy Resins	resin catalyst flexibilizer	resist acid, alkalies and organic chemicals fillers reduce cost strengthen fractured rock
Polyester Resins	resin catalyst promoter	shrinks up to 10% strengthen fractured rock

APPENDIX C (Cont'd)

CHEMICAL GROUTS

Chemical Grouts	Components	Remarks
Phenoplasts (polyphenolic resin)	phenol (resorcinal) formaldehyde alkaline base	all components are health hazards poor resistance to wet-dry cycles permeability = 10-9
Aminoplasts	urea formaldehyde	catalyses with limestone sets under acid conditions poor resistance to wet-dry cycles
Polyurethane (foam or gel)	polyisocyonate polyol	cures with water permeability = 10-7 expensive
Acrylates	acrylate monomers methylene bisacrylamide	replaces acrylamide 1 % of toxicity permeability = 5 x 10-9 swells in water resists hydrocarbons shrinks and reswells
Emulsions	bitumen	not effective when hydrocarbons present

APPENDIX D

UNSUITABLE ALTERNATIVE COSTS

The following items have been evaluated and are not being considered for implementation.

1. <u>Excavation of NAPL Contact Materials</u> $482,000,000.00

- drill and blast bedrock.
- excavate and relocate all NAPL contact bedrock and associated overburden material.
- provide dewatering during excavation.
- decontaminate haul trucks prior to leaving site.
- treatment of collected groundwater.
- worker safety and health plan.

The following factors were not included in the above cost estimate.

a) Siting and approval of a new hazardous waste disposal site for the Hyde Park wastes at a suitable nearby location,

b) Monitoring programs for the newly approved site,

c) Waste transport to approved site
   - waste enclosure during transport
   - use of public streets for transport

d) relocation of above and underground utilities.

e) relocation of local transportation routes.

f) Surface water control program
   - within excavation
   - around perimeter,

g) Property purchases

- land
- residences
- industrial facilities
- commercial facilities,

h) Building demolitions

i) Effect of repeated bedrock blasting on surrounding residences and facilities,

j) Air vapor control measures,

k) Air particulate control measures,

l) Safety program to address surrounding neighborhoods,

m) Slope stabilization

- overburden
- bedrock,

n) Backfilling of excavated site and source of backfill material,

o) Rehabilitation of site,

p) Permits and licenses,

q) Legal consultation,

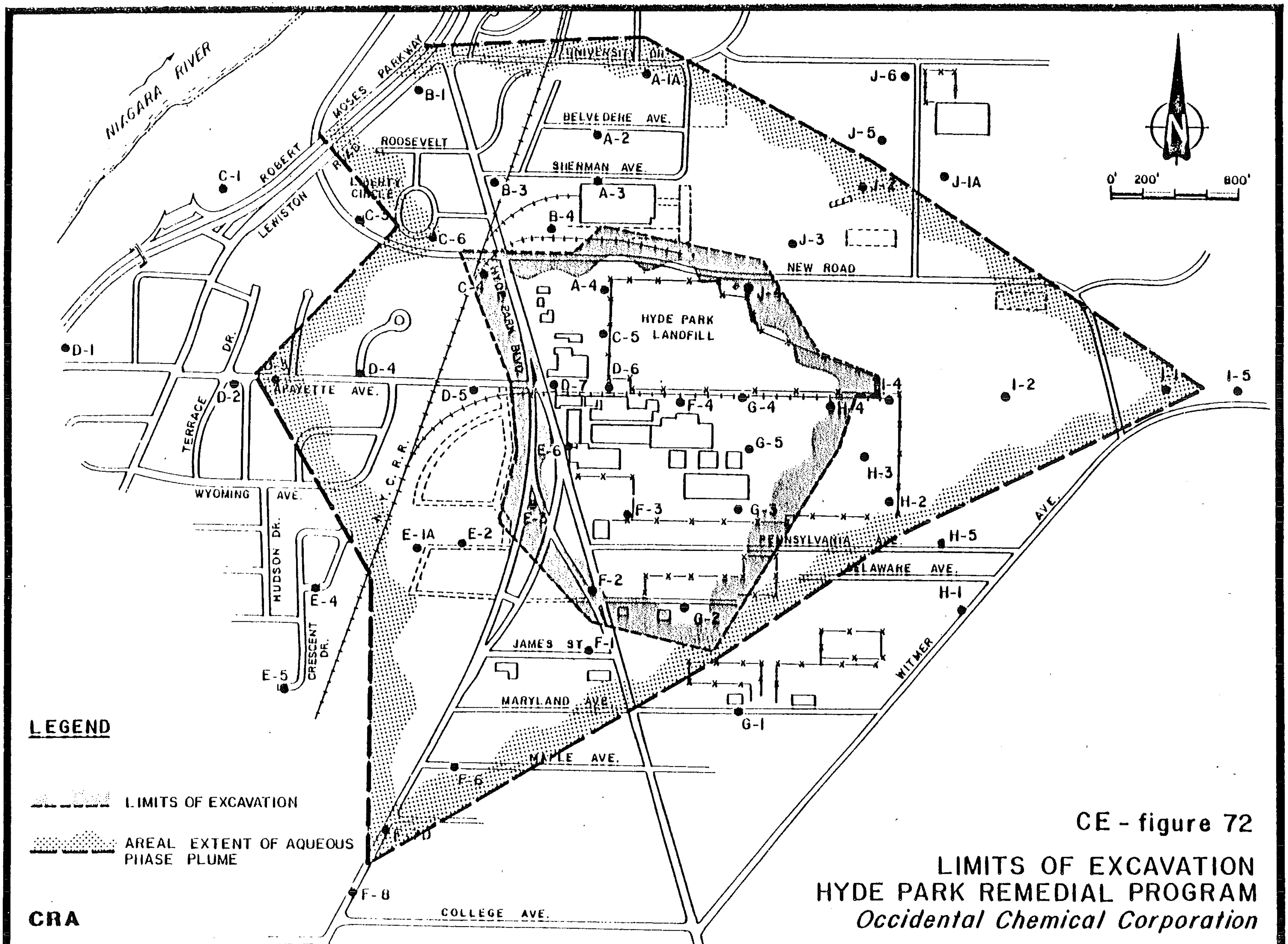
r) Engineering fees,

s) Insurance and bonding.

2. Barrier Walls

a) Overburden (slurry walls) $ 7.00 per S.F.

- excavate trench
- dispose of excavated material.



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LIMITS OF EXCAVATION
HYDE PARK REMEDIAL PROGRAM
Occidental Chemical Corporation

- supply and install slurry backfill.
- provide required worker health and safety program.

b) Bedrock (continuous grout curtain wall)

incremental cost increase over conventional grout curtain wall	$11.00 per S.F.

- drill continuous overlapping boreholes in bedrock including 5 foot key into top of Rochester Formation with 30-inch diameter down-hole-hammer.
- the waste material is to be disposed at the Hyde Park Landfill.
- supply and place designed backfill material
- site restoration.
- provide required worker health and safety.

  (grout curtain wall costs range from $6.00 to $7.80 per square foot

APPENDIX E

ENHANCED OIL RECOVERY PROCESSES

A. THERMAL PROCESSES

Of the three types of recovery methods, thermal processes account for 68% of all the enhanced oil recovery systems. These processes usually apply to reservoirs of low gravity, high viscosity and high porosity. There are three basic types of thermal processes and all three are discussed below.

i) Steam Stimulation

This process consists of injecting 80-90% quality steam into a well for 2 - 3 weeks, shutting in the well for a few days to allow the heat to "soak" the formation and then removing the oil from that same well. The results of this method have often been better than predicted by model studies due to well clean-up and improved viscosity in the vicinity of the well.

The injection of steam increases the temperature of the oil and therefore increases the mobility of heavy oils. Other benefits of this process include thermal expansion of the fluids, compression of solution gas, reduced residual oil saturation and well bore cleanup effects.

The effects of steam stimulation usually decrease for successive treatments and the duration of each cycle becomes shorter.

The steam injection rate does not affect performance and there is a negligible affect due to soak time. Better results have been found with more viscous oils but the effectiveness decreases in thick reservoirs. Smaller steam slugs have been found to be more effective in earlier cycles but there is no net cumulative effect.

ii) <u>Steam Flooding</u>

Steam flooding is a natural follow-up to steam stimulation. It is generally more expensive than steam stimulation but has given better results. The two methods combined have produced 60% (405 MBOD) of the world's enhanced oil recovery production.

In steam flooding, the steam is injected at a number of wells while oil recovery continues at the adjacent wells. The steam forms a saturated heat zone around the injected well. The further the steam moves from the injection well, the lower the temperature. With the drop in temperature, the steam condenses and forms a hot water bank. The oil is displaced by steam distillation in the steam zone and by a number of physical changes in the hot water zone. These changes include thermal expansion of the oil, reduction in viscosity and residual saturation and changes in relative permeability.

The actual results are usually less than expected because the steam moves along the easiest conduit and does not contact all of the oil. Estimating heat losses is essential to designing steam equipment and these can be excessive.

Steam flooding is limited by depth (<5,000 feet) and reservoir thickness (>10 feet).

In comparison with in situ combustion, steam injection is easier to control, uses lower temperatures, can be used on high gravity oils (does not produce sufficient coke for combustion), has a shorter response time and has a lower investment cost. However, in situ combustion uses much less water and considerably less fuel to produce an equivalent volume of oil.

iii) In Situ Combustion

Forward combustion consists of ignition of the oil and then injecting air to push the combustion front and the oil towards the producing wells. The same theory applies to reverse combustion but the air is injected at adjacent wells pushing the oil back through the combustion zone and recovering it from the ignition well. The high temperatures reduce the viscosity of the oil. The only problem with reverse combustion is that when the air is injected at the other wells, the oil may ignite and choke off the air supply thus reverting the process to forward combustion. Lab and field studies indicate that improved results can be attained using wet or partially quenched combustion. In this case, water flashes into superheated steam and when it crosses the combustion front, the oxygen is used in burning. The steam then mixes with nitrogen and carbon dioxide to displace the oil. This process uses one-third the amount of air but requires water.

As fuel prices increase, in situ combustion becomes more economical than steam injection. Larger distances, larger injection rates, high porosity and high oil saturation favor combustion but so far technical success has not been followed by economical success.

B. CHEMICAL PROCESSES

Development of chemical processes has been slow due to high costs, high risks and complicated technology. Only 1% of the enhanced oil recovery in the U.S. has been by chemical processes. Discussed below are the three types of chemical processes: surfactant-polymer injection, polymer flooding and caustic flooding.

i) Surfactant-Polymer Injection

This process attempts to reduce the surface tension between the oil and the reservoir rock. A low concentration surfactant solution dissolves in the oil to reduce interfacial tension while a high concentration solution displaces the oil. As it moves, the high concentration solution is diluted and the process reverts to a low concentration flood.

The process consists of injecting a surfactant slug to displace the oil and create an oil bank with a greater mobility than the surfactant. A polymer mobility buffer is then injected which is less mobile than both the surfactant and the oil and this controls mobility. A loss of mobility results in fingering and reduced displacement.

Slug breakdown can occur due to adsorption of the surfactants. The higher equivalent weight sulfonates which are mainly responsible for lowering interfacial tension are also adsorbed more easily. A pre-flush of inorganic salts (i.e. sodium silicate) will reduce adsorption.

Surfactant-polymer injection has good potential but more field testing needs to be done to overcome the continual chemical reactions and the economics.

ii) Polymer Flooding

Polymer flooding involves adding a thickening agent to water to increase its' viscosity. Polymers also increase the sweep efficiency (i.e. the volume of oil contacted by the injected fluid).

Mobility is measured as permeability over viscosity and the mobility ratio is the ratio of the mobility of water to the mobility of the oil. Polymers increase the viscosity of water instead of reducing the viscosity of oil to effect displacement.

This process is not effective for all reservoirs. A highly fractured, porous media is not capable of providing effective response and the existing mobility ratio should be between 2 and 20. The process is too costly with higher mobility ratios and not effective with lower ones.

The two basic types of polymers are polysaccharides and polyacrylamides, both of which require much more lab testing. One problem is the shearing effect which causes degradation of the polymer chains.

iii) Caustic Flooding

Caustic flooding involves the injection of an alkali in water with a pH of 12-13 to improve recovery. This usually involves the addition of 1-5% sodium hydroxide (NaOH) to an existing waterflood. Few tests have been done and results have not been encouraging. There is no record of successful field applications.

For caustic flooding, reservoirs must be susceptible to waterflooding and there must be low interfacial tension between the oil and the caustic solution. Oil viscosity should be less than 200 cp or thermal processes would be more effective. Injection in limestone or dolomite with no extensive fractures has shown promise since it is unreactive.

A major problem of this method is the dissolving of gypsum by the solution. This gypsum is deposited near the well bores and plugs up the production wells.

C. MISCIBLE DISPLACEMENT PROCESSES

These processes involve the introduction of a fluid to dissolve oil and thus eliminate oil retention forces in rock. The injection usually involves a slug of solvent followed by liquid or gas. Twenty-nine percent of the world's enhanced oil recovery has been through these processes.

i) Miscible Hydrocarbon Displacement

The three basic types of miscible hydrocarbon displacement are: i) the miscible slug process which involves injection of liquid hydrocarbons followed by natural gas. ii) the enriched gas process which uses enriched natural gas followed by lean gas.

iii) the high pressure lean gas process which uses lean gas at high pressures.

All of the above processes require high pressures so depths must be anywhere from 2,000 feet to more than 5,000 feet.

Injecting water alternatively with gas helps to reduce the mobility ratio and therefore improve efficiency. The enriched gas process is becoming more popular due to the high cost of liquid propane gas although lean gas prices are not all that more economical.

ii) Carbon Dioxide Injection

This process is similar to the high pressure lean gas process. Carbon dioxide and oil are not initially miscible but pressure and temperature changes can affect this relationship. The injection of carbon dioxide swells the oil so that viscosity is reduced.

A simultaneous injection of CO_2 and water or alternating injections have given better results than just carbon dioxide, however, the combination of CO_2 and water is very corrosive. Water injections help to reduce fingering and in this way, help sweep efficiency.

Carbon dioxide's big advantage is that it is neither hazardous nor explosive.

The full potential is not known but studies say CO_2 flooding could recover 40-50% of the oil reserves.

iii) Inert Gas Injection

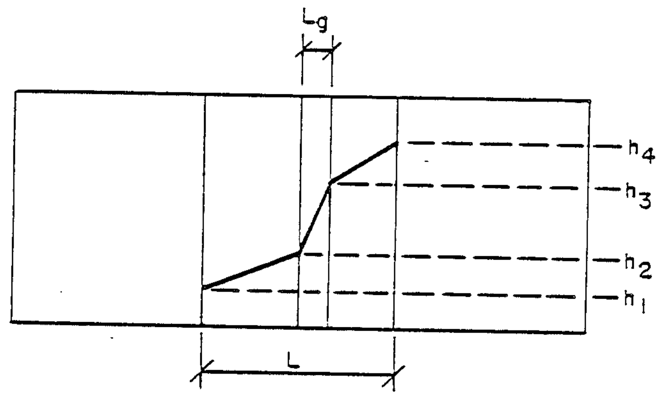
A narrow range of conditions involving pressure, temperature and fluid composition has resulted in few reservoirs being candidates for inert gas injection. Considerable depths are required due to high pressures and repeated contacts with the oil must be made to provide miscible displacement. The main disadvantage is the corrosiveness of inert gas.

Under ideal conditions with sufficiently high temperatures all the oil can essentially be recovered. Inert gas projects have been on the increase due to lower costs.

APPENDIX F

CALCULATION OF HARMONIC MEAN
HYDRAULIC CONDUCTIVITY FOR
ELEMENTS WITH GROUT CURTAIN WALL

Schematic A shows three elements, with the middle element containing a grout curtain wall with lower hydraulic conductivity than the porous media.



The head loss profile is indicated in Schematic A illustrating that for one dimensional flow and a constant q, a larger head loss occurs through the grout curtain wall.

For constant q the following can be stated:

$$q = K_{ave}\left(\frac{h_4-h_1}{L}\right) = K_g\left(\frac{h_3-h_2}{L_g}\right) = K_L\left(\frac{h_4-h_3}{L_R}\right) = K_L\left(\frac{h_2-h_1}{L_L}\right) \qquad \text{(A-1)}$$

where: q = Darcy flux (L/T)

L = length of element (L)

L_g = thickness of grout curtain (L)

L_R = thickness of element to the right of the grout curtain (L)

L_L = thickness of element to the left of the grout curtain (L)

K_g = hydraulic conductivity of grout curtain (L/T)

K_L = hydraulic conductivity of porous media (L/T)

Rearranging terms in Equation (A-1) give

$$q \frac{L}{K_{ave}} = h_4 - h_1 \qquad \text{(A-2)}$$

$$q \frac{L_g}{K_g} = h_3 - h_2 \qquad \text{(A-3)}$$

$$q \frac{L_R}{K_L} = h_4 - h_3 \qquad \text{(A-4)}$$

$$q \frac{L_L}{K_L} = h_2 - h_1 \qquad \text{(A-5)}$$

Therefore

$$q \frac{L_g}{K_g} + q \frac{L_R}{K_L} + q \frac{L_L}{K_L} = q \frac{L}{K_{ave}} \qquad \text{(A-6)}$$

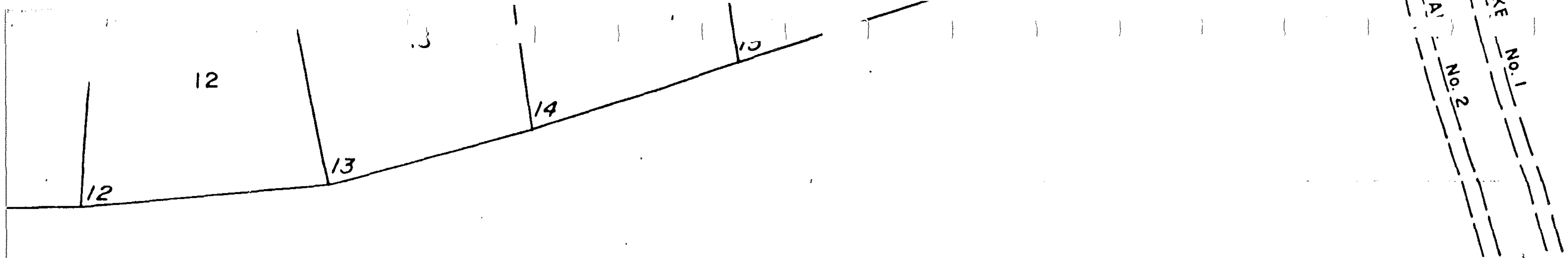
Deleting q, Equation (A-6) is rewritten as

$$\frac{L}{K_{ave}} = \frac{L_g}{K_g} + \frac{L_R}{K_L} + \frac{L_L}{K_L} \qquad \text{(A-7)}$$

Since $L_R + L_L = L - Lg$

$$\frac{L}{K_{ave}} = \frac{L_g}{K_g} + \frac{L - L_g}{K_L} \qquad \text{(A-8)}$$

Equation (A-8) was employed to calculate the harmonic mean hydraulic conductivity for those elements which contained a grout curtain wall.



LEGEND

13 ELEMENT DESIGNATION

13 NODE DESIGNATION

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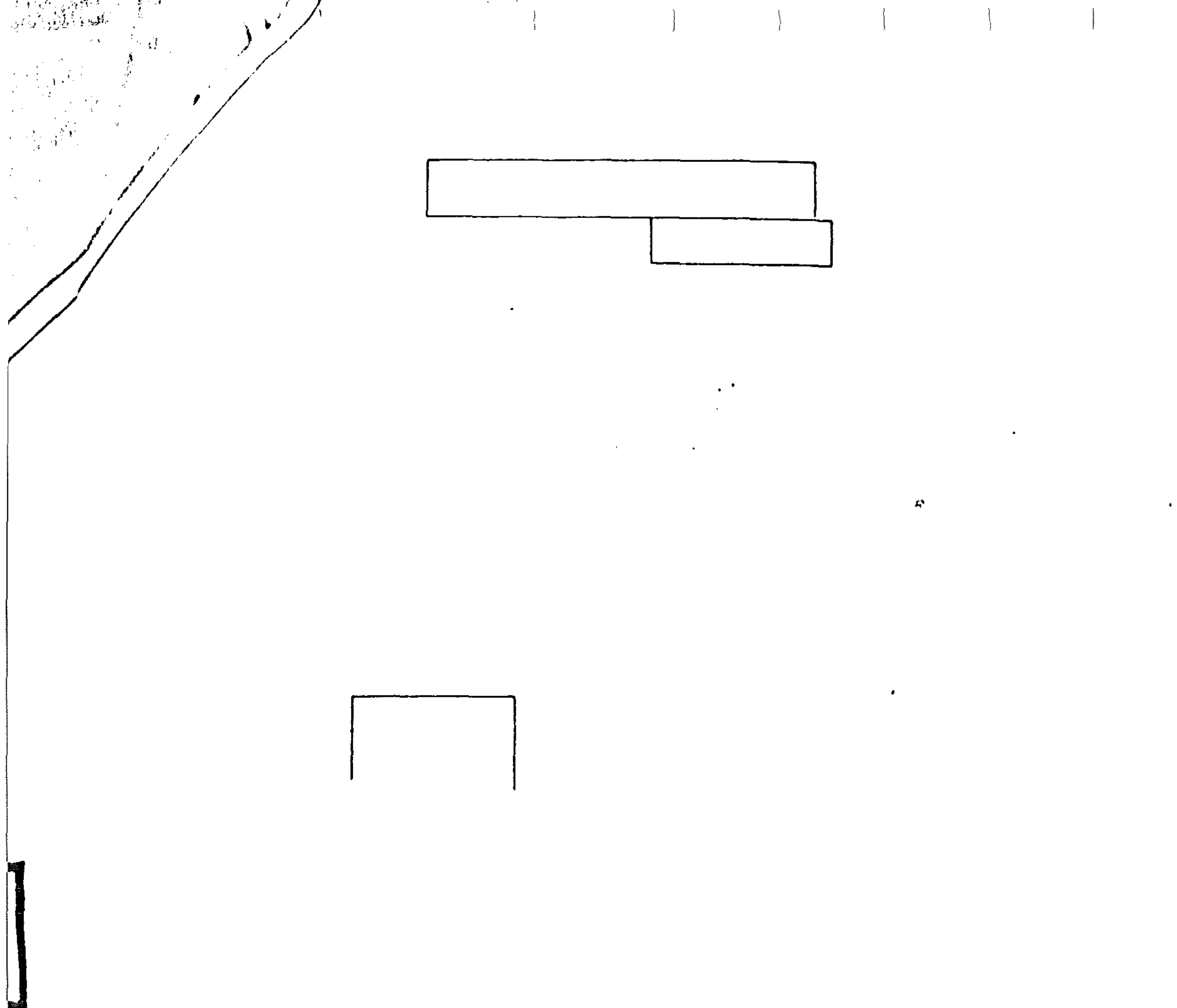
FINITE ELEMENT GRID
PLAN VIEW

CRA Consulting Engineers
CONESTOGA-ROVERS & ASSOCIATES LIMITE
651 Colby Drive, Waterloo, Ontario Canada N2V 1C2

Drawn by BYO	Scale 1" = 500'	Date MARCH 27 ,1984	File Nº OCC E-6	Rev 0
Designed by K.D.S.	Field book	Project Nº 1069	Drawing Nº 37 MAP 2	
Checked by: K.D.S.				



BEDROCK GROUNDWATER MONITORING WELL

AQUIFER SURVEY WELL

NON AQUEOUS PHASE LIQUID PLUME (NAPL)

AQUEOUS PHASE LIQUID PLUME (APL)

560000 GAL.
49000 GAL.
PLUME VOLUME (U.S. GALLONS)

HYDE PARK REMEDIAL PROGRAM
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PLUME VOLUMES

CRA Consulting Engineers
CONESTOGA-ROVERS & ASSOCIATES LIMITED
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Designed by: D B	Field book:	Project Nº: 1069	Drawing Nº: 42 MAP 1	
Checked by: J K				

LEGEND

297 ELEMENT DESIGNATION

330 NODE DESIGNATION

(A) BOUNDARY DESIGNATION

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AQUEOUS PHASE
FINITE ELEMENT GRID
SECTION AA - AA'

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651 Colby Drive, Waterloo, Ontario Canada N2V 1C2

Drawn by: ZP	Scale HOR: 1" = 200' VER: 1" = 20'	Date: MARCH 27, 1984	File Nº CCC E-6	Rev.
Designed by KDS	Field book	Project Nº: 1069	Drawing Nº 38 MAP 3	
Checked by KDS				

221

D

ROCHESTER FORMATION

LEGEND

ELEMENT DESIGNATION

NODE DESIGNATION

BOUNDARY DESIGNATION

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HYDE PARK REMEDIAL PROGRAM
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TWO PHASE
FINITE ELEMENT GRID
SECTION BB - BB'

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Drawn by ZP	Scale HOR: 1" = 200' VER: 1" = 20'		Date: MARCH 27, 1984	File Nº OCC E-6	Rev N 0
Designed by KDS	Field book	Project Nº: 1069	Drawing Nº 39 MAP 4		
Checked by KDS					

LOCKPORT FORMATION JOINTING AT THE HYDE PARK LANDFILL SITE

Prepared for:

WHITEMAN OSTERMAN & HANNA

Prepared by:

WOODWARD-CLYDE CONSULTANTS
Plymouth Meeting, Pennsylvania

29 March 1983

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LOCKPORT FORMATION JOINTING AT THE HYDE PARK LANDFILL SITE

INTRODUCTION

Groundwater flow in fractured rock systems is often controlled by structural features within the rock unit comprising the aquifer. In order to assess the possibility of such structural control in the vicinity of the Hyde Park landfill site, a study was undertaken which involved remote sensing work as well as field work in the site area. This report presents the results of that work.

An aerial photography review of the Niagara Falls area has shown that there is a consistent lineament pattern expressed in the photography. It is believed that this lineament pattern results from geological conditions in the area, primarily the fracture network in the bedrock formation. In order to detail the aerial photography work and to assess the nature of the geologic controls in the area, a geologic field survey was conducted. This survey was restricted to the Hyde Park disposal site area. The location of the Hyde Park site with respect to the general Niagara Falls area is shown in Figure 1.

The field survey involved a detailed study of the bedrock cores taken from beneath the Hyde Park site, an inspection of the face of the Niagara Gorge and field measurements of the bedrock fracture network exposed at the face of the gorge. The groundwater hydrology of the site area was also reviewed with respect to the geologic controls. The results of the field work and subsequent assessment are presented in this document. This work was conducted in order to gain a broader insight into the potential movement patterns of contaminated groundwater.

BEDROCK GEOLOGY

The Hyde Park site is underlain by the Lockport Formation of Middle Silurian Age (see Figure 2). As described in Zenger 1965, the Lockport is principally a dolomite and as a formation is characterized by a brown-gray color, medium granularity, medium to thick bedding, carbonaceous partings (laminae), vugs and stylolites (thin irregular suture-like structure). The approximately 200-foot section of Lockport in the Niagara Falls vinicity is divided into five members which will be discussed below. Regionally, the bedrock in the Niagara Falls area strikes near east-west and dips to the south at approximately 30 feet per mile. Test borings indicate the depth to bedrock in the Hyde Park site area ranges from approximately 5 to 36 feet.

STRATIGRAPHY

As shown in Figure 3, the Lockport Formation is divided into five members. They are from oldest to youngest as follows: (1) DeCew, (2) Gasport, (3) Goat Island, (4) Eramosa and (5) Oak Orchard. Over half the thickness of the Lockport consists of the Oak Orchard Member. The portion of the Lockport Formation which will be dealt with specifically in this discussion of the Hyde Park landfill site includes the Oak Orchard and Eramosa members.

Eramosa Member: As reported in Zenger, the Eramosa is typically a medium to dark gray, fine-grained, thin to medium bedded, argillaceous (containing clay) and bituminous (containing carbonaceous matter) dolomite. This member is characterized by carbonaceous shale partings and locally having mineralized vugs containing calcite, gypsum, sphalerite and galena while also being pyritic in part. The thickness of the Eramosa in the Niagara Falls area, as reported in Zenger, is from 16 to 18 feet. Rock was encountered in test borings (e.g. OW 1-78 and OW 1-79) conforming to the above description of the Eramosa member.

Oak Orchard Member: The description of the Oak Orchard member as presented in Zenger is summarized as follows. This member is a brown-gray to dark gray, fine to medium grained, thin to mostly thick bedded, saccaroidal (well developed, uniform size crystals), bituminous dolomite. As with the underlying Eramosa member, the Oak Orchard has carbonaceous partings, vugs and stylolites. The exact thickness of the Oak Orchard in the Niagara Falls area is not known, but is estimated to be at least 120 feet. Therefore, the section of Oak Orchard at the Hyde Park site is most likely confined to the middle and lower portions of this member since beds resembling the underlying Eramosa member were encountered in the test borings.

Nature of the Oak Orchard–Eramosa Contact: The contact between the Oak Orchard and Eramosa members is conformable (no erosional discontinuities) and is described as sharp by Zenger. Further discussion of this contact by Zenger is as follows: "...the light-weathering, thin to medium bedded Eramosa is overlain by brown, massive, bituminous, very vuggy dolomite that is the basal Oak Orchard. Limonitic stains at this contact probably are not significant. The writer attributes this coloration to groundwater that, having dissolved iron bearing minerals (e.g. pyrite) as it migrated downward through the overlying vuggy stratum, subsequently deposited them on issuing forth at the contact with the less permeable Eramosa."

REMOTE SENSING STUDY

The bedrock formations in the Niagara Falls region gently dip to the south forming what is referred to as a homocline. The most prevalent structural element in the bedrock is a regional joint (fracture) system which manifests itself in the form of features such as lineaments (regional scale, linear features) as well as relatively small scale planar surfaces within the bedrock. The presence of well developed jointing in the Lockport Formation can significantly influence the movement of groundwater within the bedrock. Areas of faulting are briefly described by Zenger, but no geologic faults were mentioned that would unfavorably impact this study.

The assessment of the degree of jointing, orientation of joints and the regional versus local nature of the joint structure was initially performed by examination of air photography. Three types of photo-imagery were reviewed: Landsat images, high altitude photographs, and low altitude photographs.

LANDSAT IMAGES

A computer printout of all available Landsat images for the Niagara Falls area with less than 10% cloud cover and good quality (5 or better on the EROS quality scale) was obtained from the U.S. Geological Survey EROS Data Center. The images most revealing of geologic structure were ordered to be printed at 1:250,000 scale for further examination.

These images showed the major rock fracture pattern within about 50 miles of the Hyde Park site. These images did not show fractures passing through the Hyde Park site, but a regional fracture pattern was definitely defined. Regional lineament trends in order of most to least prominent were noted as follows:

1) N 50° E
2) N 45° W
3) E - W
4) N - S

The nearest Landsat lineament to the site that was noted was an indistinct N 50° E lineament partially formed by the Niagara River bank to the west of the Hyde Park site. This lineament passes a few hundred yards west of the Hyde Park site.

HIGH-ALTITUDE PHOTOGRAPHS

A computer print-out of all available high-altitude photographs for the Niagara Falls area with less than 10% cloud cover and good quality (7 or better on the EROS quality scale) was obtained from the U.S. Geological Survey. These high-altitude photographs were examined at the EROS Data Center, and high-altitude photographs most revealing of geologic structure were ordered.

The most prominent lineament on high-altitude photographs is formed by the Niagara escarpment a few miles north of the disposal site. The escarpment strikes about N 75° E.

The lineament nearest the site on the high-altitude photographs strikes N 50° E along the Niagara River west of the site. This is the same lineament as noted on the Landsat imagery. Other prominent lineaments trending N 50° E intersect the escarpment to the northeast of the site. There are also N-S striking lineaments along the Niagara River. These are north of the site, and, if extended, would pass a few hundred yards west of the disposal site. Several lineaments, exemplified by the one at the Whirlpool, strike N 45° W. There is also a short E-W lineament, along the Niagara River south of the city of Niagara Falls.

LOW-ALTITUDE PHOTOGRAPHS

Photographs examined included several black and white aerial photographs taken of the site area every few years by the U.S. Department of Agriculture, standard black and white aerial photographs taken by a local aerial photographic company, and color infrared (non-thermal) aerial photographs taken by the U.S. Environmental Protection Agency. This photography covers a period of time from 1938 to 1979.

Of all lineaments related to the Hyde Park site, the most significant seems to be a N 50° W striking lineament (or lineament set) partially formed by the valley of the small stream (Bloody Run) which enters the Niagara River gorge northwest of the disposal site. This lineament, visible on the 1975 photograph and also the Lewiston, Ont. - N.Y. 7-½' Quadrangle Map, appears to extend southeastward through the west-central part of the disposal site. Bedrock fractures probably control this N 50° W lineament. A fracture in this location could allow a significant amount of downward movement of contaminants migrating from the Hyde Park site.

On the 1979 color IR photographs there is off-color vegetation just west of the Robert Moses Parkway along the stream which forms the above mentioned N 50° W lineament. The off-color vegetation may be due to stress caused by the presence of chemicals, or may be a result of some other activity. A field study oriented toward determining the nature of the stressed vegetation would need to be done to establish the cause of the impact on the vegetation.

The 1938 photograph shows prominent N 50° W lineaments north and SE of the Hyde Park disposal site. In some cases these lineaments are streams. Other lineaments on this photograph strike N 45° E and N-S.

REMOTE SENSING CONCLUSIONS

The overall remote sensing study revealed a significant bedrock joint pattern in the vicinity of the Hyde Park site. A map showing the lineament pattern, as interpreted from air photography, is shown as Figure 4. In order to verify that these lineaments are due primarily to bedrock jointing, a field assessment was conducted.

FIELD ASSESSMENT

A site visit was made to verify the conditions noted in the remote sensing study. During this site visit the bedrock exposures along the face of the Niagara Gorge near the landfill site, were examined. Also, the bedrock cores taken from the holes drilled during installation of the monitoring wells were reviewed.

Bedrock cores from monitoring wells OW-1, 4, 7, 11, 13, 16, 17, 18, 20 and 22 were logged by WCC. In addition to lithology and stratigraphy, particular attention was directed toward fractures/joints. There was little evidence to suggest any significant amount of fractured rock occurring at the overburden interface, thus indicating sound, intact surface rock conditions. The rock cores tend to split along the nearly horizontal shaly partings which made RQD (Rock Quality Designation) determination somewhat difficult in selected core runs. Overall, however, RQD values are rated as good (75-90%) to excellent (90-100%). In Table 1 a summary of RQD values is presented. Occurring throughout the rock cores examined there are occasional low angle (less than 45 degrees) to high angle (greater than 45 degrees) joints or fractures. These planar surfaces are often oxidized but usually have a "tight" or closed interface.

Excellent exposures of the lower members of the Lockport Formation and underlying Clinton Group (including the Rochester Shale) are present along the south haul road of the Niagara Power Plant. The DeCew, Gasport, Goat Island and Eramosa members of the Lockport are exposed in a 70-foot vertical section in excavated rock along the east bank of the Niagara Gorge located approximately 2,000 feet northwest of the Hyde Park site. These exposures were visited due to their proximity to the site and pertinent geologic data were collected.

Jointing was particularly well developed in the Eramosa and Goat Island members and generally was less frequent in the underlying Gasport and DeCew members and Rochester Shale. A plot of the strike of jointing is presented in Figure 5. There is a relatively strong grouping of strikes to the northwest, north and northeast. These directions are consistent with the regional lineament pattern shown on the landsat imagery. Figure 5 is a plot of the observed range in the joint dips. It is immediately evident that the majority of joint dips are between 80 and 90 degrees and could be described as near vertical to vertical.

The joints are well expressed in outcrop. Individual joints are commonly spaced from six inches to two feet. The length of joints ranged from less than one foot to in excess of 15 to 20 feet. Two large fissure-like "fractures" were observed along the haul road. They are near vertical and strike approximately east-west and are expressed as an opening from less than one inch to approximately one foot wide. Other joint related features observed along the haul road include a prominant notch in the gorge face northwest of the Hyde Park site. This topographic feature is apparently a result of rock dislodged along intersecting northeast and northwest striking joint sets.

CONCLUSIONS

The lineament patterns revealed during the remote sensing study have been verified as joints (or fractures) in the Lockport Dolomite by the field assessment. Near the Hyde Park site the main fracture trend is N 45° W. The second most predominant set of fractures is oriented N 45° E and the least predominant fracture set trended N-S. A small number of fractures were noted to trend E-W, one fracture of which was fairly significant in size, but this trend was minor.

The fracture sets are essentially all vertical or near vertical, with dips ranging between 80° and 90°. Some bedding joints may be present due to separations along shaly partings that occur along the bedding planes. These bedding planes, however, are believed to be relatively tight.

The major route for bedrock groundwater movement and contaminant movement from the Hyde Park landfill site would appear to be the fracture pattern in the bedrock. The regional southerly bedrock dip may also have some impact on the movement of contaminants, but this impact may be minor because in the rock cores that were examined the bedding planes appeared to be relatively tight. Contaminant movement along the N 45° E fracture network toward the south should not be confused with contaminant movement southerly along the bedrock bedding planes.

A more detailed assessment of the impact of the fracture patterns in the bedrock on contaminant movement can be obtained by drilling angle holes. These holes should be drilled at 45° to the horizontal and should be oriented to intersect the major fracture traces in the site area, specifically the N 45° E and the N 45° W trending fractures. The location of these borings should be picked consistent with the observe fracture patterns at the site and the observed contaminant conditions.

TABLE 1
ROCK QUALITY DESIGNATION
HYDE PARK SITE BEDROCK CORES

BORING NO.	CORE RUN NO.	DEPTH FT.	ROCK QUALITY DESIGNATION (RQD)
OW-1	R-1	36.5 - 41.5	100%
	R-2	41.5 - 46.5	100%
	R-3	46.5 - 50.2	100%
OW-4	R-1	28.5 - 33.5	92%
	R-2	33.5 - 38.5	98%
	R-3	38.5 - 43.5	100%
OW-7	R-1	5.8 - 10.8	96%
	R-2	10.8 - 15.8	76%
	R-3	15.8 - 20.8	96%
OW-11	R-1	4.9 - 5.9	0%
	R-2	6.8 - 10.8	52%
	R-3	10.8 - 16.2	64%
	R-4	16.2 - 21.5	80%
	R-5	21.5 - 27.1	40%
OW-13	R-1	16.2 - 21.2	100%
	R-2	21.2 - 25.2	63%
	R-3	25.2 - *	—
	R-4	* - *	—
OW-16	R-1	31.5 - 36.5	70%
	R-2	36.5 - 41.5	72%
	R-3	41.5 - 46.5	90%
	R-4	46.5 - 51.5	88%
OW-17	R-1 (boulder)	27.7 - 29.3	75%

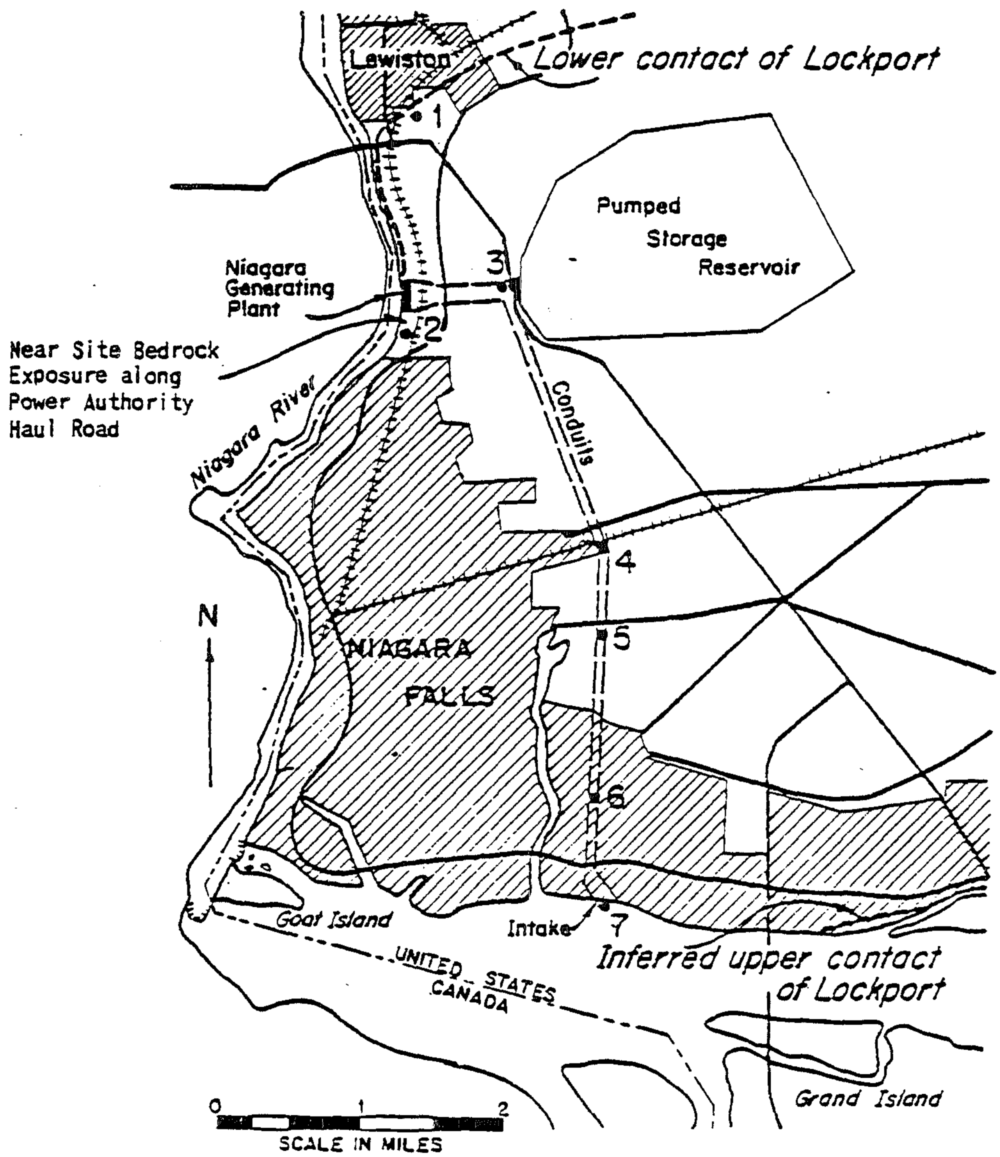
TABLES

BORING NO.	CORE RUN NO.	DEPTH FT.	ROCK QUALI DESIGNATIO (RQD)
OW-18	R-1	(35.6 - 40.6)	100%
	R-2	(40.6 - 45.6)	94%
	R-3	(45.6 - 50.6)	88%
	R-4	(50.6 - 55.6)	80%
	R-5	(55.6 - 60.6)	94%
	R-6	(60.6 - 65.6)	92%
	R-7	(65.6 - 67.6)	100%
	R-8	(67.6 - 72.6)	96%
	R-9	(72.6 - 77.6)	100%
	R-10	(77.6 - 82.6)	100%
	R-11	(82.6 - 85.5)	100%
OW-20	R-1	(25.4 - 30.4)	98%
	R-2	(30.4 - 35.4)	80%
	R-3	(35.4 - 40.4)	94%
	R-4	(40.4 - 45.4)	98%
OW-22	R-1	(23.3 - 33.3)	89%
	R-2	(33.3 - 43.3)	100%
	R-3	(43.3 - 49.0)	100%

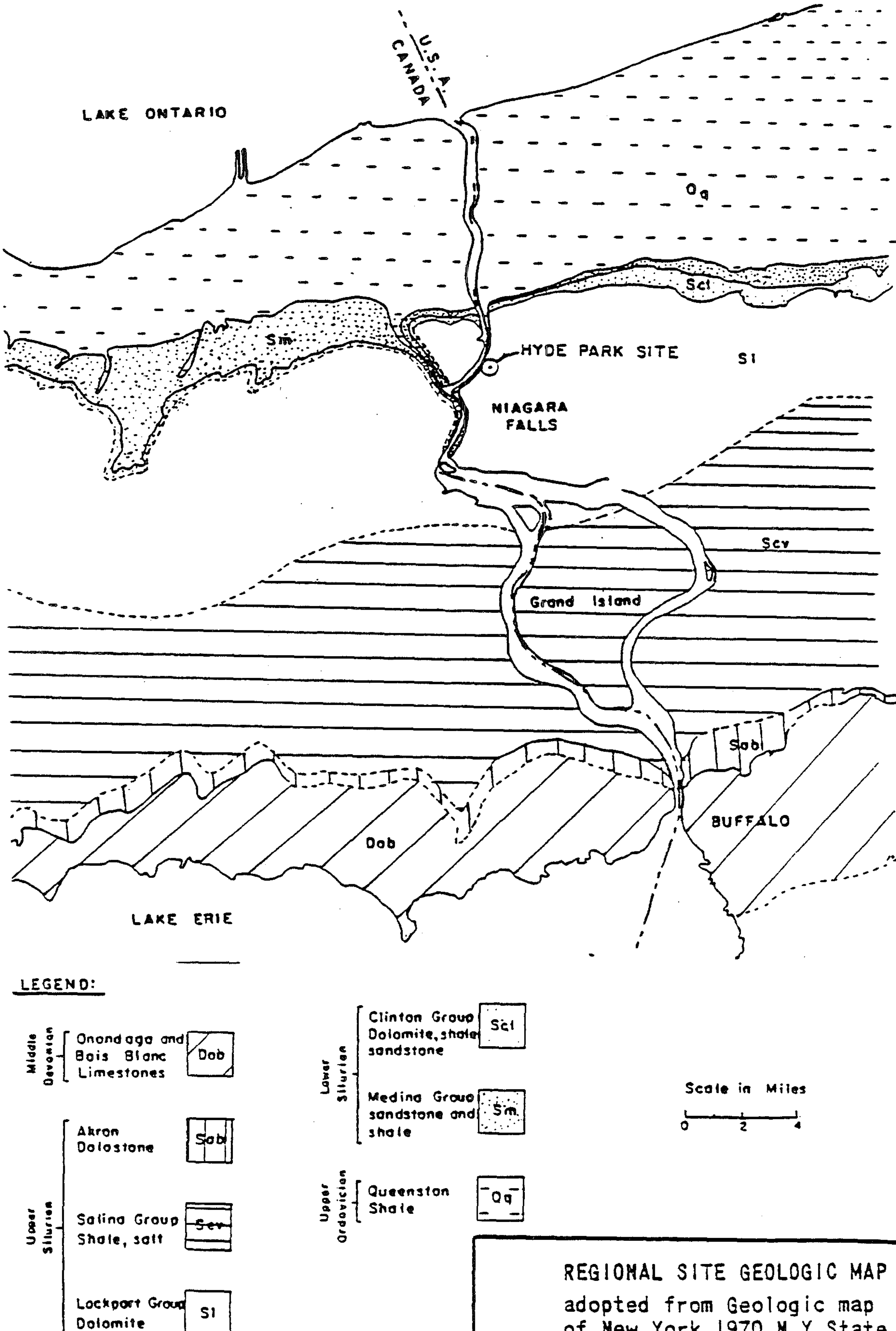
*Depths not recorded on core boxes

( ) Estimated depths, therefore, RQD values might vary slightly

PLATES



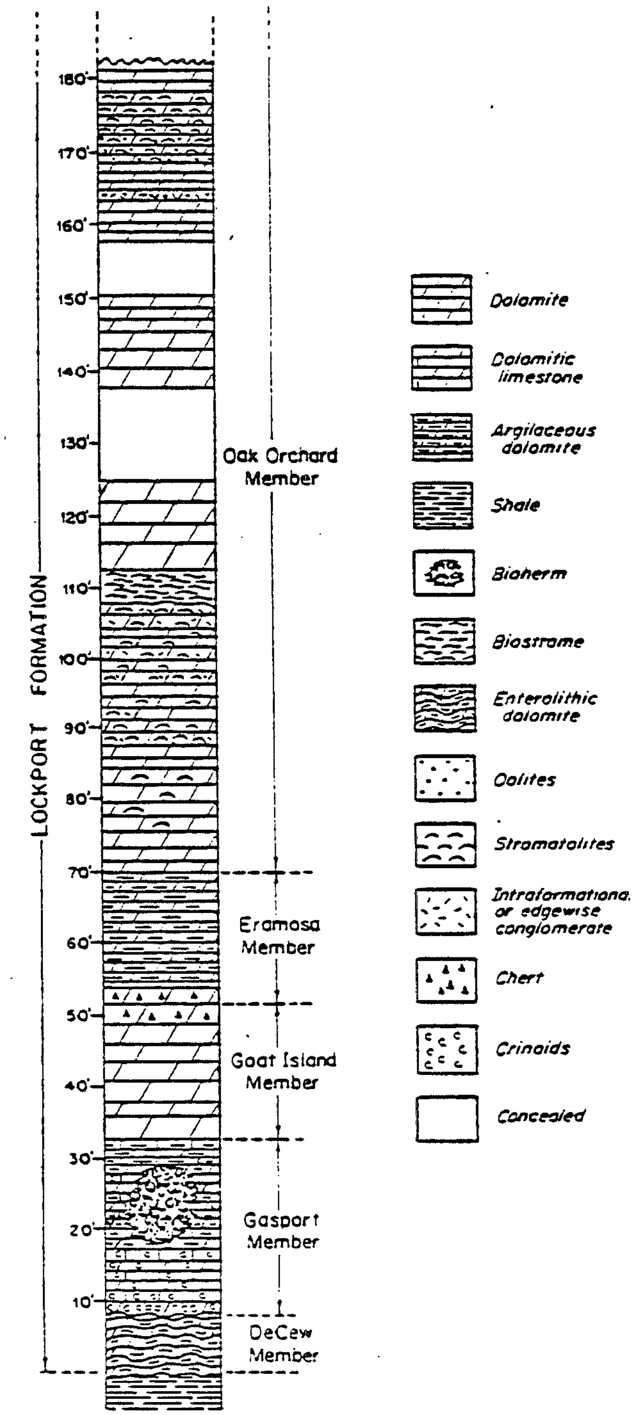
SITE LOCATION AND GENERAL GEOLOGY
after N.Y.State Museum and Science
service bulletin no. 404



REGIONAL SITE GEOLOGIC MAP

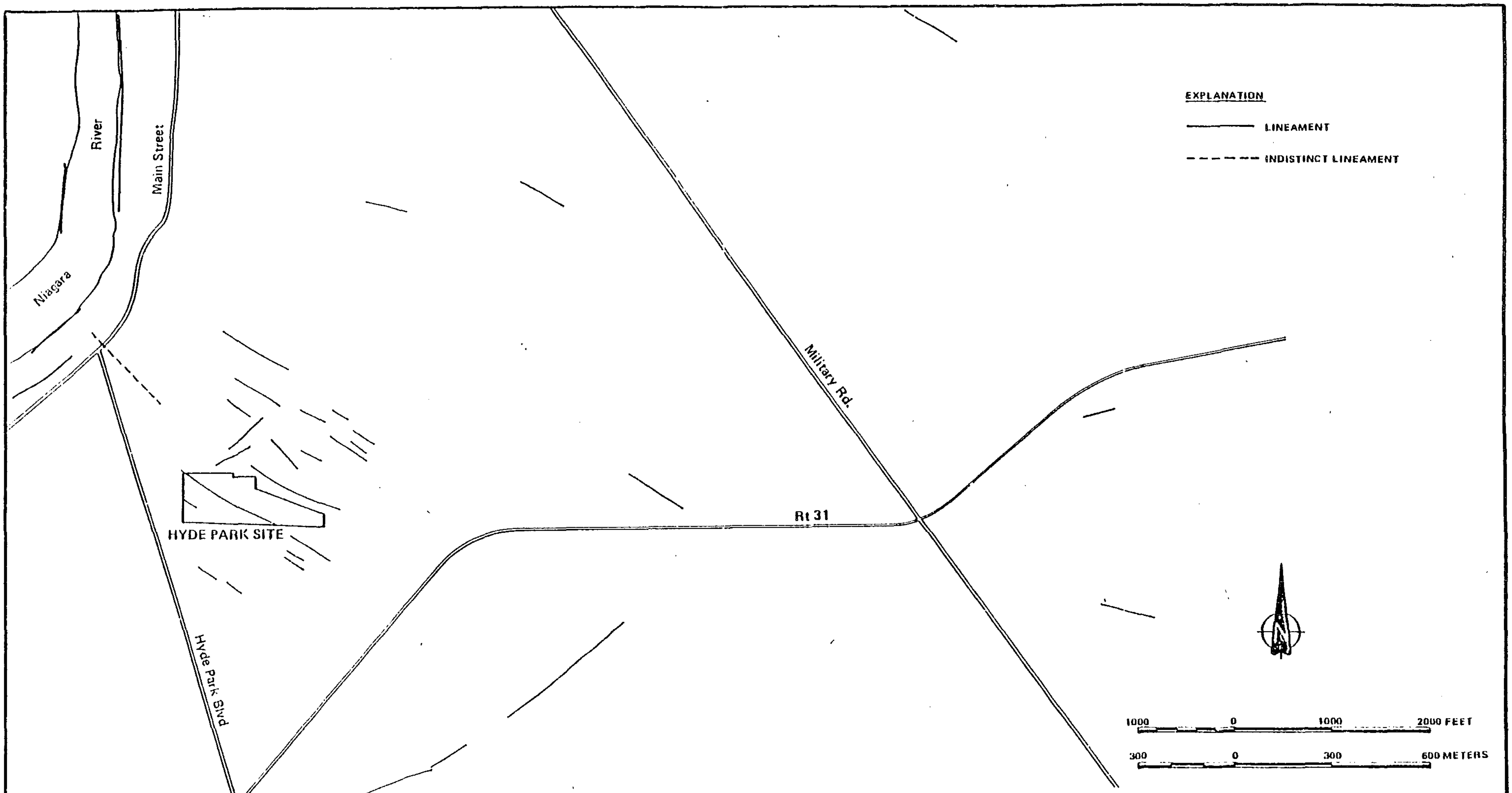
adopted from Geologic map of New York, 1970 N.Y. State Museum and Science service

FIGURE 2

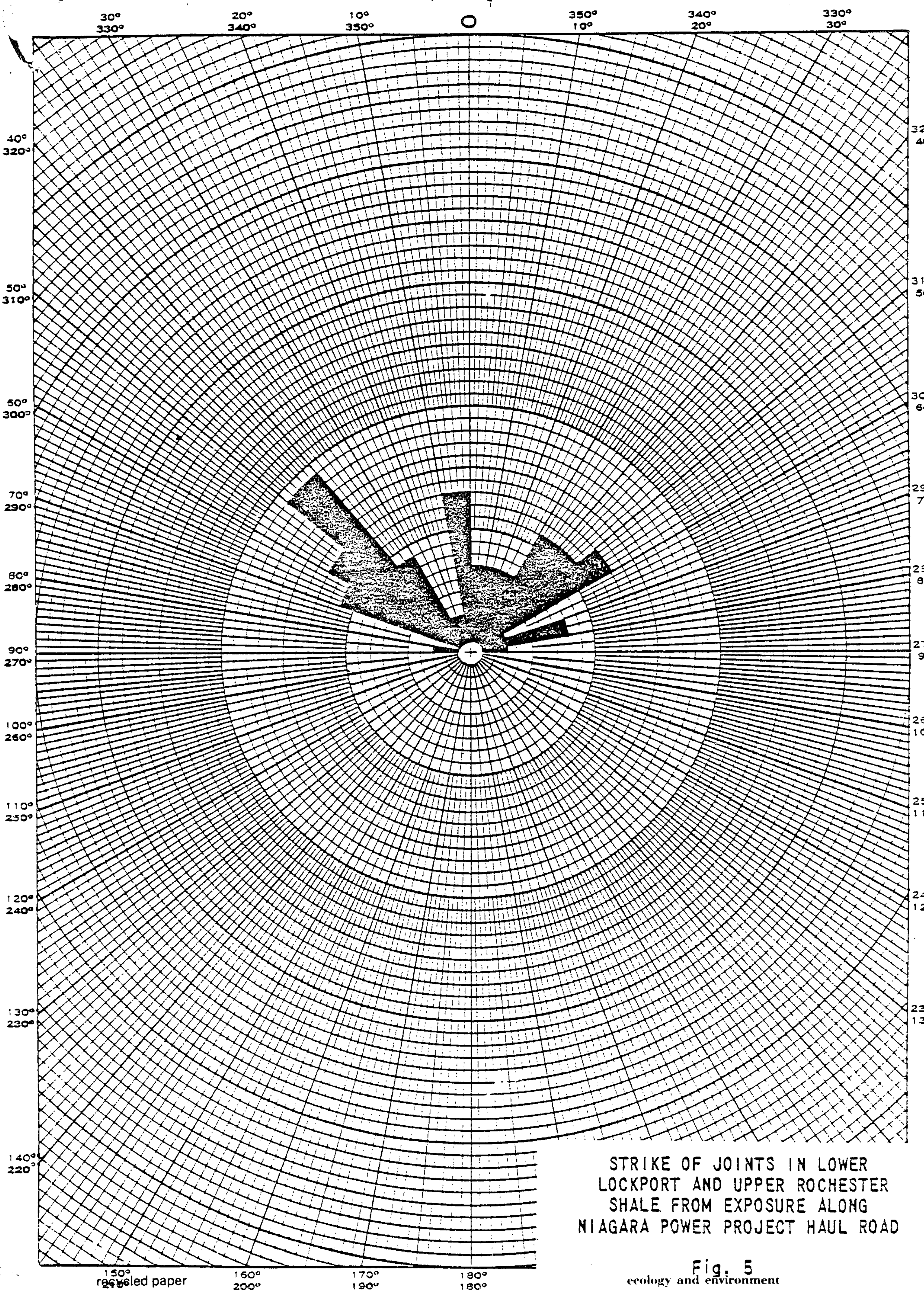


COLUMNAR SECTION OF LOCKPORT FORMATION IN NIAGARA FALLS AREA

after N.Y.State Museum and Science service bulletin no. 404

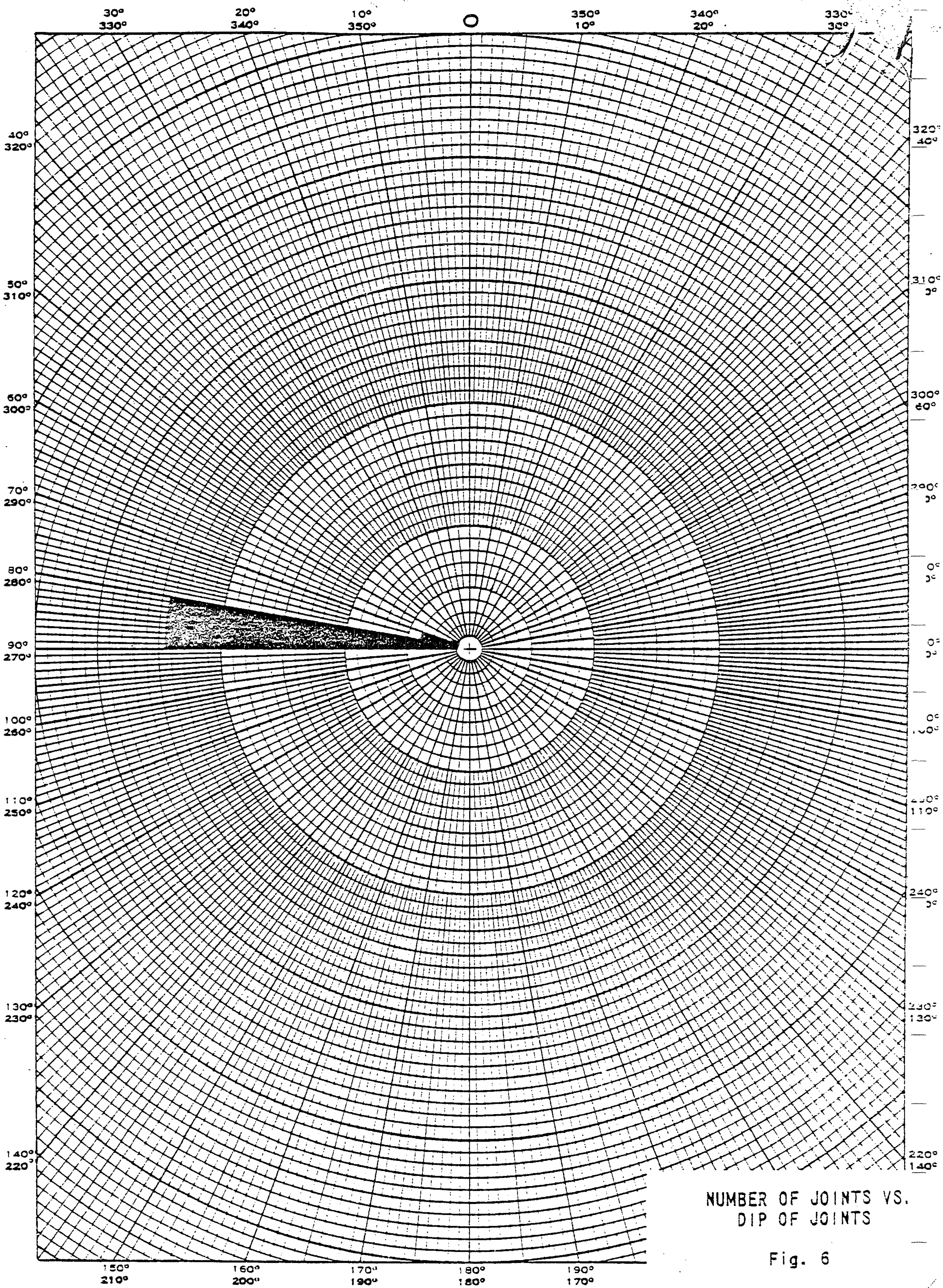


HYDE PARK WASTE DISPOSAL SITE LINEAMENT MAP		
WOODWARD–CLYDE CONSULTANTS CONSULTING ENGINEERS, GEOLOGISTS AND ENVIRONMENTAL SCIENTISTS CLIFTON, NEW JERSEY		
DR. BY: BTD	SCALE: AS SHOWN	PROJ. NO.: 79C01231
CK'D. BY: JWS	DATE: Rev. 3/29/83	Fig. 4



STRIKE OF JOINTS IN LOWER LOCKPORT AND UPPER ROCHESTER SHALE FROM EXPOSURE ALONG NIAGARA POWER PROJECT HAUL ROAD

Fig. 5



NUMBER OF JOINTS VS.
DIP OF JOINTS

Fig. 6