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CELA



The Corporation of the
City of Burlington

**EVALUATION OF HALTON REGION
LANDFILL TECHNICAL STUDIES
Site F - Burlington**

HYDROGEOLOGIC REPORT

- **Part A - Definition of Hydrogeologic Suitability
for Landfilling**
- **Part B - Critique of Trow Technical Studies**
- **Part C - Interpretation of Site F Hydrogeology and
Suitability for Landfilling**

February 8, 1988

Reference No. 1354-10

Mr. Mike McQuaid
Weir & Foulds
Barristers & Solicitors
2 First Canadian Place
P.O. Box 480
Toronto, Ontario
M5X 1J5

Dear Mike:

Re: Site F - Hydrogeologic Report
Halton Environmental Assessment

Please find enclosed our report entitled "Evaluation of Halton Region Landfill Technical Studies, Site F Burlington, Hydrogeological Report".

This report has been prepared on behalf of the City of Burlington as part of the ongoing Environmental Assessment for the new Halton Region landfill.

Should you have any questions, please do not hesitate to contact us.

Yours truly,

CONESTOGA-ROVERS & ASSOCIATES



Anthony J. Crutcher, P. Eng.

AJC/js
Encl.

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1.0 INTRODUCTION

Conestoga-Rovers & Associates (CRA) has been involved in reviewing the hydrogeologic suitability of landfilling at Site F and vicinity for the City of Burlington (City) since 1984.

Initially CRA participated in the 1984 hearing for the expansion of the existing Burlington Landfill. CRA's involvement entailed reviewing the hydrogeologic studies of the Burlington Landfill Site and the hydrogeologic studies undertaken on the adjacent National Sewer Pipe (NSP) lands (now referred to as Site F) and the adjacent Bayview Park Landfill.

Since 1984 CRA has also been involved in reviewing the work undertaken on the Region of Halton Environmental Assessment (Landfill Component). The work entailed extensive comments on the Stage 2C reports, reviewing and commenting on the hydrogeologic field work undertaken during the EPA level of investigations and a detailed review of the final EPA reports.

CRA was also retained by the City to design and supervise the construction of a leachate collection system for the Bayview Park Landfill, and to undertake on-going monitoring of the Site.

In addition to the Trow technical studies reviewed herein, reviews of the following investigations have been conducted by CRA:

- 1) Review of investigations conducted for National Sewer Pipe (NSP) for a private landfill at the now identified Site F (Morrison-Beatty 1980, 1981, Cherry and Lush 1980),
- 2) Review of studies undertaken by the University of Waterloo to delineate the extent of groundwater contamination downgradient of the Bayview Landfill site (Hewetson Exhibit E-214),
- 3) Review of the Stage 2-C studies undertaken for the current Regional Municipality of Halton (RMH) landfill site selection process (Trow Hydrology Consultants Ltd. Exhibit H-38 and H-38A),
- 4) Review of hydrogeologic investigation for the continued use of the Burlington Landfill (Gartner-Lee 1984, Exhibit H-217),
- 5) Review of plume delineation and groundwater quality monitoring at the Burlington Landfill site (Gartner-Lee 1986, Exhibit H-83, 1987),

- 6) Monitoring and an assessment of groundwater quality data at the Bayview Landfill (CRA 1987),
- 7) Inspection of field investigations undertaken by Trow in conjunction with the current RMH landfill site selection process, and
- 8) Installation of leachate wells and construction of leachate collection system at the Bayview Landfill site.

Given the above, CRA is very familiar with the hydrogeologic environment at Site F and vicinity.

This report provides a review of the hydrogeology of Site F and it is written in three parts.

Part A provides CRA's criteria to determine the hydrogeologic suitability of a site for landfilling.

Part B reviews the technical studies prepared by Trow Hydrology Consultants Limited (Trow) for RMH. These studies consist of Volume 1 - Hydrogeologic Report and Volume 2 - Appendices of a three-volume report entitled "Halton Region Landfill Technical Report - Site F Burlington". CRA concludes in Part B that the conclusions presented by Trow are not supported by the field data and data evaluation provided.

Part C contains CRA's interpretation of the site as developed from the previous investigations, reviews conducted earlier by CRA as well as reports prepared by Trow. CRA concludes that the hydrogeology of the site as interpreted by Trow is not supported by the previous investigations and does not meet the hydrogeologic suitability criteria as defined in Part A.

**PART A: DEFINITION OF SITE HYDROGEOLOGIC
SUITABILITY FOR LANDFILLING**

A.1.0 INTRODUCTION

In order to conduct an assessment of the hydrogeologic suitability of a site for landfilling the following items must be addressed:

- 1) The hydrogeologic setting.
- 2) The level of engineering necessary to provide effective primary protection.
- 3) The boundary impacts for landfill design option considered.
- 4) A monitoring program to monitor both the natural hydrogeologic setting as well as the engineering landfill system.
- 5) A contingency plan in the event that the system, natural or engineered, does not perform as expected.

The hydrogeologic setting is defined initially as this provides the basis upon which to develop the required engineering design. The final three of the

) above points are addressed following the definition of the hydrogeologic system and the conceptual engineering design.

From this assessment a hydrogeologically acceptable landfill site must meet the following minimum criteria:

- 1) Migration of contaminants from the landfill must be predictable and the impact from this migration must be determined to be acceptable.
- 2) The site must be able to be adequately monitored in order to verify the predictions on the behavior of the contaminant migration.
- 3) Feasible contingency plans must be developed such that if unacceptable contaminant migration results, as determined from the monitoring, this migration can be mitigated.

) The following sections provide a discussion of these items.

A.2.0 HYDROGEOLOGIC SETTING

It is accepted practice to site landfills in a hydrogeologic setting which minimizes the risk to public health and safety. Such sites are typically characterized by low permeable materials with a significant capacity to attenuate migrating leachate. Emphasis is placed on a high degree of protection provided by the natural environment because of concerns regarding the reliability and long term performance of an engineered landfill system. The engineered component is generally designed to enhance the natural environment most typically by the collection and removal of leachate in order to reduce the volume of leachate that may potentially migrate from the site.

Acceptable sites are generally characterized by clayey or low permeable strata in which the rate of groundwater movement is measured in centimeters per year. In addition, the capacity of clays to attenuate contaminated groundwater enhances their suitability as landfill sites. Thick clay strata are therefore considered to comprise the most favourable hydrogeologic setting for a landfill.

Commonly however, in most hydrogeologic settings the low permeable strata are interbedded with sandy or more permeable strata in which the rate of groundwater flow is measured in the order of meters or even tens of

meters per year. Such strata are conduits for horizontal movement of groundwater and may comprise an exploitable groundwater resource.

It is therefore the basis of any hydrogeologic investigation aimed at assessing the suitability for landfilling to establish the geologic sequence of the site in order to define those strata which provide for the attenuation and limited migration of contaminants, and those which provide for off-site lateral migration. Such investigations require drilling to identify the geologic strata present, the installation of observation wells, and the conducting of various hydrogeologic tests, in both the field and laboratory, to quantify those hydrogeologic parameters that govern the rate and direction of groundwater flow in each of the geologic strata identified. The measurement of water levels, grain size analyses, single well response tests and pumping tests comprise the more common hydrogeological tests undertaken in such investigations.

Groundwater quality is normally established prior to landfilling. Samples are collected from observation wells which have previously been developed to ensure that the samples are representative of the groundwater. A number of these observation wells are typically included in a monitoring program. In addition to analyzing for background

water quality, laboratory tests may be completed to assess the attenuation properties of the geologic strata.

The data generated are presented in plan view and cross-section in a manner which represents the groundwater flow system at the site. From an understanding of the flow system it is then possible to define the pathways along which leachate generated at the site may potentially migrate off site. Having defined the contaminant pathways, the travel time taken for contaminants to migrate in the groundwater environment to a discharge point or to the site boundary is undertaken. This assessment may involve a few simple calculations or the application of sophisticated computer models incorporating groundwater flow and contaminant transport.

A.3.0 CONCEPTUAL ENGINEERING DESIGN

The design of an engineered landfill site is controlled by the hydrogeologic conditions encountered as well as the physical feature of the site (slope, surface water control, etc.) The various design options of the engineered system are evaluated to ensure they are compatible with the hydrogeological system. This section of the report addresses those hydrogeologic factors that influence the design of a landfill.

The objectives of an engineered sanitary landfill are threefold:

- 1) Minimize the rate of leakage of leachate from the facility.
- 2) Provide for the collection of leachate during and following the operational life of the facility.
- 3) Minimize the amount of leachate production following closure of the facility.

For those sites founded on low permeable materials, the first two objectives are addressed in the design of the leachate collection system. Where the integrity of the subgrade materials is questionable, a low

permeable surface (liner) may have to be installed at the base of the excavation to minimize the potential for leachate to migrate from the landfill. Clay liners are typically designed to a hydraulic conductivity of ~~10~~ 10^{-8} cm/sec.

The third objective is addressed by the installation of an engineered cover at closure of the site.

The objective of the engineering for a sanitary landfill is to provide for the collection of leachate which is generated when precipitation percolates through the landfilled materials. This is usually accomplished by providing a drainage layer at the base of the excavation with gravity drainage to a collection point. This drainage layer may consist of a sand blanket, a series of collection pipes or french drains or any combination of the above. This system is designed to control the elevation of leachate within the landfill and to minimize the potential for the leakage of leachate from the landfill. For a low permeable base, the rate of leakage of leachate from the site is low and a less sophisticated drainage layer is feasible. For a more permeable base, the rate of leakage is higher and a more effective leachate control system is required.

 The thickness and hydraulic conductivity of the strata immediately underlying the landfill is of critical importance.  Materials having a hydraulic conductivity on the

order of 1×10^{-7} cm/sec or less, and a good capacity to attenuate leachate (e.g. clays) are considered most suitable for landfilling. In general a minimum thickness in the order of 3 to 5 m of low permeable clayey till materials should be maintained below the base of excavation. The thickness of the till allows for some variability in the till lithology while still maintaining a degree of assurance as to the integrity of the unit as a whole.

A.4.0 IMPACT ASSESSMENT

This section addresses the estimated time of travel and anticipated peak concentrations of leachate, which may migrate in the groundwater environment to:

- 1) Discharge to adjacent surface water and/or,
- 2) Reach the site boundary and/or,
- 3) Reach a receptor of groundwater.

It is the intention of an impact assessment to demonstrate that, based on the observed hydrogeologic system and the proposed engineered landfill system, that operation of the landfill will meet accepted criteria regarding groundwater quality at the site boundary.

It should be pointed out, however, that the effects of a failure of the engineered system must also be assessed. This is assessed by assuming that there is an uncontrolled build up of leachate in the site which has the effect of increasing the rates of groundwater flow and reducing the time of arrival of contaminants at the site boundary.

The significance of these effects are assessed in light of the landfill design and the monitoring program and contingency options proposed.

A.5.0 MONITORING PROGRAM

A monitoring program is designed to provide data upon which to assess the performance of both the natural hydrogeologic system and the engineered system.

The intent of the groundwater monitoring program is:

- 1) To define background water quality to provide a data base against which to assess the significance of any observed groundwater quality changes.
- 2) To ensure that any change in the groundwater quality in the defined contaminant pathways is readily detected.
- 3) To assess the significance of any change detected with respect to acceptable groundwater quality criteria.

Monitoring of the engineered component of the landfill is designed to evaluate the effectiveness of the leachate control and collection system.

A.6.0 CONTINGENCY PLANS

A contingency plan is required in order that an unacceptable environmental impact if it occurs can be mitigated. The contingency options assessed are aimed at preventing the continued migration of contaminated groundwater. This prevention may be undertaken by extraction, hydraulic control or physical containment. Design of an effective containment system requires a thorough understanding of the groundwater flow system. In addition the monitoring program must be able to provide an early warning of potentially unacceptable changes in the groundwater quality in order that a contingency system can then be implemented.

PART B: CRITIQUE OF TECHNICAL STUDIES

B.1.0 INTRODUCTION

CRA has reviewed the Trow Hydrology Consultants (Trow) report entitled "Halton Regional Landfill Technical Report - Site F, Burlington", September, 1986 (Exhibit H-40) in detail and the following sections provide CRA's critique of this report. While the report deals with many facets, the basic principle of the hydrogeologic interpretation of the site as put forward by Trow is that the shale bedrock at the site can be differentiated into four discrete subhorizontal hydrostratigraphic units. This principal is fundamental to Trow's evaluation of the site as the computer modelling, potential contaminant pathways and the impact assessment are based on this interpretation.

Therefore CRA has concentrated this critique of the Trow report principally in this area. Part C of this report provides CRA's hydrogeologic evaluation of Site F.

It is CRA's understanding that Trow interpretation of these four hydrostratigraphic units were based on the following:

- 1) inspection of rock cores,
- 2) water levels and vertical hydraulic gradients,

- 3) single well response tests and pressure packer tests,
- 4) pumping tests, and
- 5) computer modelling.

This review conducted by CRA examined the same data and concludes that the bedrock at the site cannot be differentiated into the hydrostratigraphic units postulated by Trow. The fracture pattern at the site is complex. The location, orientation and degree of connection of those fractures along which groundwater at the site migrates is unknown and unpredictable. In such a complex groundwater system there is a significant amount of uncertainty associated with quantifying those hydrogeological parameters that govern the rate, and on a local scale the direction, of groundwater flow.

In addition, the impact on groundwater at the adjacent landfill sites was assessed. CRA's review of this data shows that it has not been demonstrated that the monitoring well screens straddle the most permeable fracture zones and therefore do not necessarily represent the groundwater quality of the major contaminant pathways. The deep flow path, as defined by potentiometric data, has not been monitored. The monitoring work done at the adjacent landfills highlights the difficulties in delineating a contaminant plume in this environment.

The groundwater flow model used by Trow was examined in detail by CRA. CRA concludes from this examination that the infiltration is very sensitive to changes in the hydraulic conductivity and further, that a variety of hydraulic conductivity values of the shale bedrock can produce a similar pattern of regional groundwater flow. Based on this review, CRA concludes that a calibrated groundwater flow model does not confirm the hydrostratigraphy postulated by Trow. In addition, CRA does not support the results of the contaminant transport modelling and the hydrogeologic aspects of the landfill design both of which are in part based on the results of the groundwater flow modelling.

Portions of the Trow report are not incorporated in this review as they are not considered critical to the conclusions regarding the hydrogeologic suitability of the site. Absence of specific review of these sections does not necessarily imply agreement with their contents.

Figure B.1.1 (p.19) and Plan 1 (back pocket) show the site plan and borehole locations which are referred to in detail in subsequent sections of this report.

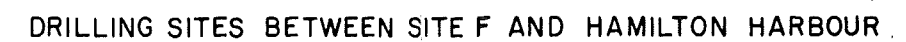
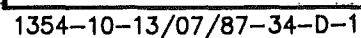


figure B.I.1
SITE PLAN AND BOREHOLE LOCATIONS
Site 'F'-Hydrogeology- City of Burlington

B.2.0 GEOLOGY

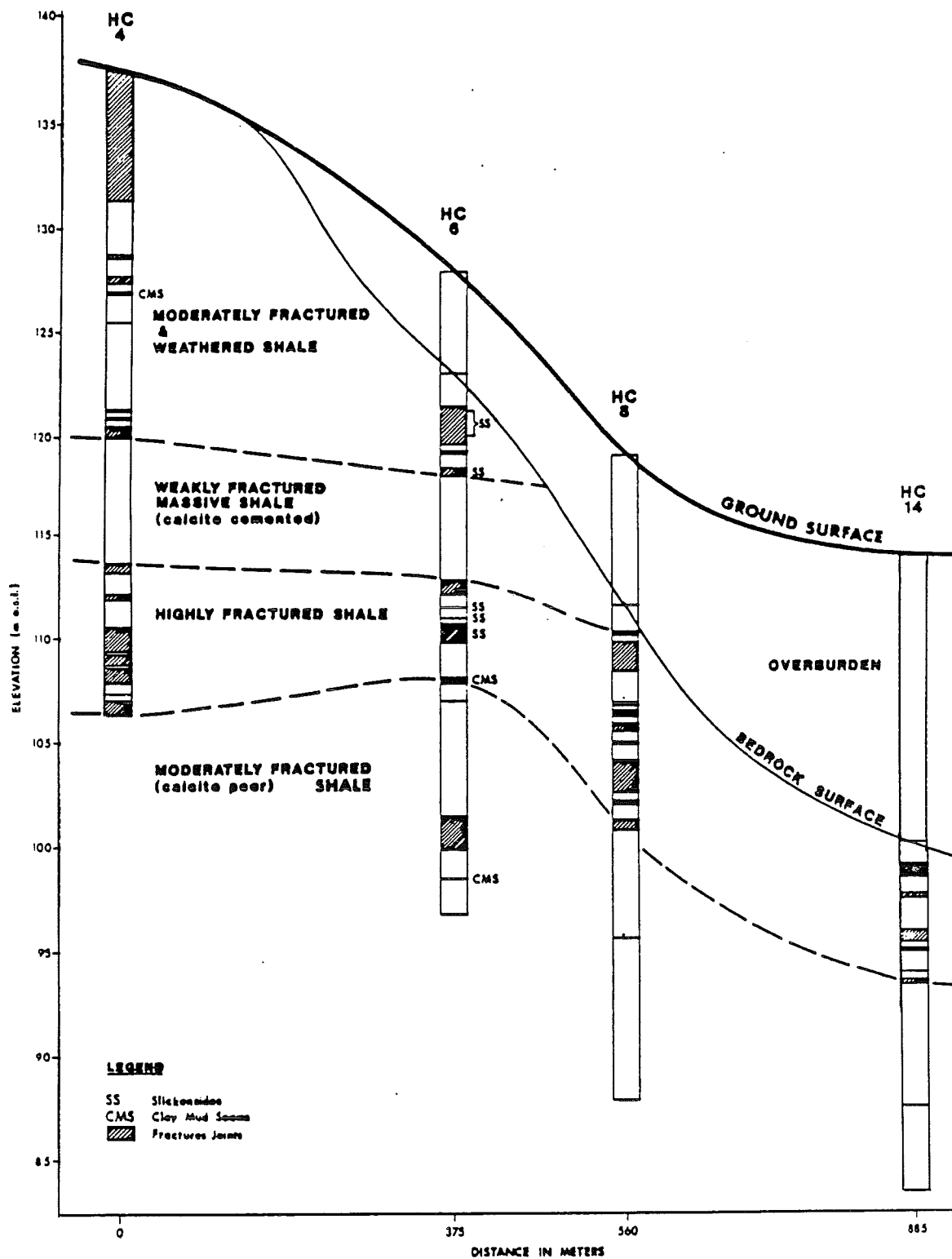
B.2.1 LOCAL GEOLOGY

CRA agrees with the general description of the geologic materials encountered at the site. This sequence is described as a thin covering of clayey silt till to silt till and/or exposed weathered shales overlying shales of the Queenston Formation. The upper portions of the shale bedrock are weathered and fractured. In general the fracture frequency decreases with depth but is highly variable.

B.2.2 BEDROCK FRACTURE PATTERN

CRA does not agree with the delineation of the fracture zones presented by Trow as interpreted from a detailed study of the core logs. This delineation as presented by Trow (Figure 2, p. 35; Exhibit H-40) is reproduced here as Figure B.2.1 (p. 21).

There is insufficient data presented by Trow to support the differentiation of the rock core into four discrete zones of variably fractured shale. Further, CRA concludes that the lateral extent of these zones cannot be demonstrated by correlation between boreholes.



SOURCE: TROW (1986), FIGURE 2, pg35, EXHIBIT H-40

figure B.2.1

CRA

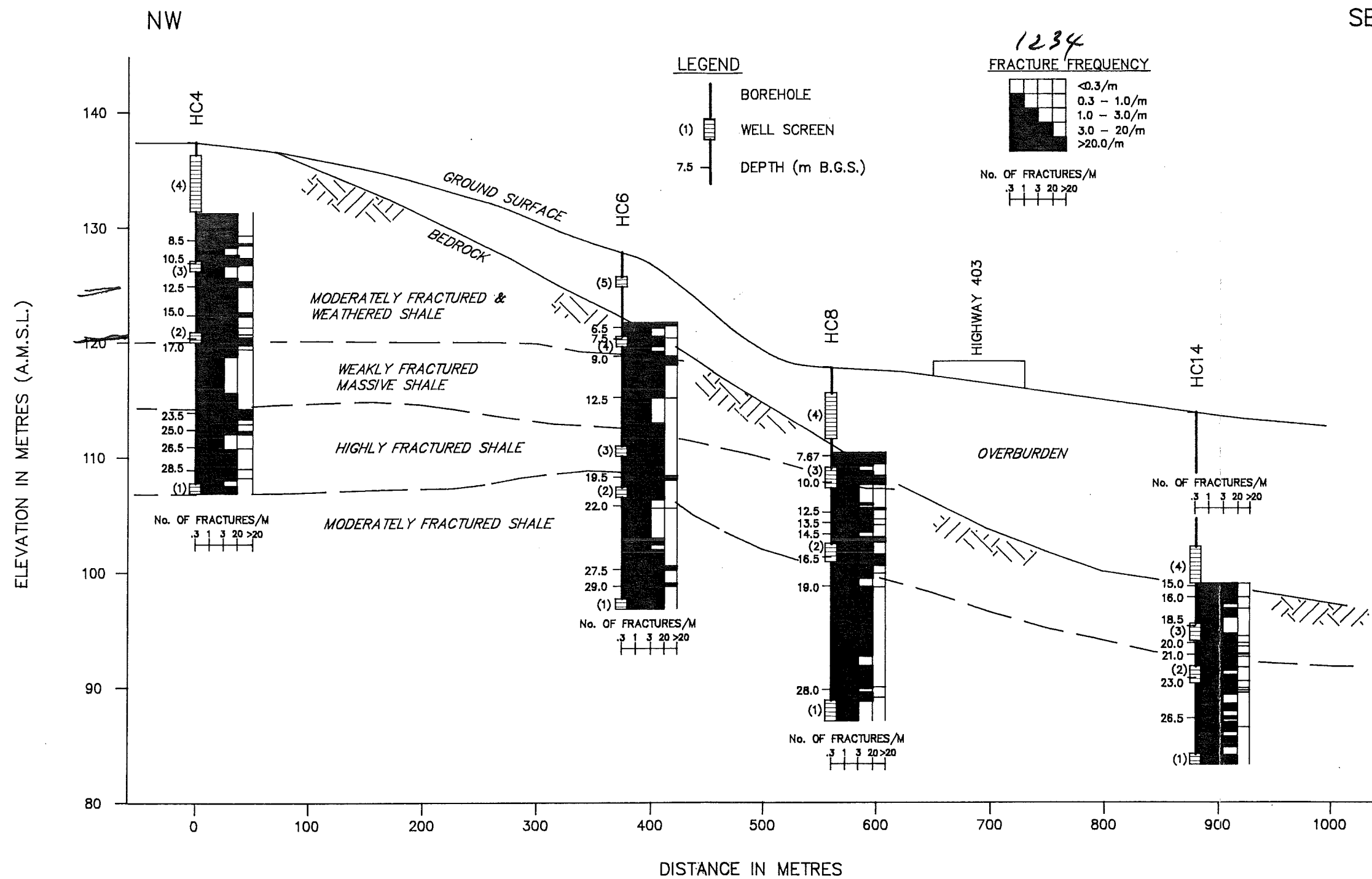
FRACTURE DISTRIBUTION
Site 'F'-Hydrogeology - City of Burlington

A general fracture network was interpreted by Trow. The principle elements considered by Trow in delineating these fracture zones are visual inspection of the rock core, fracture frequency, and the presence of slickensides and mudseams.

CRA has examined the core log records presented by Trow in Appendix 2 (Exhibit H-40A) and has plotted, for the boreholes shown on Figure B.2.1, the fracture frequency (number of fractures per metre length of rock core) against elevation. These fracture frequency profiles are shown on Figure B.2.2 (p.23).

The fracture frequency logs (column 14 on the coring records presented by Trow in Appendix 2; Exhibit H-40A) delineate highly fractured zones or zones of very closely spaced joints. Thin zones of as little as 5 cm in width are identified on these logs. The fracture frequency profile should therefore reflect the fracture zones interpreted by Trow.

Examination of Figure B.2.2 does not support Trow's contention of identifiable zones of similar fracture frequency but does illustrate the erratic nature of the fracture distribution. Specifically, zones which appear at least as fractured as the 'highly fractured shale' occur within the 'moderately fractured and weathered shale' at HC4, and within the 'moderately fractured shale' at HC6. Zones



SOURCE: TROW (1986) EXHIBIT H-40
1) DEFINITION OF FRACTURE ZONES - figure 2, p.35;
2) FRACTURE FREQUENCY AND BOREHOLE DEPTHS FROM APPENDIX 2

figure B.2.2
FRACTURE FREQUENCY PROFILES
Site 'F'-Hydrogeology - City of Burlington

which appear equivalent to the 'weakly fractured shale' occur within the 'highly fractured shale' at HC4, HC8, and HC14, and within the 'moderately fractured shale' at HC6, HC8 and HC14.

The core from HC4 was inspected by CRA at the laboratory of Trow. This inspection showed that the fracture frequency was highly variable along the length of the borehole. In general, it was noted that the fracture frequency correlates with that presented in Appendix 2 (Trow, Exhibit H-40A) for this borehole. Lengths of core with a low fracture frequency were generally encountered between depths of 12.5 to 16 m (124.9 to 121.4 masl), 18.5 to 21.5 m (118.9 to 115.9 masl) and 25.5 to 26.5 m (111.9 to 110.9 masl). These depths indicate that weakly fractured rock is encountered in each of the 'moderately weathered and fractured shale' the 'weakly fractured massive shale' and the 'highly fractured shale'.

This point was brought out in cross-examination of Mr. Beechinor (Vol. 37, p. 8171) when examining the core samples from HC6. As indicated by Mr. Beechinor, there is considerable variation in the fracture pattern within each of the geologic units. With reference to the two core boxes for HC6 which cover a portion of the 'highly fractured shale' as well as the underlying

'moderately fractured shale' (Boxes 7 and 8; Exhibit H-223), Mr. Beechinor indicated (Vol. 37, p. 8171) that there wasn't much geological difference between these units.

CRA concludes that based on variations in the fracture pattern, these geological units cannot be delineated from inspection of the core samples. The core samples do show the existence of discrete fractured zones, often less than 10 cm thick, which are bounded by a less fractured rock mass. There is a significant variation in the frequency of fracturing within this less fractured rock mass with some lengths of core showing fewer than 3 fractures per metre length.

CRA does not consider it possible to correlate the fracture pattern between the boreholes and therefore can not differentiate the site into distinct geologic units each with its characteristic fracture pattern.

B.3.0 HYDROSTRATIGRAPHY

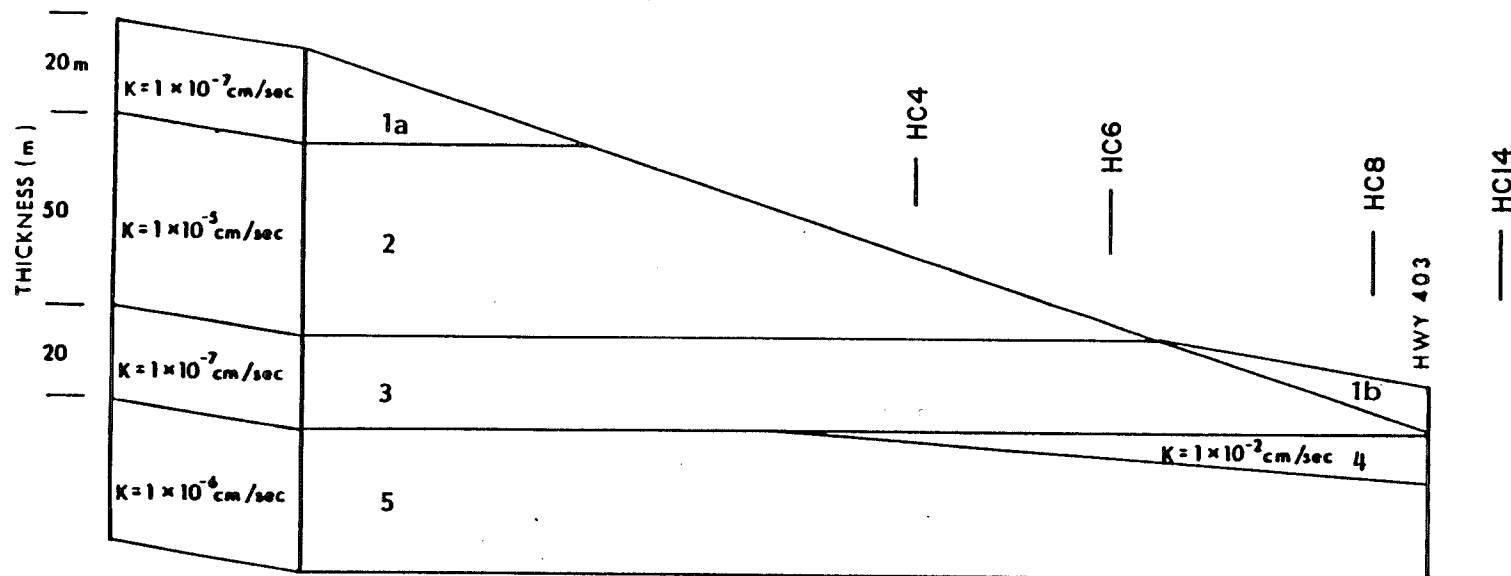
B.3.1 HYDROSTRATIGRAPHIC UNITS

CRA does not agree with the hydrostratigraphy presented by Trow. Each of the supporting arguments addressed by Trow, namely water levels, hydraulic gradients, hydraulic conductivity, pumping tests and the tritium distribution are examined in turn by CRA and shown to provide insufficient support.

Trow (p.48; Exhibit H-40) differentiates the site into five major hydrostratigraphic units, namely;

- 1) overburden
 - a) till unit at north end of site
 - b) discharge zone till
- 2) fractured shale unit - highly weathered at surface
- 3) weakly fractured massive shale unit
- 4) highly fractured shale unit
- 5) moderately fractured bulk shale unit

These units and their assigned hydraulic conductivity values are shown schematically on Figure B.3.1 (p. 27) which is reproduced from Trow (Figure 4, p.49; Exhibit H-40).



- 1) OVERBURDEN
 - a) TILL UNIT
 - b) DISCHARGE ZONE TILL
- 2) FRACTURED SHALE UNIT - HIGHLY WEATHERED AT SURFACE
- 3) WEAKLY FRACTURED MASSIVE SHALE UNIT
- 4) HIGHLY FRACTURED SHALE UNIT
- 5) MODERATELY FRACTURED BULK SHALE UNIT

figure B.3.1

SUMMARY OF HYDROSTRATIGRAPHIC UNITS
Site 'F'-Hydrogeology - City of Burlington

SOURCE: TROW (1986), FIGURE 4, pg49 EXHIBIT H-40
 NOTE: BOREHOLE LOCATIONS ADDED BY CRA

CRA

It should be noted that the above hydrogeologic units appear to correspond to the previously defined geologic units (Figure B.2.1, p.21). It is not readily apparent from the Trow report (Exhibit H-40) that the hydrogeologic units are not equivalent to the geologic units. In addition, nowhere in the report are the hydrogeologic units delineated with respect to each (or any) borehole.

The above was resolved during cross-examination of Mr. Sobanski and Mr. Beechinor (Vol. 36, p.7872 onward). The detailed working drawing showing the hydrostratigraphic framework of the site was presented by Trow as Exhibit H-220. This exhibit, reproduced here as Figure B.3.2, (p. 29) details the fracture frequency data, the hydraulic conductivity data determined from slug tests, packer tests and pumping tests as well as tritium data for most of the boreholes on site. [It should be noted that Figure B.3.2 reproducing Exhibit H-220, may not correspond exactly to the current Exhibit H-220 as additional markings were plotted by Trow during cross-examination.] Based on this exhibit, Mr. Beechinor delineated the hydrostratigraphic units for boreholes HC4, HC6, HC8 and HC14 (Vol. 37, P. 8078).

Figure B.3.3 (p. 30) summarizes this hydrostratigraphic delineation and shows for comparative purposes, the position of the geologic unit, the 'highly fractured shale', as taken from Figure B.2.2 (p. 23).

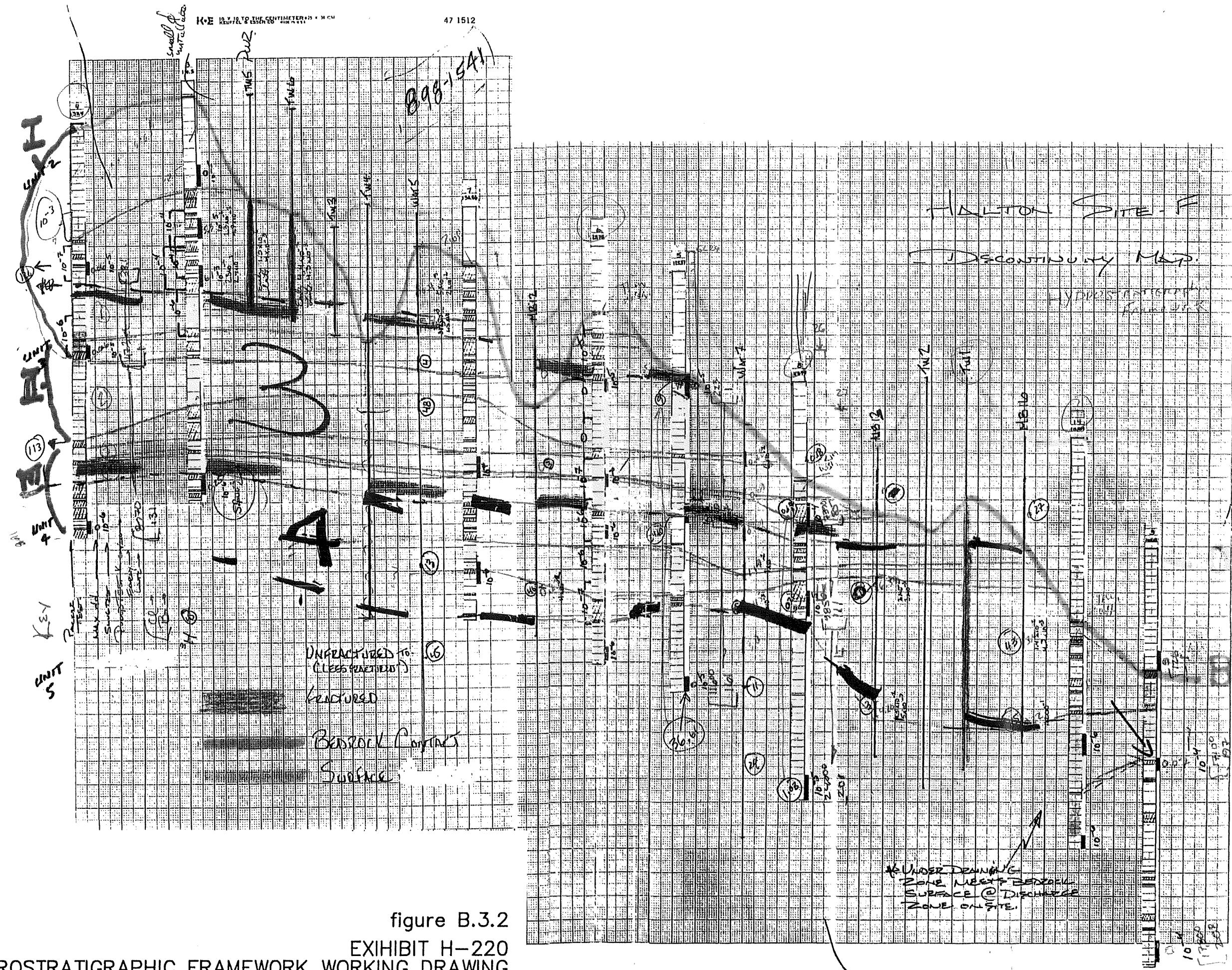
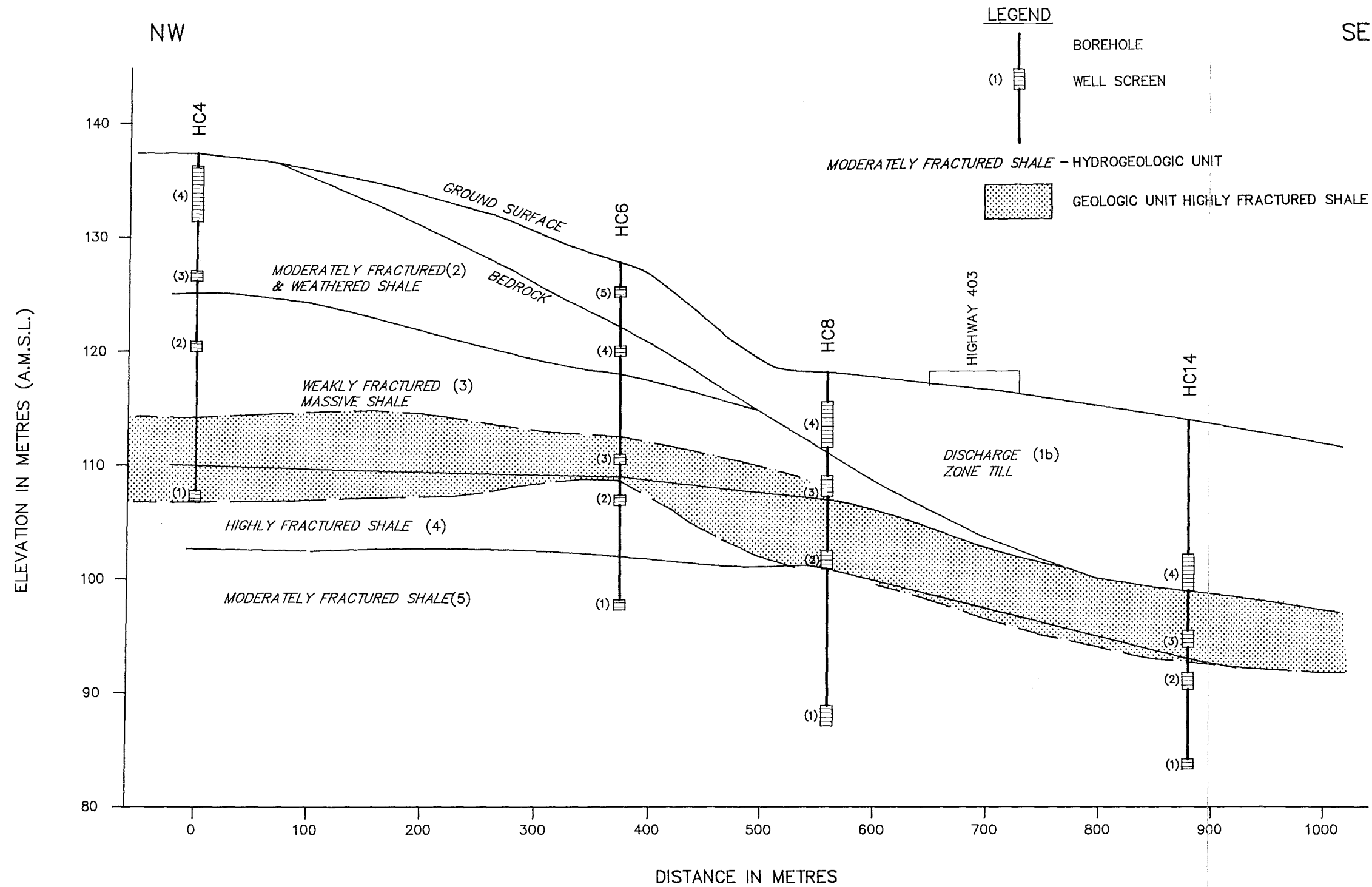


figure B.3.2
 EXHIBIT H-220
 HYDROSTRATIGRAPHIC FRAMEWORK WORKING DRAWING
 Site 'F'—Hydrogeology — City of Burlington



VERTICAL SCALE 1:400
HORIZONTAL SCALE 1:4000

SOURCE: 1) BOREHOLE LOCATIONS AND SCREEN ELEVATIONS - TROW (1986) EXHIBIT H-40A
2) HYDROSTRATIGRAPHIC UNITS - EXHIBIT H-220
3) GEOLOGICAL UNITS - TROW (1986) FIGURE 2 EXHIBIT H-40

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figure B.3.3
HYDROSTRATIGRAPHY
Site 'F' - Hydrogeology - City of Burlington

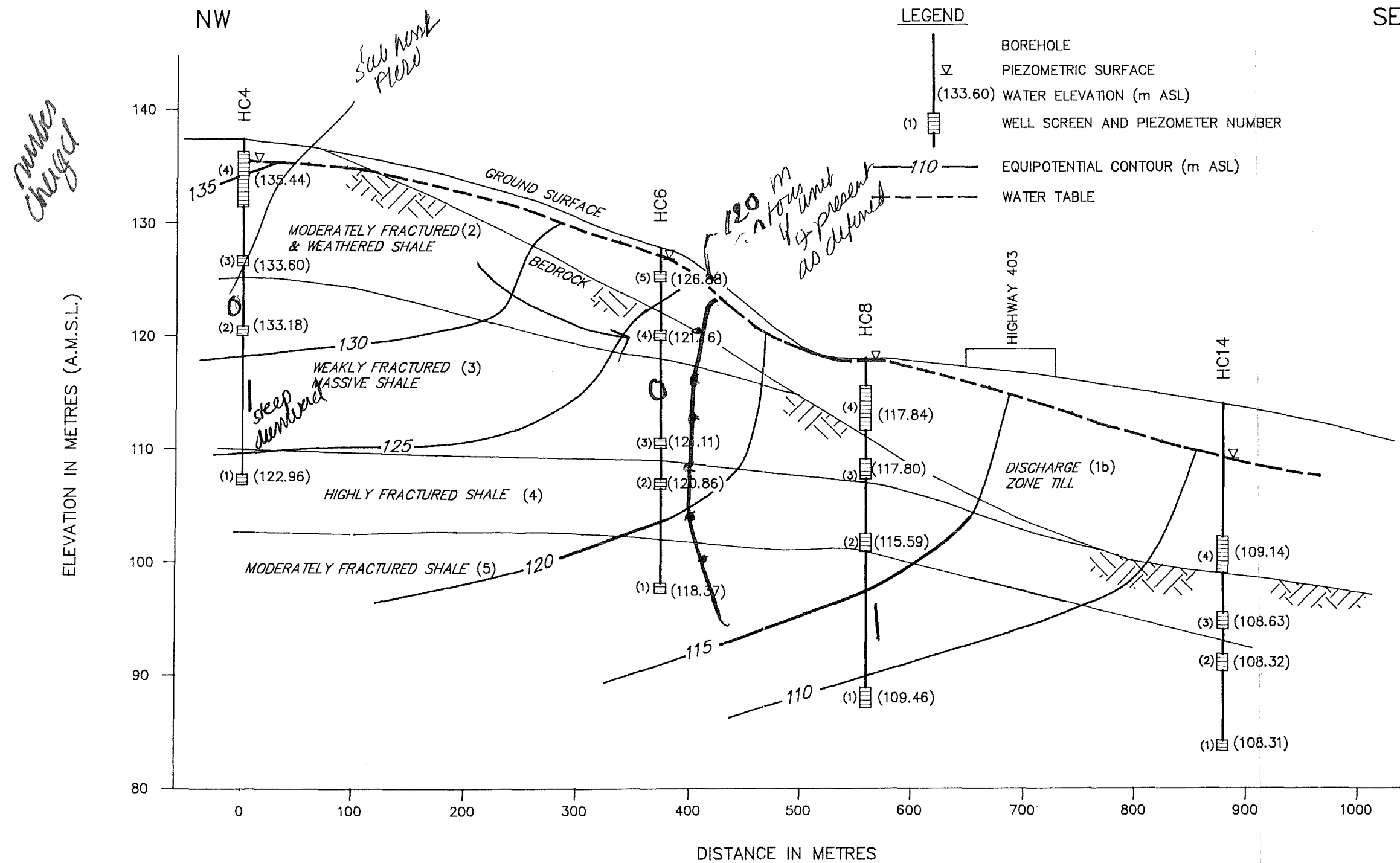
This figure shows that the 'highly fractured shale' (geologic unit) at HC6 lies within the 'weakly fractured massive shale' (hydrogeologic unit 3). This confirms Mr. Beechinor's conclusion, that it is not possible to define the position of the hydrogeologic units based solely on inspection of the fracture frequency or discontinuities (Vol. 37, p. 8131).

From inspection of Figure B.3.2 (Exhibit H-220), a thickness for the 'weakly fractured shale' (Unit 3) of 20 m is not supported by the data provided. Similarly, the wedge shape for Unit 4 is not supported by the field data.

Accordingly, it is CRA's opinion that the working drawing (Exhibit H-220) does not support the hydrostratigraphy summarized on Figure B.3.1 (p. 27).

B.3.2 WATER LEVELS

The available water level data for April 1986 (Trow, Appendix 5; Exhibit H-40A) are plotted and contoured on the base hydrogeological cross-section (Figure B.3.4, p. 32). This contoured distribution shows a trend to lateral groundwater flow in the upper portions of the bedrock



SOURCE: 1) BOREHOLE LOCATIONS AND SCREEN ELEVATIONS - TROW (1986) EXHIBIT H-40A
 2) HYDROSTRATIGRAPHIC UNITS - EXHIBIT H-220
 3) WATER ELEVATIONS - APRIL, 1986 - TROW (1986) (Appendix 5) EXHIBIT H-40A

figure B.3.4
POTENTIOMETRIC DISTRIBUTION
Site 'F' - Hydrogeology - City of Burlington

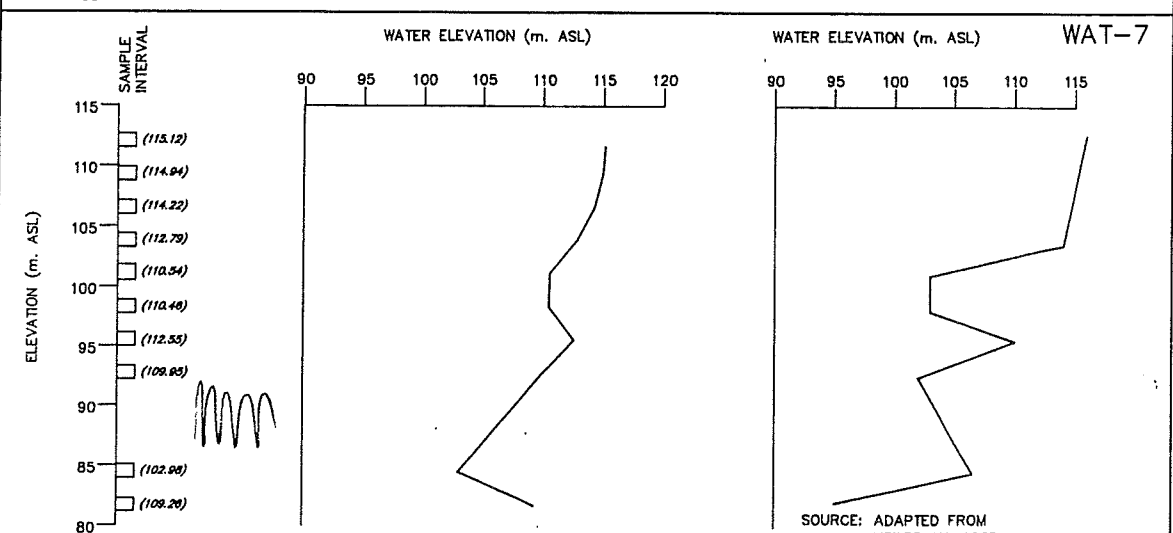
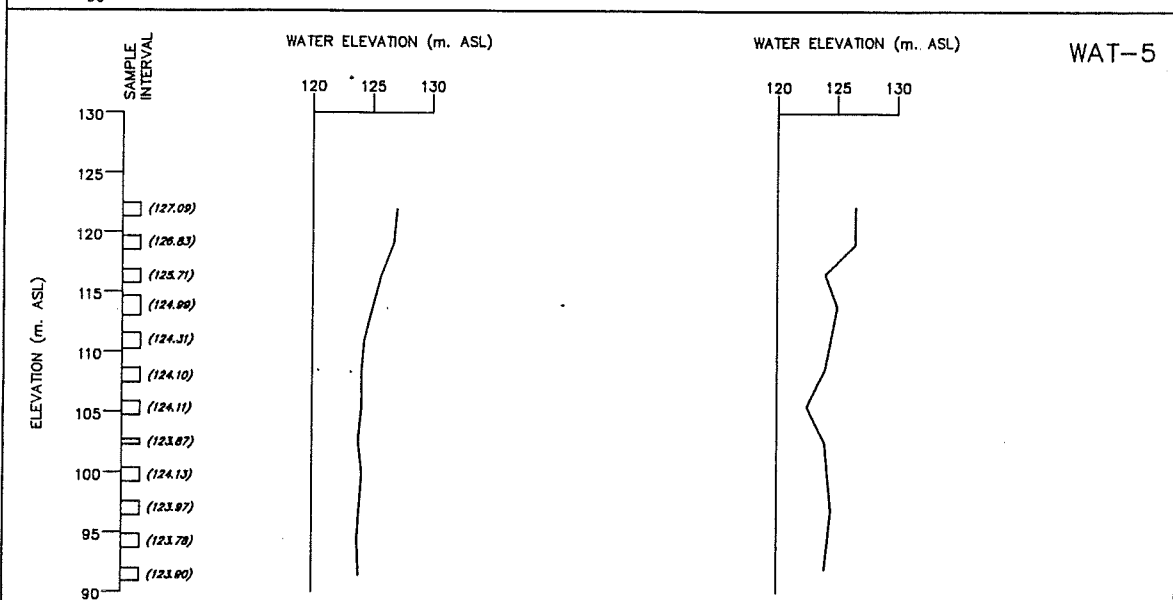
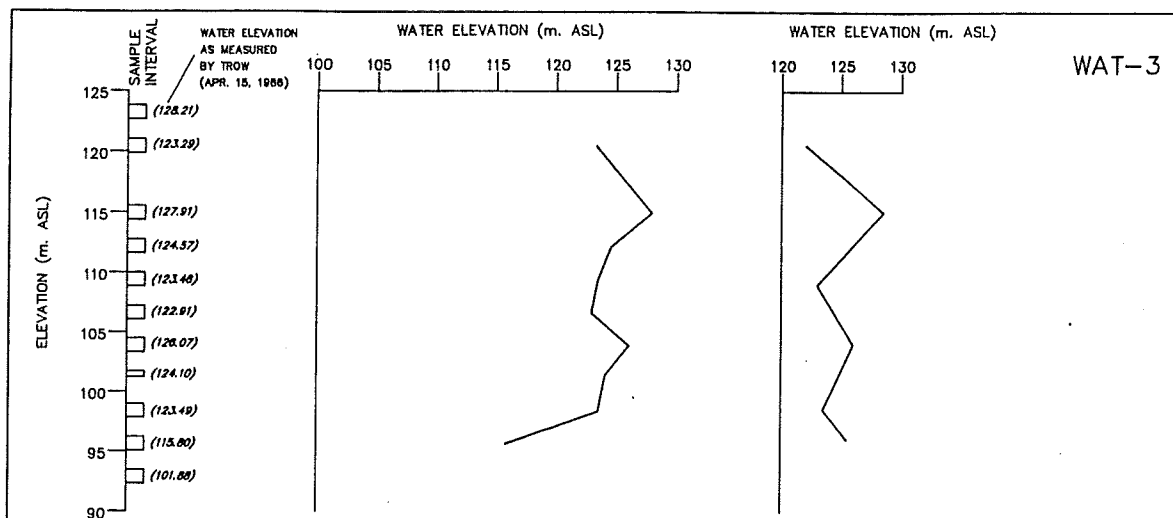
profile. At depth the groundwater flow is steeply downwards although this trend is less pronounced to the southeast i.e. downslope of the site.

(The existence of a 'highly fractured shale' (Unit 4) in which lateral flow would be expected to dominate is not apparent from this contoured pattern.) In addition, at HC4 steep downward flow is apparent only in the lower portions of the 'weakly fractured massive shale' (Unit 3) and not completely across this unit as is expected.

The small scale complexity of the ^{local} groundwater flow system is demonstrated by the water level data obtained from the University of Waterloo multilevel piezometers, the WAT series of installations. Figure B.3.5 (p.34) provides the water level data for those WAT installations monitored by Trow (Appendix 5, Exhibit H-40A). In addition, the hydraulic head profiles developed by Hewetson (Figure 5, Exhibit E-214) are also shown.

It should be noted that the water level data for Hewetson is not provided in Exhibit E-214. Rather the pressure head profiles were reproduced directly from Figure 5 (Exhibit E-214).

These patterns indicate, that on a local scale, the groundwater flow pattern is complex, characterized



SOURCE: TROW, APR., 1986.
EXHIBIT H-40A

SOURCE: ADAPTED FROM
HEWETSON, 1985.
EXHIBIT E-214

figure B.3.5

CRAHYDRAULIC HEAD PROFILES
Site 'F'—Hydrogeology — City of Burlington

by numerous reversals in hydraulic gradient. The low water level at the lowest piezometers in WAT-3 indicate a strong downward gradient at this point. Also there are some significant variations in the profiles developed by Hewetson (Exhibit E-214) and that plotted by CRA from the Trow data (Appendix 5, Exhibit H-40A). Trow (Exhibit H-40) did not address the water level data obtained from the WAT installation.

cross by tidal addressed sub. but content is statement. Hamer. local total complex

CRA concludes that the available water level data indicate that the groundwater flow system is complex. The water level data does not support the hydrostratigraphic interpretation provided by Trow.

B.3.3 HYDRAULIC GRADIENTS

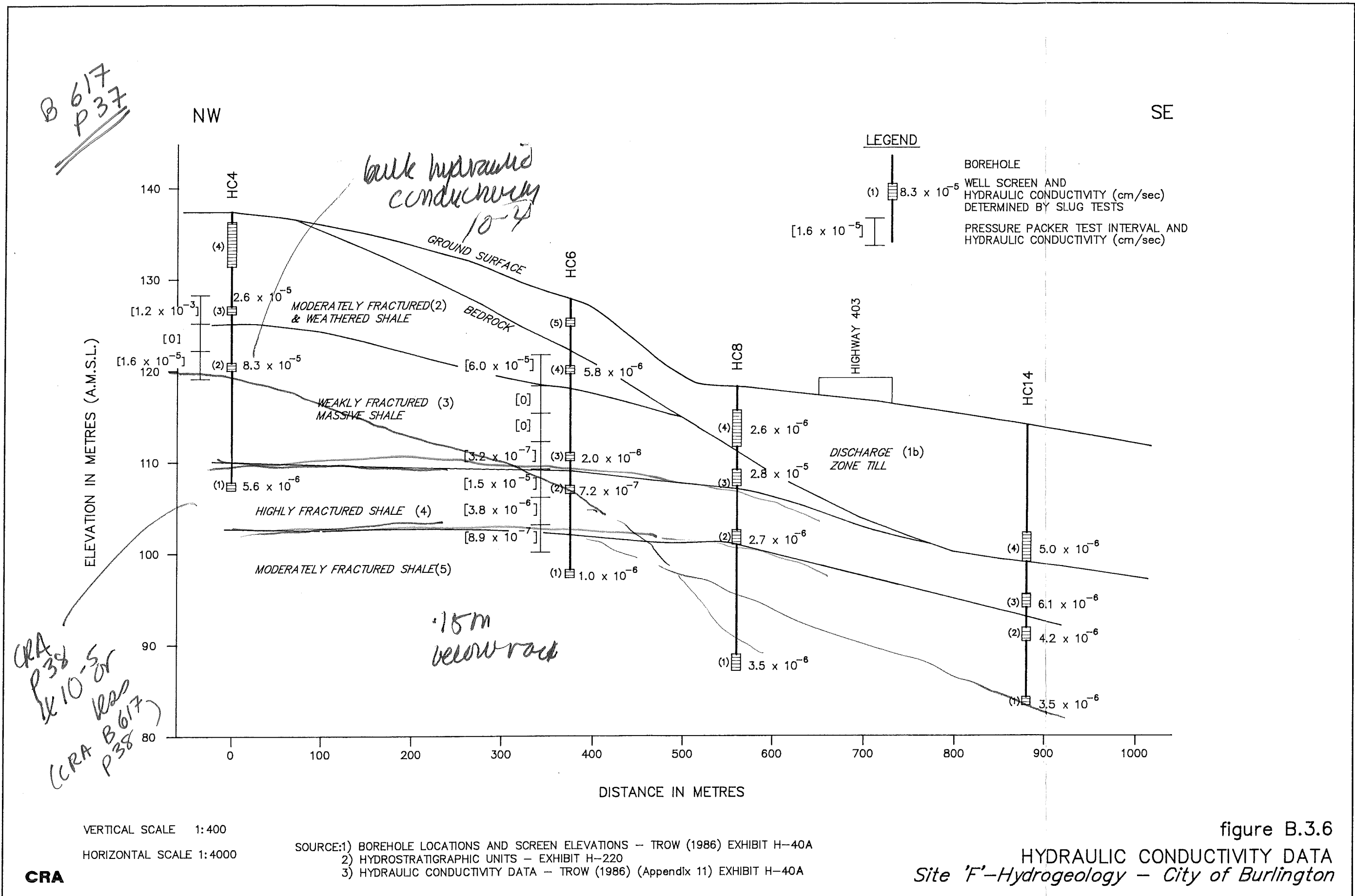
The vertical hydraulic gradients were determined by CRA for each well nest for each available set of water level data. These gradients are provided in Appendix I. In general, these data show strong downward gradients on site with values in excess of 0.2 and some even in excess of 0.5. Downward gradients are much less steep immediately south of Highway 403, generally less than 0.05. Further to the south gradients are sub-horizontal with some upward gradients encountered.

CRA concludes that these gradient data do not support the delineation of the site into the four hydrostratigraphic units as postulated by Trow for the following reasons:

- 1) Strong downward gradients are encountered within and below the 'highly fractured shale' (Unit 4) at HC5, HC6, HC8 and HC9, MB13 and MB16, where if the unit existed horizontal gradients would be expected.
- 2) Low downward gradients are encountered within the 'weakly fractured shale' (Unit 3) at HC4 and HC6, where if this unit existed as defined, steep downward gradients would be expected.
- 3) Pronounced downward gradients are encountered off-site adjacent to the discharge zone e.g. at HC9 (1-2) a downward gradient of in excess of 0.1 is recorded which does not support the upward (discharge) gradient interpreted by Trow.

B.3.4 HYDRAULIC CONDUCTIVITY

The available hydraulic conductivity data (Trow, Appendix 11; Exhibit H-40A) are plotted on the base hydrogeological cross-section (Figure B.3.6, p. 37).



These data were generated by Trow from analysis of slug test data and pressure packer tests.

During cross-examination, Mr. Sobanski (Vol. 22, p. 5111) indicated that there is some uncertainty associated with the interpretation of the slug test data. He indicated that, because of the effects of borehole smearing, the slug test data may not realistically indicate the hydraulic conductivity of the 'highly fractured shale' (Unit 4). It should be noted that reasonable agreement is achieved where similar sections of a borehole were tested by both procedures.

The only pressure packer data for Unit 4 (at HC6) yield calculated hydraulic conductivities of 3.8×10^{-6} and 1.5×10^{-5} cm/sec. CRA has calculated an average value of 9.4×10^{-6} cm/sec from these data. On the basis of the pressure packer test data, CRA concludes that the hydraulic conductivity of this unit does not exceed 1×10^{-5} cm/sec.

As detailed in the cross-examination of Mr. Feenstra (Vol. 45, p. 10204; Vol. 46, p. 10304 and Exhibit H-262), there are 13 values for the hydraulic conductivity of the 'weakly fractured shale' (Unit 3) provided by Trow (Appendix 11, Exhibit H-40A). These values

) range from '0' to a high of 8.3×10^{-5} cm/sec. Mr. Feenstra assumed a value of 1×10^{-8} cm/sec for the '0' values, and a calculated geometric mean of 6.3×10^{-7} cm/sec. This value contrasts with the 10^{-7} to 10^{-8} cm/sec that Trow (p.51, Exhibit H-40) indicates is the hydraulic conductivity of this unit. This difference indicates that the value used by Trow is 6 to 60 times lower than the mean value determined from field tests.

) The highest hydraulic conductivity values for Unit 3 were encountered at HC4. Pressure packer testing between an interval of 119.1 and 122.2 masl yielded a hydraulic conductivity of 1.6×10^{-5} cm/sec (Trow, Appendix 11, Exhibit H-40A). If only 0.3 m of this interval is considered to be fractured, then a fracture hydraulic conductivity of 1×10^{-4} cm/sec is calculated for this 0.3 m interval.

CRA concludes that the available data demonstrate that hydraulic conductivities in the 'weakly fractured shale' (Unit 3) can range over four orders of magnitude, from 10^{-8} cm/sec for the more massive rock to 10^{-4} cm/sec for a discrete fracture zone.

) It is to be noted that the highest hydraulic conductivity value determined from the pressure packer tests (1.2×10^{-3} cm/sec) was encountered in the 'moderately

) fractured shale' (Unit 2) at HC4. Similar values were also encountered in this unit at HC3.

CRA concludes that the available hydraulic conductivity data do not support the schematic hydrogeologic sequence postulated by Trow. The existence of the 'highly fractured shale' (Unit 4) with a hydraulic conductivity of 1×10^{-2} cm/sec cannot be supported.

) The low permeable strata (Unit 3) has a four order of magnitude range in the hydraulic conductivity and is significantly more variable than that indicated by Trow.

) The 'moderately weathered and fractured shale' (Unit 2) has hydraulic conductivity values that range up to 10^{-3} cm/sec which is two orders of magnitude in excess of the 10^{-5} cm/sec determined by Trow for this unit (Figure B.3.1, p.27).

B.3.5 PUMPING TESTS

CRA cannot support the conclusions developed by Trow from analyses of these pumping tests.

) Three pumping tests were conducted by Trow at this site to characterize the hydraulics of the bedrock.

The pumping wells were drilled using cable-tool methods and no stratigraphic or fracture frequency data were provided.

Two of the wells penetrate the 'moderately fractured and weathered shale' (Unit 2) while the third (PW-1) is reported to penetrate the 'highly fractured shale' (Unit 4) (p. 52, Exhibit H-40).

Exhibit H-220 (Figure B.3.2, p. 29) locates these wells in cross-section.

For the analyses of these tests no attempt is made to describe the hydraulic response to pumping in terms of a fractured rock environment. Indeed the analyses were performed on the assumption that the system acts as an equivalent porous media. No support for this assumption is offered by Trow.

Unrealistically high values for the aquifer transmissivity are calculated based on the small amount of drawdown recorded in some observation wells. CRA contends that the small amount of drawdown recorded in some observation wells reflects the lack of a direct fracture connection to the pumping well and not a highly transmissive 'aquifer'.

For each of the pumping tests, the hydraulic conductivity was calculated using an 'aquifer' thickness of 0.3 m. As detailed in cross-examination, Mr. Sobanski (Vol. 22, p. 5123) assumed that the sum of the fracture apertures exposed on the wall of the pumping well was 0.3 m which was then equated to the 'aquifer' thickness. For each test, the same 'aquifer' thickness was assumed. CRA considers it more appropriate to use the well depth below bedrock surface as the 'aquifer' thickness in order to calculate the hydraulic conductivity of that portion of the bedrock exposed to the effects of pumping.

The hydraulic conductivity of the 'highly fractured shale' (Unit 4) calculated by Mr. Sobanski reflects that of a discrete fracture zone and may be described as a fracture permeability. It does not represent the hydraulic conductivity of the complete thickness of the 'highly fractured shale' (Unit 4) as proposed by Trow.

B.3.5.1 Pumping Test PW-1

A pumping test was conducted on PW-1 which as shown on Exhibit H-220 is interpreted by Trow as being completed within the 'highly fractured shale' (Unit 4). This well is located on the property boundary near the south-east corner of the site.

This borehole is uncased over an interval of 20.4 m from 87.4 to 107.8 masl (Appendix 10, Exhibit H-40A).

It should be noted that this pumping well drew down dramatically after some 500 minutes of pumping (Appendix 10, Exhibit H-40A). It is not known if the pumping rate quoted ($2.5 \times 10^{-4} \text{ m}^3/\text{sec}$) is averaged over the test interval or is applicable to the initial 500 minutes of pumping. No drawdown data were provided for this well after 700 minutes of pumping to the end of the test, which appears to be at about 1500 minutes.

This pronounced change in drawdown occurs at an elevation of approximately 103 m which corresponds to a depth of approximately 7 m below the top of the bedrock. CRA concludes that the bulk of the water pumped from this well was derived from above this elevation i.e. from the upper 7 m of the bedrock. Below this elevation, significantly less water was produced which resulted in the observed pronounced decline in the water level in the pumping well.

The response of water levels at WAT-7 support this argument. Measurable water level declines were observed by Trow at the shallower piezometers 4, 5 and 6 at WAT-7. These piezometers lie between elevations of 97.7 and 104.4 masl (Trow, Appendix 10; Exhibit H-40A). No response was observed in the deeper piezometers 7 to 12 at this same

nest (WAT-7). These piezometers are set between elevations of 96.10 to 81.16 masl. It is to be noted that the bottom of the pumping well lies at an elevation of 87.4 masl.

A response to pumping was also observed at MB13 and MB16 indicating a variable degree of fracture connection to these observation well nests.

CRA concludes that the upper most portions of the bedrock respond more readily to pumping. CRA does not agree with Trow (p. 52, Exhibit H-40) that the results of this test demonstrate the existence of a 'highly fractured shale' (Unit 4).

A mean transmissivity value of 4.1×10^{-5} m²/sec is calculated by Trow from analysis of the pumping test data (p.53; Exhibit H-40). For those observation wells which show an obvious drawdown (OW1-86, MB16-2, PW-1, MB13-2 and MB13-3) a geometric mean transmissivity of 1.3×10^{-5} m²/sec is calculated by CRA.

*m evid.
well 5m*

For an 'aquifer' thickness of 7 m, being the thickness of the bedrock contributing the bulk of the water to the well, a hydraulic conductivity of 2×10^{-4} cm/sec is calculated by CRA. This contrasts with the value of 1.3×10^{-2} cm/sec calculated by Trow (p. 53, Exhibit H-40).

B.3.5.2 Pumping Test PW-2

This test was undertaken in a well completed in Unit 2, the 'moderately fractured and weathered shale' as defined by Trow. This pumping well (PW-2) and the adjacent observation well are shown in cross-section on Exhibit H-220.

The transmissivity values calculated by Trow (Exhibit H-40A) range from 7.4×10^{-6} to 1.8×10^{-4} m²/sec. CRA has calculated a geometric mean value of 1.1×10^{-5} m²/sec from these data. Dividing by the 8 m open interval of the pumping well gives a hydraulic conductivity of 1.4×10^{-4} cm/sec. This value supports the results of the pressure packer tests conducted on the adjacent well, HC3, which yielded hydraulic conductivities averaging 9×10^{-4} cm/sec.

(A)
It is to be noted that HC3-1 was monitored during pumping for a total drawdown of 0.25 m (Exhibit H-84). This piezometer is screened at an elevation of 110.6 m and is defined by Trow (Exhibit H-220) as lying within the 'highly fractured shale' (Unit 4). This response implies a significant degree of vertical hydraulic connection across the 'weakly fractured shale' (Unit 3) and hence a higher hydraulic conductivity than that determined by Trow for this unit.

CRA agrees with Trow that the response of the observation wells and pumping well indicates the existence of a fractured zone within the weathered bedrock in this area.

B.3.5.3 Pumping Test PW-3

The third pumping test was also conducted in the 'moderately fractured and weathered shale' (Unit 2). The resulting transmissivities range from 2.7×10^{-6} to 4.8×10^{-5} m²/sec for a geometric mean value of 1.8×10^{-5} m²/sec. For an open borehole length of 5 m a mean hydraulic conductivity of 3.6×10^{-4} cm/sec is calculated by CRA.

CRA does not agree with Trow that these data indicate a fractured zone of limited lateral extent. The drawdown pattern from this test (Appendix 10, Exhibit H-40A) appears to demonstrate the existence of a relatively continuous fracture network in the 'moderately weathered and fractured shale' (Unit 2).

B.3.5.4 Pumping Test Summary

The three pumping tests conducted all yield mean transmissivities in the order of

1×10^{-5} cm²/sec. The hydraulic conductivities calculated by CRA range from 1.4×10^{-4} (PW-2) to 3.6×10^{-4} cm/sec (PW-3) for a mean value of 2.1×10^{-4} cm/sec.

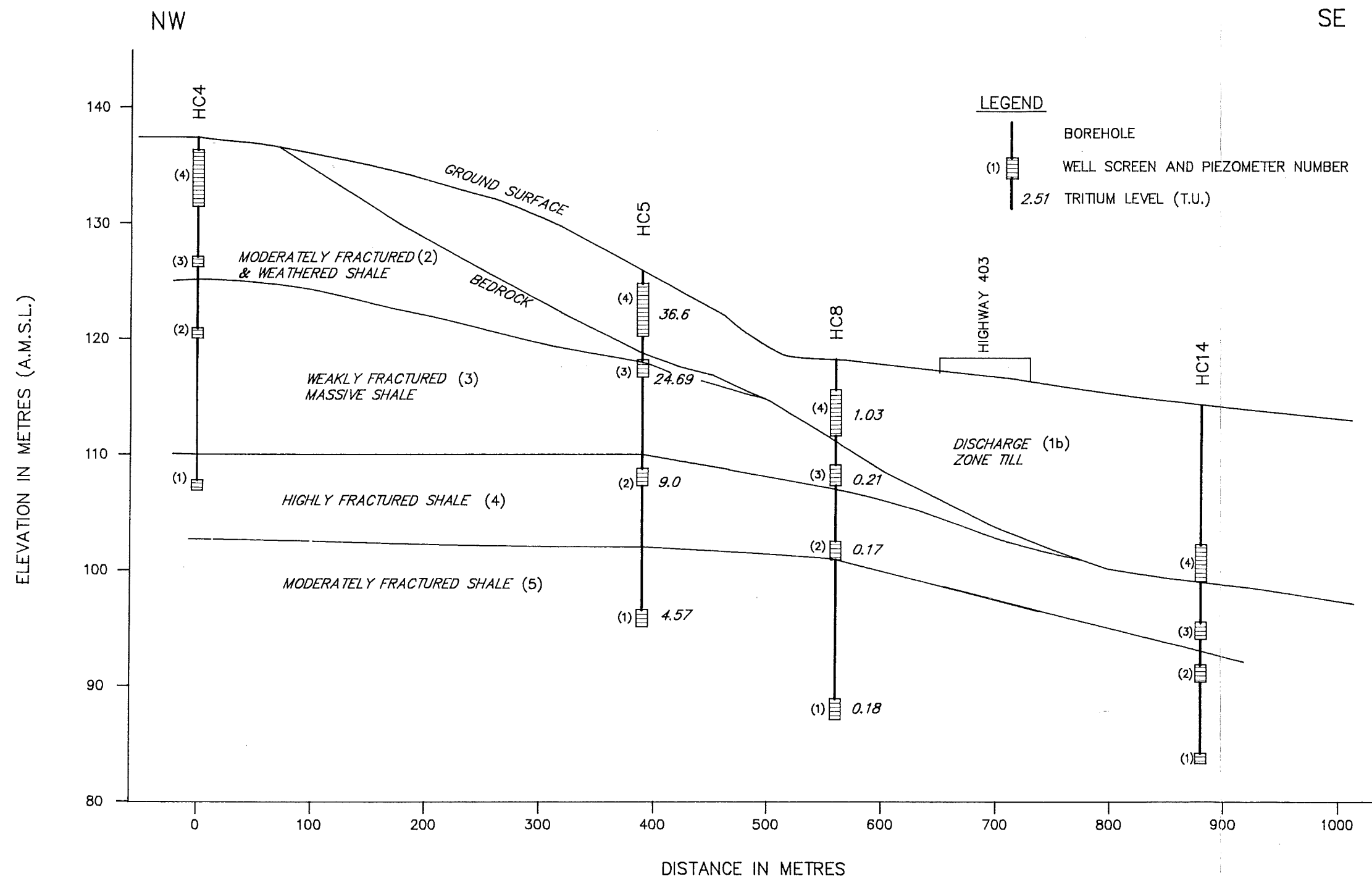
CRA concludes that all three tests demonstrated the existence of a permeable fractured zone within the uppermost portions of the bedrock.

CRA does not agree that the test results from PW-1 confirm the presence of the 'highly fractured shale' (Unit 4). Rather CRA concludes that these results demonstrate the existence of 'weathered and fractured shale' which mantles the bedrock at each of the three pumping test sites. This shale has a mean hydraulic conductivity of 2×10^{-4} cm/sec and is more extensive than the 'moderately weathered and fractured shale' (Unit 2) as interpreted by Trow (Figure B.3.1, p. 27).

B.3.6 TRITIUM DISTRIBUTION

Samples for tritium analysis were collected by Trow from well nests HC5 and HC8 and the results provided on Drawing 20 (Exhibit H-40). The resulting tritium levels are plotted by CRA in cross-section on Figure B.3.7, p. 48. The hydrostratigraphic sequence at HC5 as shown on this figure was determined from inspection of Exhibit H-220.





VERTICAL SCALE 1:400
HORIZONTAL SCALE 1:4000

CRA

SOURCE: 1) BOREHOLE LOCATIONS AND SCREEN ELEVATIONS - TROW (1986) EXHIBIT H-40A
2) HYDROSTRATIGRAPHIC UNITS - EXHIBIT H-220
3) TRITIUM PROFILES ADAPTED FROM TROW (1986) DWG. 20 EXHIBIT H-40

figure B.3.7
TRITIUM DISTRIBUTION
Site 'F' - Hydrogeology - City of Burlington

The tritium profile at HC5 shows a trend to decreasing value with depth. Post 1952 or recent water was encountered at each piezometer at HC5.

At HC8, pre 1952 water was encountered at all piezometers sampled. Local seasonal upward gradients were encountered at the upper most pair of piezometers at this nest (HC8) which may explain the absence of recent water at HC8-3.

Tritiated waters are encountered within the 'highly fractured shale' (Unit 4) at HC5. Given the lateral linear groundwater velocity of 3150 m/yr (Trow, Table 12, p.97; Exhibit H-40) for this unit it is expected that water elsewhere in this unit should also be tritiated. The absence of tritiated waters at HC8-2 demonstrates a poor lateral hydraulic connection at this elevation and does not support the existence of the 'highly fractured shale' as interpreted by Trow.

B.3.7 SUMMARY OF HYDROGEOLOGY

CRA does not agree with the hydrogeologic characterization of the site as postulated by Trow. The following are the major points of disagreement:

- 1) Hydraulic gradients are directed downwards beneath the 'highly fractured shale' (Unit 4) indicating that this unit is not present as defined.
- 2) The available hydraulic conductivity data do not indicate the existence of a 'highly fractured shale' (Unit 4) with a hydraulic conductivity of 1×10^{-2} cm/sec.
- 3) The available hydraulic conductivity data obtained from slug tests and packer tests from depths in excess of 10 m below bedrock surface (Unit 3) show a significant range in hydraulic conductivities. Values as high as 1×10^{-5} cm/sec are encountered over a 3 m interval which is inconsistent with the existence of the 'weakly fractured shale' (Unit 3) as defined.
- 4) Hydraulic conductivities up to 10^{-3} cm/sec are encountered in the 'moderately fractured and weathered shale' (Unit 2). These values are significantly higher than the 10^{-5} cm/sec assigned by Trow to this unit.
- 5) The three pumping test results demonstrate the existence of a permeable zone with a hydraulic conductivity in the order of 2×10^{-4} cm/sec at the surface of the bedrock. This zone is equivalent to the 'moderately fractured and weathered shale' (Unit 2) of Trow but is interpreted to be more extensive than that indicated by Trow.

- 6) Groundwater containing elevated levels of tritium are encountered within the 'highly fractured shale' (Unit 4) at HC5-1 but not at HC8-2 indicating a poor degree of lateral hydraulic connection. This is inconsistent with the occurrence of the 'highly fractured shale' (Unit 4) as interpreted by Trow.

B.4.0 GROUNDWATER FLOW MODELLING

B.4.1 INTRODUCTION

Trow performed a number of computer simulations of the steady state groundwater flow system at Site F. A general discussion of these simulations is provided in Exhibit H-40, (p. 61) in which a calibrated result is produced. This result is herein referred to as Trow's Model 1. The results of a number of these simulations were subsequently provided (Exhibit H-85). During cross-examination a "better fit model" was produced by Trow (Exhibit H-221). This is referred to herein as Trow's Model 2.

All these simulations were run using the model FLONET. FLONET is a steady-state, two dimensional, finite element model, which for given hydraulic conductivities and a prescribed water table, simulates infiltration, water levels and flow patterns in a porous media. Comparison of the water levels at the appropriate node with those measured at the observation wells allows for an estimation of how well the model fits.

A review by CRA of Trow's modelling shows the following:

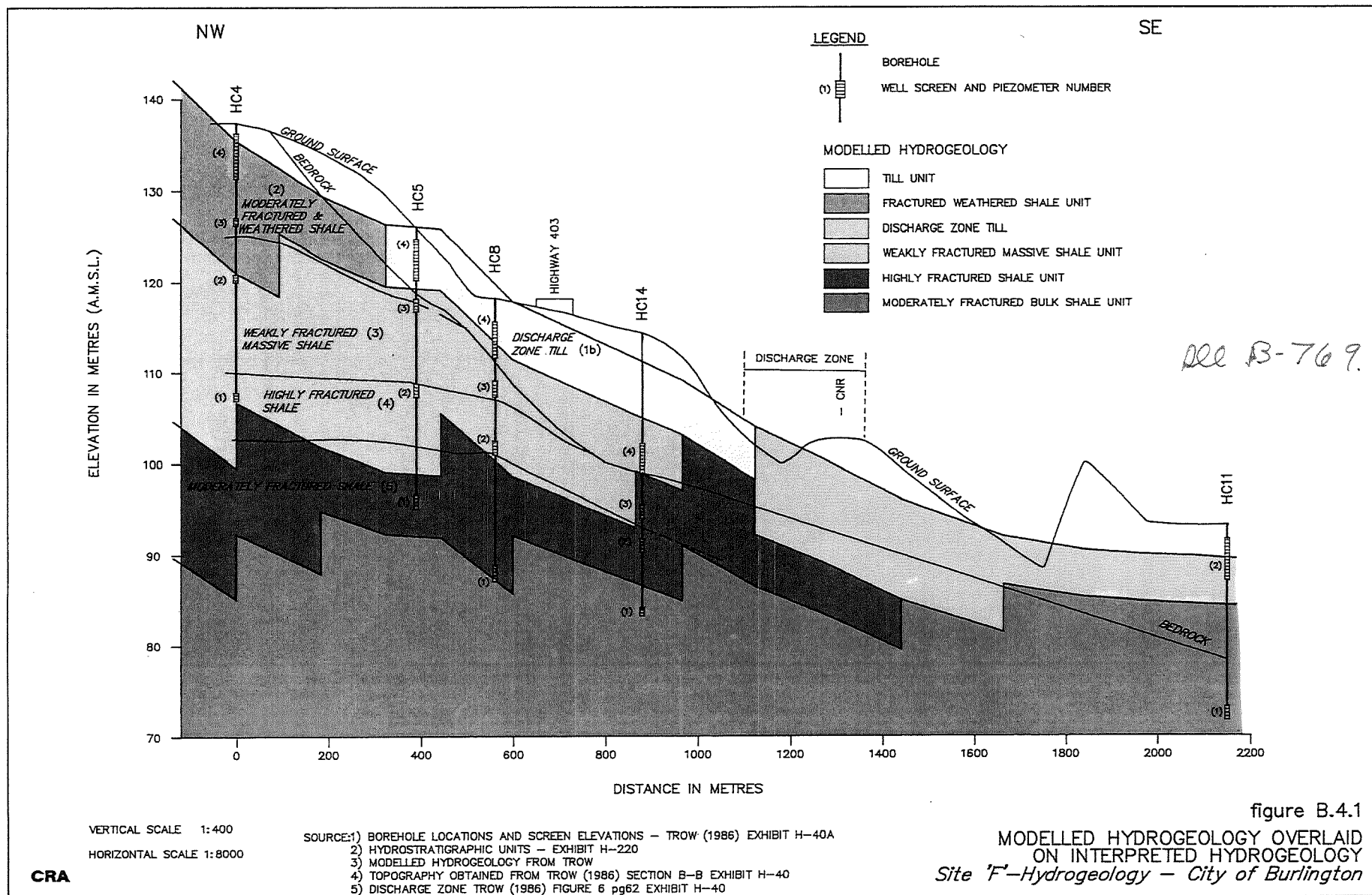
- 1) The definition of the site hydrogeology cannot be supported.
- 2) The upgradient boundary is inconsistent with the groundwater flow system proposed by Trow.
- 3) The 'reasonable' fit of both Trow's Models 1 and 2 indicates that this groundwater flow system is complex.

These three issues are discussed below.

B.4.2 HYDROSTRATIGRAPHIC DEFINITION

Inspection of Figure 6, page 62 (Trow Exhibit H-40) which presents the results of the computer simulation for Model 1 shows that the modelled flow patterns are strongly controlled by the distribution of the 'highly fractured shale' (Unit 4) and the 'discharge zone till' (Unit 1b).

The distribution of these units was examined in some detail. CRA obtained from Trow the finite element grid and the input hydraulic conductivities for both Trow's Model 1 and Model 2. Figure B.4.1 (p. 54) provides the modelled grid overlying the interpreted hydrogeology. It is to be noted from inspection of this figure that:



- 1) The modelled 'highly fractured shale' (Unit 4) lies at a lower elevation than that interpreted by Trow,
- 2) The positioning of the downgradient extent of the 'highly fractured shale' and the upgradient limit of the 'discharge zone till' direct groundwater flow in the shale to discharge to the surface at this point (i.e. the discharge zone).

No support is provided by Trow for terminating the 'highly fractured shale' in the position shown on Figure B.4.1 (p. 54). Were this unit to extend further downgradient, then the discharge zone would also be more extensive.

The infiltration is determined by summing the number of stream tubes or flow lines produced by the model. For Model 1, Trow determined an infiltration rate of 13.7 mm/yr (p. 63, Exhibit H-40) and 1.2 mm/yr for Model 2 (Exhibit H-221). CRA considers an infiltration rate of 1.2 mm/yr as unreasonably low.

As indicated by Trow, both models provide a 'reasonable' fit yet show more than an order of magnitude variation in the infiltration rate. This demonstrates that the infiltration rate is very sensitive to variations in the

hydraulic conductivity of the bedrock, more sensitive than the piezometric elevations.

Without sound interpretation of the site hydrogeology, and thus the input parameters for the model, any modelling effort is a numerical exercise only. Modelling can be used to increase the knowledge of a groundwater flow system where data is lacking, but the model must accurately represent the available field data.

B.4.3 BOUNDARY CONDITIONS

The no-flow upgradient boundary is inappropriate unless it can be demonstrated that a groundwater divide exists at the particular location or that the modelled results are insensitive to the choice of boundary condition. The choice of the no-flow boundary condition contradicts Trow (p.55, Exhibit H-40), who state

"the data also indicate a deep groundwater flow system in the bedrock that probably originates above the Niagara Escarpment north of Site F. This flow system moves under Site F at depth (greater than 30 m) and discharges in the vicinity of Lake Ontario".

B.4.4 MODEL CALIBRATION

Trow states (p. 61, Exhibit H-40)

"The resulting computer simulation represents a reasonable approximation of the field data with respect to the geology, the hydraulic conductivity, water table, and the groundwater gradient, (horizontal and vertical)"

Inspection of the calibrated water levels for Models 1 and 2 as provided by Trow (Exhibit H-221), show the following:

- 1) For the isotropic case (Model 1), five calibration points out of 17 have a difference of 3 meters or more between the simulated and measured potentiometric surface. This is a large percentage of monitoring points with a significant difference, even though the model requires a prescribed head water table boundary.
- 2) For the anisotropic case (Model 2), one out of the 17 calibration points has a difference of three meters or more indicating a better fit.
- 3) The piezometers set in the 'highly fractured shale' (Unit 4) at well nests HC4, HC5 and HC8 show poor fits with the field data for both models.
- 4) The upward (discharge) gradients observed at HC11 are not reproduced by the models.

A 3-meter difference between the predicted and measured potentiometric head values within a vertical distance of 8.38 to 28.09 m below the prescribed water table surface translates to a net difference of between 10 and 35 percent. CRA does not believe that this is a reasonable approximation of the field conditions and demonstrates that a complex fractured flow system is difficult to simulate with an equivalent porous media model.

An additional method to verify the calibration of the computer model is to examine the gradients. This is an important check as it is the gradient that provides the driving force for the movement of the groundwater and contaminant migration.

The vertical gradients calculated from Models 1 and 2 were compared with those calculated from the measured water levels of April 1986. Table B.4.1 (p. 59) summarizes these gradient data. Inspection of these data show some significant differences. At both HC5 and HC8, there is a much stronger downward gradient measured below the 'highly fractured shale' (Unit 4) than that determined by the models. The downward gradients in the bedrock at HC14 are not simulated nor is the upward (discharge) gradient at HC11.

TABLE B.4.1

SUMMARY OF VERTICAL HYDRAULIC GRADIENTS

See 128.

<u>Well Nest</u>	<u>Field Measurements (April 1986)</u>	<u>Model 1</u>	<u>Model 2</u>
HC1-85(1-2)	-0.233	-0.003	-0.028
(2-3)	-0.240	-0.394	-0.371
HC4-85(1-2)	-0.780	-1.366	-1.008
(2-3)	-0.068	0.000	-0.005
(3-4)	-0.209	-0.022	-0.007
HC5-85(1-2)	-0.328	-0.356	-0.002
(2-3)	-0.610	-0.461	-0.991
(3-4)	-0.135	-0.516	-0.057
HC8-85(1-2)	-0.444	-0.151	-0.051
(2-3)	-0.356	-0.327	-0.144
(3-4)	+0.003	-0.217	-0.191
HC14-86(1-2)	-0.001	0.00	-0.005
(2-3)	-0.084	0.00	+0.008
(3-4)	-0.035	-0.234	-0.133
HC11-85(1-2)	+0.153	-0.002	-0.020
HC16-86(1-2)	-0.098	-0.015	-0.021
(2-3)	-0.081	-0.009	-0.040

Note: Calibrated water levels from Exhibit H-221.

Vertical gradients calculated using mid-screen separation.

Both Trow's Models 1 and 2 do not reasonably reproduce the groundwater patterns in the 'highly fractured shale' (Unit 4) and the discharge gradients at HC11. This suggests that the groundwater flow system at this site is complex.

B.4.5 SENSITIVITY ANALYSIS

The calibrated isotropic model (Trow's Model 1) of the groundwater flow system was developed following more than 50 simulations. The results of some simulations are presented by Trow in Exhibit H-85.

It is not possible to evaluate the completeness of the sensitivity analysis since;

- 1) The calibrated water level data are provided in tabular form for only two of the additional simulations provided.
- 2) There is little discussion of sensitivity analyses for various boundary conditions, and hydraulic conductivities or physical dimensions of the hydrostratigraphic units.

B.4.6 SUMMARY

Review of the Trow groundwater flow modelling has revealed the following points of concern:

- 1) The modelled hydrostratigraphy does not match the interpreted hydrostratigraphy.
- 2) The downgradient limit of the 'highly fractured shale' (Unit 4) can not be supported by the field data, yet the location of the discharge zone is very dependent on the extent of this unit.
- 3) The infiltration rate is very sensitive to variations in the hydraulic conductivity, more so than the piezometric elevations.
- 4) The upgradient boundary condition is inconsistent with the flow system as described by Trow.
- 5) Both the calibrated Models 1 and 2 do not show a 'reasonable' fit of the water levels in the 'highly fractured shale' (Unit 4) nor the discharge gradient at HC11.

6) The calibrated water level data for an additional two simulations only are provided in tabular form and thus the sensitivity of the calibrated model to parameter changes cannot be adequately assessed.

CRA therefore does not agree that the site hydrostratigraphy is adequately simulated by these models.

In general, the Trow simulations indicate that the hydrogeologic system at Site F is a complex system that is very difficult to model accurately.

Because of the uncertainty of the simulated local flow regime and the sensitivity of the infiltration rate, it is improper to incorporate the results of the modelling into the definitive design of an engineered landfill system. The results can be used to evaluate the comparative effectiveness of various engineered systems, but not the actual effectiveness of the various alternatives.

B.5.0 GROUNDWATER CHEMISTRY

B.5.1 INTRODUCTION

Proposed Site F lies between the Bayview Park and Burlington Landfills. A significant amount of monitoring data has been developed for these sites. CRA has reviewed this data. In addition a review of this data was prepared by Mr. Feenstra as Exhibit H-86A. This portion of the critique focusses on the interpretation of the monitoring data.

B.5.2 BAYVIEW PARK LANDFILL

The occurrence of landfill-derived organic contamination downgradient of the Bayview Park Landfill Site has been described by Hewetson (1985, Exhibit E-214). CRA (1987) concluded from a review of this study that trace levels of synthetic volatile organic compounds principally chlorobenzene (CB), para-dichlorobenzene (PDCB) and 1,1,1-trichloroethane (TCA) extend downgradient from the Bayview Park Landfill to beyond Highway 403. However, it was noted that it was not demonstrated conclusively by Hewetson that the piezometer materials did not contribute to this distribution. These same compounds occur presently at low levels within the leachate (chlorobenzene (CB) and para-dichlorobenzene (PDCB)) while the third compound

(1,1,1-trichloroethane (TCA)) is not presently detected in Bayview Park leachate. CRA concluded from the sporadic occurrences of these compounds that contaminant migration is restricted to the interconnected fracture system, while the poorly connected fractures exhibit background quality.

CRA recommended that for the 1987 monitoring program at this site that groundwater sampling for volatile organic sampling be undertaken at selected observation wells.

In his review of this data, Mr. Feenstra concluded that the identified contamination does not result from the landfill (p. 15, Exhibit H-86A), but rather from extraneous sources such as the piezometer installations. He considered that the benzene, toluene and xylene compounds could be naturally derived from the Queenston shales. In cross-examination, Mr. Feenstra indicated that it would be conservative to consider that the chlorobenzene and dichlorobenzene data could reflect the movement of contaminants from the landfill (Vol. 46, p. 10429).

As noted in Exhibit H-86A, (Table 2 following p. 13) these contaminants are found at concentrations in excess of the maximum trip blank concentration as far downgradient as MB12 (chlorobenzene) and as far as the farthest downgradient well (MB18 (1,4-dichlorobenzene)).

As shown on Table 1 of Exhibit H-86A, both chlorobenzene and 1,4-dichlorobenzene are found in the Bayview Park Landfill leachate at concentrations as high as 51.9 and 24 ug/L, respectively.

The downgradient extent of this contamination was investigated by Trow who installed and sampled a well (HC9) located south of Highway 403 adjacent to MB18. No organic contamination was detected in this piezometer. As reported in Appendix 12 (Trow, Exhibit H-40A), the sampled piezometer was HC9-85-3. Inspection of Appendices 4 and 2 (Exhibit H-40A) shows that this well screen, set between an elevation of 96.5 to 98.0 m, is set in silt till immediately above the bedrock surface. This piezometer is not screened in the bedrock. The hydraulic gradients at HC9 are directed downwards. CRA concludes, therefore, that landfill-derived contamination migrating through the bedrock to MB18 and HC9 will not be encountered in the overlying till materials and, therefore, would not be expected to occur at HC9-85-3.

CRA (1987) concluded that organic nitrogen could be used to define the extent of groundwater contamination at the Bayview Park site. The distribution of organic nitrogen showed a contaminant plume extending downgradient to the property boundary. On the basis of this conclusion it was recommended that ammonia and total kjeldahl nitrogen (TKN), from which the organic nitrogen concentration

is determined, be included in the monitoring program for the Bayview Park Landfill. The results of this monitoring conducted during 1987 for TKN and ammonia are shown on Table B.5.1 (p. 67,68). Inspection of these data show the following:

- 1) The May 1987 results typically are significantly in excess of those obtained from the August 1987 sampling.
- 2) The background wells (HC1 and HC2) record organic nitrogen levels of 4.221 mg/L (May 1987) and 0.38 mg/L (August 1987).
- 3) Downgradient concentrations range from highs of 8.212 mg/L (HC9-3), May 1987) and 1.58 mg/L (MB13-3, August 1987) to lows of non-detect.

CRA concludes from the review of the Hewetson study (Exhibit E-214), Mr. Feenstra's witness statement (Exhibit H-86A), relevant cross-examination and the available results for the 1987 monitoring program conducted by CRA at the Bayview Park Landfill; that

- 1) Chlorobenzene (CB) and para-dichlorobenzene (PDCB) are indicators of landfill derived contamination and are detected at low levels in groundwater downgradient of the site.

Table B.5.1
Bayview Landfill Site
Groundwater Quality Data

SAMPLE (WELL DEPTH) (m TOC)	DATE	pH	Conductivity (uS)	TKN (mg/L)	Free Ammonia (mg/L) NH3	Ammonia- Nitrogen (mg/L) NH3-N	Organic Nitrogen (mg/L)
Drinking Water Objectives		6.5-8.5	--	--	--		0.15
DETECTION LIMIT		0-14	2.0	0.02	0.02	0.02	
HC1-1 (18.97)	JUN-87 AUG-87	7.16 7.07	6250 5530	2.56 0.37	0.26 0.39	0.21 0.32	2.35 0.05
HC1-1 DUP (18.97)	AUG-87	7.09	5580	0.64	0.26	0.21	0.43
HC2-1 (17.33)	JUN-87 AUG-87	7.44 7.54	1370 1763	5.16 1.00	0.94 1.06	0.77 0.87	4.39 0.13
HC2-1 DUP (17.33)	JUN-87 AUG-87	7.53	1380	2.84	0.93	0.76	2.08
HC9-1 (30.95)	JUN-87 AUG-87 DEC-87	6.49 7.09 6.93 (F)	>20000 >20000 10900 (F)	11.40 13.40 13.50	4.29 14.00 6.65	3.52 11.50 5.45	7.88 1.90 8.05
HC9-2 (15.45)	DEC-87 DEC-87	6.64 (F) 6.64 (F)	42000 (F) 42000 (F)	8.25 9.05	4.82 6.04	3.95 4.95	4.30 4.10
HC9-3 (9.78)	JUN-87 AUG-87	7.39 7.34	3080 3660	9.14 0.52	0.92 0.67	0.76 0.52	8.38
MB11-1 (3.27)	JUN-87 AUG-87	6.71 6.80	2760 2800	4.02 1.34	3.00 1.61	2.46 1.32	1.56 0.02
MB11-3 (23.21)	JUN-87 AUG-87	7.17 7.26	13800 12300	9.82 5.44	10.90 5.20	8.98 4.26	0.84 1.18
MB12-1 (5.41)	WELL BROKEN						
MB12-2 (9.71)	NOT SAMPLED						
MB13-1 (4.77)	JUN-87 AUG-87	7.12 7.07	1650 1680	1.15 1.23	0.240 0.63	0.20 0.52	0.95 0.71
MB13-2 (12.36)	DEC-87	7.02 (F)	9610 (F)	4.26	2.68	2.20	2.06
MB13-3 (20.42)	JUN-87 AUG-87 DEC-87	7.25 7.27 7.31 (F)	8650 8360 7300 (F)	9.62 5.18 3.70	9.13 3.60 2.42	7.48 2.95 1.98	2.14 2.23 1.72
MB16-1 (7.50)	JUN-87 AUG-87	6.99 7.04	2650 2580	4.26 0.31	<D.L 0.31	<DL 0.26	4.26 0.05
MB16-2 (16.22)	DEC-87	7.28 (F)	705 (F)	1.65	0.12	0.10	1.55
MB16-3 (22.30)	JUN-87 AUG-87 DEC-87	11.01 10.98 10.89 (F)	1470 2020 2020 (F)	4.00 1.45 2.28	3.02 1.51 0.98	2.48 1.24 0.80	1.52 0.21 1.48
MB17-1 (7.57)	JUN-87 AUG-87	7.45 7.12	1820 2200	8.58 2.20	0.37 2.61	0.31 2.14	8.27 0.06
MB17-3 (21.71)	JUN-87 AUG-87	6.79 6.93	18400 18760	10.70 9.94	9.17 9.34	7.52 7.66	3.18 2.28

Table B.5.1
Bayview Landfill Site
Groundwater Quality Data

SAMPLE (WELL DEPTH) (m TOC)	DATE	pH	Conductivity (uS)	TKN (mg/L)	Free Ammonia (mg/L) NH3	Ammonia- Nitrogen (mg/L) NH3-N	Organic Nitrogen (mg/L)
Drinking Water Objectives		6.5-8.5	--	--	--		0.15
DETECTION LIMIT		0-14	2.0	0.02	0.02	0.02	
MB17-3	DEC-87	6.80 (F)	26200 (F)	5.90	4.51	3.70	2.20
MB17-3 DUP (21.71)	JUN-87	6.78	16900	11.00	9.22	7.56	3.44
MB18-1 (8.12)	JUN-87	7.15	1530	3.52	0.05	0.04	3.48
	AUG-87	7.02	1630	0.38	0.32	0.26	0.12
MB18-2 (14.23)	JUN-87						
	DEC-87	8.60 (F)	13150 (F)	6.20	2.93	2.40	3.80
BLANK	JUN-87	8.14	<D.L	0.32	0.18	0.15	0.17
	AUG-87	7.10	11.0	0.03	<D.L	<DL	0.03
	DEC-87						
*2" PVC (4.82)	MAY-87	6.68	1860	1.20	1.17	0.96	0.24
	AUG-87	6.69	2060	1.46	0.32	0.26	1.20

NOTES: * - at base of landfill, adjacent to ditch
 (F) - field measurement
 <D.L - Less than Detection Limit
 > - greater than

- 2) These same compounds are at present detected at low levels in the leachate.
- 3) Organic nitrogen cannot be used to definitively delineate groundwater contamination at this site.

B.5.3 BURLINGTON LANDFILL

Contaminated groundwater is encountered downgradient of the Burlington Landfill as described by Gartner Lee (Exhibit H-83). This contamination [TCA and Trichloroethylene (TCE)] was encountered at low levels at observation well GL24-III located 25 m downgradient of the landfill. TCA was also encountered in wells described by Gartner Lee as upgradient. In addition, TCA was encountered further downgradient near Highway 403. GL (Exhibit H-83) reported long holding times between the sampling and extraction for VOC analyses of samples collected from this site. These results are therefore suspect.

Recent sampling conducted on the same wells plus two new wells installed south-east and west of the Burlington Landfill, as provided on Exhibit H-246, showed the apparent widespread occurrence of TCA. As discussed in cross-examination (Vol. 45, p. 10138), Mr. Feenstra concluded that the TCA contamination is extraneous. CRA agrees that

the widespread occurrence of low-level TCA contamination contrasts with the previously reported results (Exhibit H-83), and suggests that it is unlikely to be derived from the landfill.

The natural groundwater quality at the site results in uncertainty in interpreting the analytical data. As indicated by Gartner Lee (p. 18, 1987), the phenols, COD TKN and ammonia analytical data are compromised by natural groundwater quality at the site.

It should be pointed out that the Gartner Lee study was undertaken in order to identify a contaminant diagnostic of the leachate and further, to define the limit of contamination downgradient from the Burlington Landfill. To date, the results of this study are inconclusive. As discussed in cross-examination by Mr. Feenstra (Vol. 45, p. 10132), the well instrumentation cleaning and sampling procedures adopted by Gartner Lee are suspect and may in part contribute to the observed contamination.

B.5.4 DISCUSSION

The preceding has highlighted some of the uncertainty regarding the interpretation of the available

groundwater quality data for both the Bayview Park and Burlington Landfills; specifically that

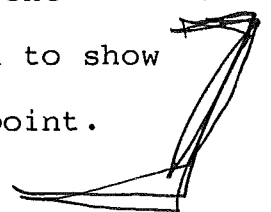
- 1) With the exception of CB and PDCB at the Bayview Park Landfill, a contaminant diagnostic of a landfill derived source has not been conclusively identified.
- 2) Many of the synthetic organic compounds detected probably result from extraneous sources.

Two other concerns were raised in cross-examination, and warrant additional discussion. Both these concerns relate specifically to the difficulty of monitoring contaminant transport in the fractured rock at these landfill sites. These concerns are:

- 1) The position of the monitoring well screens with respect to the more transmissive fractures.
- 2) The position of the monitoring well screens with respect to the deep contaminant pathway as defined by Trow.

Other than for the Trow monitoring wells constructed for the RMH, the fracture permeability characteristics of the bedrock were not determined at each borehole prior to installing the monitor well screens. It therefore cannot be demonstrated that the most significant

fractures underlying and downgradient of the landfills are screened. In addition it is not known if any fractures intersected are connected to the landfill. Thus, CRA concludes that the absence of contaminants at any one observation well cannot be confidently interpreted to show that the bedrock is not contaminated beyond this point.



Two potential contaminant flow paths have been identified by Trow (Figure 11, p. 102, Exhibit H-40):

- 1) Laterally within the 'moderately weathered and fractured shale' (Unit 2).
- 2) Vertically downwards and thence laterally within the 'highly fractured shale' (Unit 4).

The existing observation well network at both sites is inadequate to assess the contaminant distribution in the deeper flow path because the flow path is not delineated at the Burlington and Bayview Park sites. In addition the well screens may not be set at a sufficient depth to intercept this flow path.

CRA concludes that there is a significant amount of uncertainty associated with the interpretation of the available monitoring data. Specifically;

1) Much of the synthetic organic contamination in groundwater probably results from extraneous sources.

~~2)~~ The position of the observation well screens with respect to the more transmissive fractures is unknown.

3) It is not known if the fractures monitored are connected to the landfill.

4) The deeper contaminant pathway as defined by Trow is not adequately monitored.

CRA concludes that the existing data base cannot be confidently interpreted to indicate that a new landfill in the area will not result in potentially unacceptable groundwater contamination.

B.6.0 IMPACT ASSESSMENT

CRA does not agree with the methodology and conclusions used in the impact assessment undertaken by Trow (p. 73, Exhibit H-40) for the following reasons:

- ✓ 1) CRA does not agree with the site hydrostratigraphy as defined by Trow.
- 2) The values of hydraulic conductivity used in this assessment were taken from the groundwater flow model (Trow's Model 1). It does not necessarily follow that a groundwater flow model that 'reasonably' reproduces the water level measurements confirms the values of the hydraulic conductivity used.
- 3) Contaminant transport modelling in fractured rock must be based on the groundwater velocity within the fractures and not on some (average value) that reflects both the fractured and unfractured rock.

The first of the above points is discussed in detail in Section B.3 of this report.

The analysis undertaken by CRA to examine the sensitivity of the groundwater flow model (FLONET) used by Trow is reported in Section C.4 of this report. Here, CRA

demonstrates that a similar fit of the measured water levels can be obtained with different values of hydraulic conductivity. CRA, therefore, concludes that the model FLONET, as used by Trow, cannot be used to reliably determine the hydraulic conductivity of the bedrock at this site.

Active groundwater flow within the fractured rock at this site is for the most part restricted to the network of interconnected fractures. Groundwater flow in the interconnected fractures is expected to be significantly more rapid than in the adjacent rock matrix. Thus, an assessment of the rate of contaminant transport in fractured rock should be based on the rate of groundwater flow within the fractures and not on a rate of flow that is averaged over the fractured and unfractured rock.

The method used by Trow to assess the rate of groundwater flow in the 'highly fractured shale' (Unit 4) was essentially based on a fracture permeability.

In cross-examination (Vol. 22, p. 5124), Mr. Sobanski considered that the sum of the fracture widths within this 'highly fractured shale' (Unit 4) was 0.3 m. Thus, from the pumping test results, he calculated a hydraulic conductivity of 1×10^{-2} cm/sec. For the given hydraulic gradient (0.01) and effective porosity (0.01), Trow calculated a linear groundwater velocity of

3150 m/yr (Table 12, p. 97, Exhibit H-40) for the fractures within this unit.

For the 'weakly fractured shale' (Unit 3), a hydraulic conductivity of 1×10^{-7} cm/sec and an effective porosity of 0.1 was assigned by Trow (Table 12, p. 97, Exhibit H-40). These values indicate that Trow considers groundwater flow to occur within the rock matrix rather than being restricted to the fractures. The hydraulic conductivity data for Trow's Unit 3, as discussed in Section B.3 (p. 26) of this report, show that values can range up to 10^{-4} cm/sec. This demonstrates that there are zones of fracturing within this unit in which the rate of groundwater flow is expected to be significantly in excess of the 0.3 m/yr calculated by Trow (Table 12, p. 97, Exhibit H-40).

Trow did not incorporate the effects of matrix diffusion or adsorption into the contaminant transport modelling undertaken (p. 104, Exhibit H-40). As discussed by Mr. Feenstra in cross-examination, the results of the contaminant transport simulations are "probably excessively conservative" (Vol. 46, p. 10316) since the effects of matrix diffusion were not considered. This conclusion was based on the apparent absence of contaminants within the 'moderately fractured and weathered shale' (Unit 2) as compared to their occurrence predicted from the contaminant transport modelling.

The effects of matrix diffusion on contaminant transport within the fractured shale are unknown. For rapid transport in the more transmissive fractures within the 'moderately fractured and weathered shale' (Unit 2) and the 'highly fractured shale' (Unit 4), the effects of matrix diffusion are not expected to be significant.

Given that fracture zones were encountered in the 'weakly fractured shale' (Unit 3) and further, that the most rapid contaminant transport is expected in such zones, then the effects of attenuation may not have a significant effect on the time of arrival of contaminants at the base of the Unit 3.

Given that the effects of matrix diffusion are unknown, CRA considers it prudent to ignore this process when evaluating contaminant travel times in the fractured shale at this site.

The results of the transport simulations for migration within the bedrock of the deep pathway cannot be realistically compared with the available monitoring data. As discussed in cross-examination with Mr. Feenstra (Vol. 45, p. 10179) it has not been demonstrated that the monitoring well screens downgradient from Burlington or Bayview Park Landfills are set in the 'highly fractured shale' (Unit 4).

The effects of attenuation in the shallow (lateral) flow system characterized by a bulk hydraulic of 2×10^{-4} cm/sec are not expected to be pronounced.

CRA concludes that the assumptions upon which the impact assessment was based are invalid and therefore CRA cannot agree with the conclusions reached by Trow. Specifically, CRA concludes that;

- 1) The hydraulic conductivity determined from the calibrated FLONET model does not represent the hydraulic conductivity of the fracture zones through which the more rapid contaminant transport occurs.
- 2) The hydrostratigraphy of the site as evidenced from the calibrated flow model (Trow's Model 1) is not supported by the field data. Therefore the contaminant flow paths and the position of the discharge zone as delineated by Trow can not be adequately defined.
- 3) The hydraulic conductivity used by Trow for the 'weakly fractured shale' (Unit 3) is not conservative with respect to the range of values determined for this unit.

-)
- 4) The results of the simulations for the deep contaminant flow path cannot be compared to the available monitoring data as the position of the well screens relative to this pathway are not known.
- 5) The uncertainty attached to interpreting the existing monitoring data for the shallow contaminant flow path indicates that these data cannot be confidently used to confirm the results of the contaminant transport simulations.

B.7.0 LANDFILL DESIGN CONSIDERATIONS

The landfill design proposed by Trow is based upon the following hydrogeologic interpretation.

1) The 'weakly fractured massive shale' (Unit 3) limits the vertical migration of leachate. ✓
✓

2) Leachate migrating laterally within the 'moderately fractured and weathered shale' (Unit 2) will discharge on-site. ✓

CRA does not support the latter of these interpretations. Therefore, CRA cannot support the landfill design as proposed.

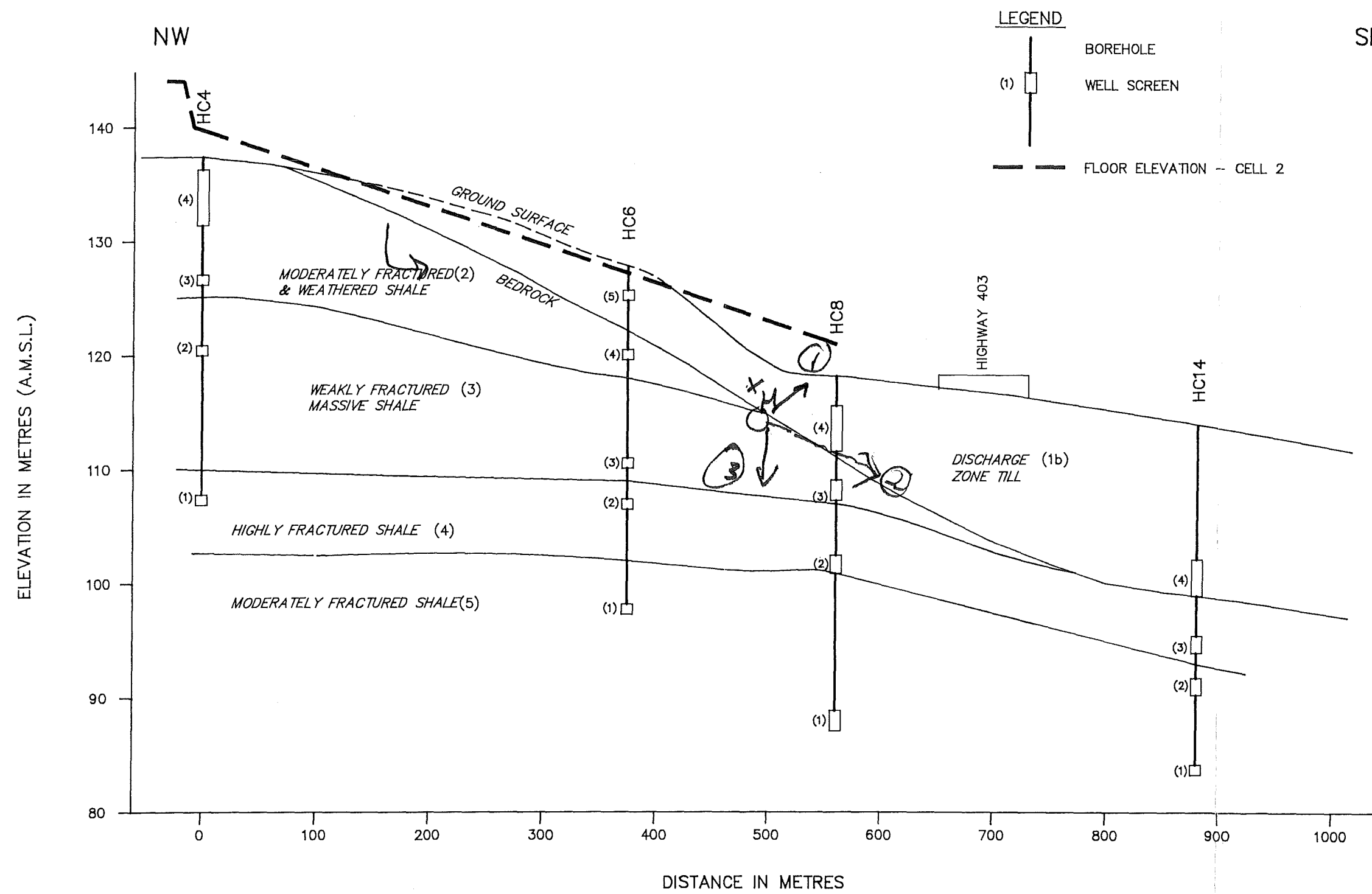
Trow states that the 'weakly fractured shale' (Unit 3) has a low hydraulic conductivity of 'no more than 1×10^{-7} cm/sec' (p. 123, Exhibit H-40). As discussed in Section B.3 of this report (p. 26), the hydraulic conductivity data provided by Trow for this unit range from an assumed 1×10^{-8} cm/sec to a high of 8.3×10^{-5} cm/sec with a geometric mean of 6.3×10^{-7} cm/sec (Exhibit H-262). As shown in Section B.3 of this report (p. 39), a fracture hydraulic conductivity of 1×10^{-4} cm/sec is calculated by CRA

from data provided by Trow for a 0.3 m thickness within this unit. CRA has demonstrated that this unit has hydraulic conductivities ranging over four orders of magnitude.

It is expected however that the 'moderately fractured shale' (Unit 3) will limit the vertical migration of leachate to some extent. A significant portion of the contaminants is expected to migrate laterally in the overlying 'moderately fractured and weathered shale' (Unit 2).

The base elevation of the proposed landfill excavation, as detailed by Trow (Drawing 5-r, Exhibit H-42B) was plotted by CRA on Trow's interpreted hydrostratigraphy (Figure B.7.1, p. 82). This figure demonstrates that a significant thickness of the 'moderately fractured and weathered shale' (Unit 2) remains below the excavation.

Given that the hydraulic conductivity of this unit as determined from pumping tests is 2×10^{-4} cm/sec, the weathered and fractured shale is expected to act as an underdrain for the landfill.



VERTICAL SCALE 1:400
HORIZONTAL SCALE 1:4000

CRA

SOURCE: 1) BOREHOLE LOCATIONS AND SCREEN ELEVATIONS - TROW (1986) EXHIBIT H-40A
2) HYDROSTRATIGRAPHIC UNITS - EXHIBIT H-220
3) CELL FLOOR ELEVATION - TROW DRAWING 5-r EXHIBIT H-42B

figure B.7.1
CROSS-SECTION OF PROPOSED LANDFILL
Site 'F' - Hydrogeology - City of Burlington

Trow did consider that this unit (Unit 2) could provide for the lateral migration of leachate below the landfill. However, on the basis of the computer modelling results (Figure 21, p. 125, Exhibit H-40), Trow concluded that this leachate would discharge to surface at the toe of the landfill immediately upgradient of HC8 where it could be collected. CRA does not agree with this conclusion for the following reasons:

- 1) As interpreted by CRA from the results of the pumping tests (Section B.3, page 40), weathered and permeable bedrock mantles much of the site and is expected to provide for lateral groundwater flow beneath the till. In fact, the presence of till beneath the toe of the landfill will restrict the upward migration of leachate to the surface of this point.
- 2) Trow's model of the landfill (Figure 21, p. 125, Exhibit H-40) appears to show the landfill as excavated through the Unit 2 and lying directly on the 'weakly fractured shale' (Unit 3) as no lateral flow is seen immediately beneath the landfill. The discharge shown between HC5 and HC8 therefore most probably reflects lateral movement of leachate within the landfill.

As demonstrated in Section C.4 of this report, the FLONET model as applied by Trow cannot be used to estimate the infiltration or discharge from this site. CRA, therefore, does not agree with the estimates provided by Trow (p. 124, Exhibit H-40) for the infiltration into or discharge from this groundwater flow system.

Trow also provides an estimate of the capture efficiency of 93 to 96 percent for the proposed leachate collection system (p. 127, Exhibit H-40). Again, CRA cannot agree with this estimate as it is based on an estimated infiltration developed from the FLONET model.

Gartner Lee (1986 Exhibit H-83) used the FLONET model in an attempt to assess the toe drain efficiency at the Burlington Landfill site. From this study, they concluded that:

"Unfortunately our results and the results of all other Darcy's Law calculations are dependent upon the hydraulic conductivity of the shale bedrock which is variable over several orders of magnitude. As a result our estimates of the toe-drain efficiency in collecting leachate vary from less than 30% to greater than 90% depending on the "average" shale hydraulic conductivity used in the calculation."

(p. 17, Exhibit H-83)

Given that the exfiltration estimate was based on the results of the FLONET model and further that CRA considers that this model cannot be realistically used at this site to estimate infiltration, then efficiency of the leachate collection system cannot be reliably determined.

CRA does not support the landfill design as proposed.

B.8.0 MONITORING PROGRAM

The ongoing monitoring programs at both the Bayview Park and Burlington Landfills highlight the difficulties in monitoring a fractured rock environment. Specifically, the difficulty relates to the positioning of a monitor well screen with respect to the more transmissive fractures that are connected to the landfill, the uncertainty attached to the interpretation of the existing data and the naturally poor groundwater quality.

These concerns were considered in cross-examination by Mr. Feenstra, (Vol. 46, p. 10450) who suggested that the methodology of observation well
installation designed to address these difficulties should consist of the following:

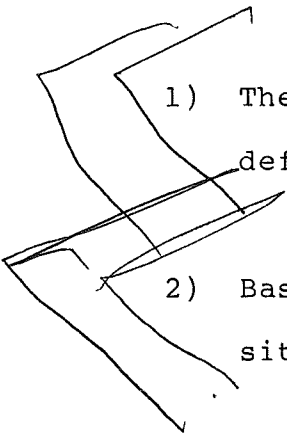
- 1) Stainless steel materials steam cleaned on site.
- 2) Analysis of rinse water following cleaning to confirm the absence of contaminants on the well construction materials.
- 3) Core the deepest borehole at each well nest.
- 4) Examine the core to identify zones of fractures or discontinuities.

- 5) Conduct downhole packer tests to define zones of high hydraulic conductivity.
- 6) Conduct this testing by pumping water from the borehole rather than introducing water into the fracture zones.
- 7) Instrument each of the high hydraulic conductivity zones defined.
- 8) Conduct pumping tests to demonstrate the degree of interconnection between fracture zones in the various observation wells.
- 9) Existing observation wells should not be incorporated into the monitoring program.

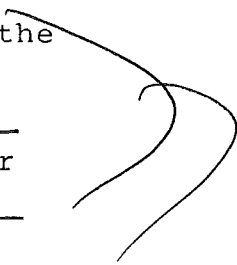
It is understood that each monitoring well would be installed in separate boreholes.

As recognized by Mr. Feenstra, the above approach to monitoring "is actually quite different than the approach taken at most disposal sites in Ontario" (Vol. 46, p. 10454), and further, "that it would probably be unlike any other monitoring program developed for a landfill in southern Ontario" (Vol. 436, p. 10460).

CRA considers that the monitoring program suggested by Mr. Feenstra is aimed at defining the contaminant flow paths and the hydraulic characteristics of those flow paths. This is inconsistent with the conclusions of the hydrogeologic investigations conducted by Trow; namely that;

- 
- 1) The hydraulic characteristics of those flow paths are defined.
 - 2) Based on this definition, the impact assessment shows the site meets acceptable criteria.

CRA concludes that the contaminant flow paths are not defined, nor is the rate of groundwater flow in such zones defined. The monitoring program suggested by Mr. Feenstra is aimed at additional characterization of the site and, therefore, should be undertaken prior to an assessment being made of the suitability of this site for landfilling.



B.9.0 CONTINGENCIES

B.9.1 INTRODUCTION

Contingency control options are required in the event that either/or:

- 1) The engineered landfill fails to perform as designed.
- 2) The natural geologic system does not provide sufficient attenuation to reduce contaminant concentrations to levels considered acceptable for off-site migration.

Groundwater quality monitoring is required to assess the performance of the engineered system and the natural geologic environment. As discussed in Section B.8 of this report, the monitoring conducted to date at Bayview Park and Burlington Landfills demonstrates the considerable difficulty in monitoring this site. CRA concludes, therefore, that groundwater monitoring to date at this site does not provide sufficient assurance that the performance of both the engineered landfill and the geologic system can be adequately assessed.

Trow (p. 136, Exhibit H-40) discuss two contingency options for this site, namely a perimeter tile collector and purge wells.

B.9.2 PERIMETER TILE COLLECTOR SYSTEM

In the event of leachate seeps occurring at the toe of the proposed landfill, a perimeter collection system could be installed to effect control. The need for such a system would be based on the visual inspection of leachate seeps. Such a system was recently installed at the Bayview Park Landfill and to date is providing effective control of the leachate seeps (CRA 1987).

It is emphasized that the perimeter tile collection system considered by Trow will not address groundwater contamination in the 'weathered bedrock' (Unit 2) underlying the till at the toe of the landfill.

B.9.3 PURGE WELLS

Based on CRA's interpretation of the three pumping tests conducted by Trow, purge wells are expected to be able to provide for control of contaminants migrating laterally in the weathered and fractured bedrock mantle ie. the shallow contaminant flow path defined by Trow (Figure 11, p. 102, Exhibit H-40). However, the number and placement of such wells has not been determined. Given the heterogeneity of the fractured bedrock, CRA considers that

the effectiveness of such a purge well system may not be easily demonstrated.

CRA concluded in Section B.3 of this report that the pumping tests conducted by Trow do not demonstrate the existence of the 'highly fractured shale' (Unit 4).

The test at PW-1 did, however, demonstrate a variable degree of lateral fracture connection within the bedrock. However, the yield of this well was low and it can not be demonstrated that the well could sustain its initial pumping rate.

CRA concludes that the pumping tests to date do not demonstrate that contaminant migration in the deep flow system ie. migrating laterally in the 'highly fractured shale' (Unit 4) can be contained by a purge well system.

PART C: INTERPRETATION OF SITE F HYDROGEOLOGY

C.1.0 INTRODUCTION

This part provides CRA's interpretation of the hydrogeology of Site F and an assessment of the suitability of this site for landfilling. In addition to the Trow report reviewed herein, this interpretation is based on a review of all available reports as well as experience gained by CRA at this site.

This interpretation of the hydrogeology of Site F is based on all the available geologic and hydrogeologic data available. These data were developed from the drilling, testing and sampling of some 70 observation wells and well nests.

C.2.0 GEOLOGY

Bedrock at the site is comprised of fractured shales of the Queenston Formation. The fracture pattern is complex, although there is a general trend to decreasing fracture frequency with depth. The uppermost surface of the bedrock is extensively weathered and fractured.

Where exposed, the rock is seen to consist of weathered and fractured shales. Fracture spacing on weathered exposures is generally less than 5 cm. The fractures are generally subhorizontal although some steeply dipping fractures have been observed.

Morrison Beatty (1980) describes "a network of vertical joints and fractures" (p. 22) and groundwater seepage was also observed along bedding planes, especially associated with the green limestone layers in the rock.

Hewetson (1985, Exhibit E-214) describes horizontal fracturing in the upper 47 m of the Queenston Formation as "quite extensive" (p. 13). Fracture spacings were reported to range from 5 to 35 cm (p.4).

C.3.0 HYDROGEOLOGY

C.3.1 INTRODUCTION

local flow

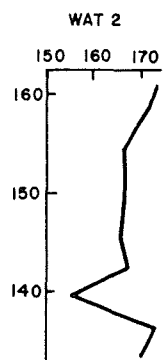
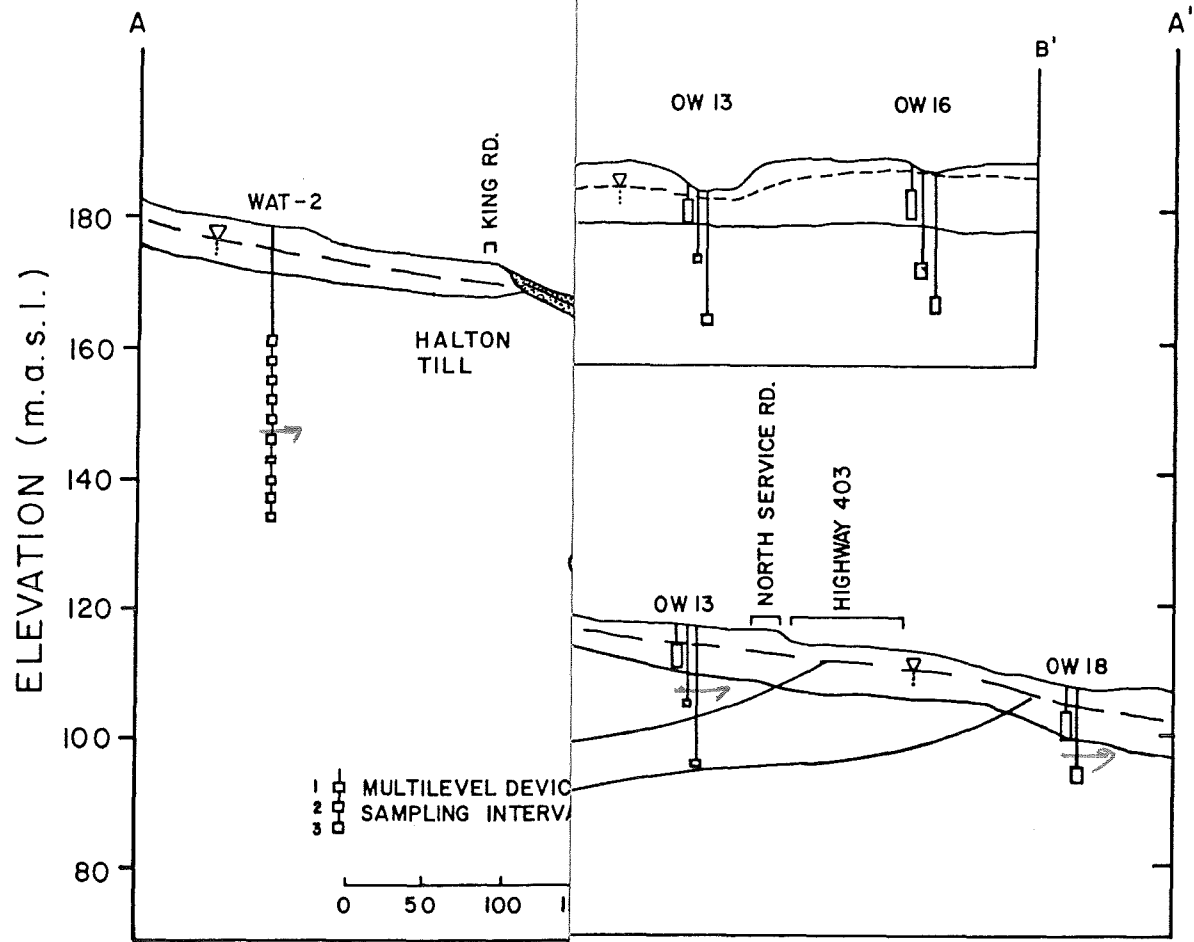
The bedrock at the site is a complex variably fractured mass, in which the orientation, distribution and the degree of interconnection of the fractures control the rate and direction of groundwater flow. Flow paths in this system cannot be defined with confidence. Horizontal groundwater flow is expected to dominate in the upper weathered and fractured portion of the rock mass. Below this horizon downward groundwater flow generally dominates although the pattern of flow is expected to be complex.

C.3.2 WATER LEVELS

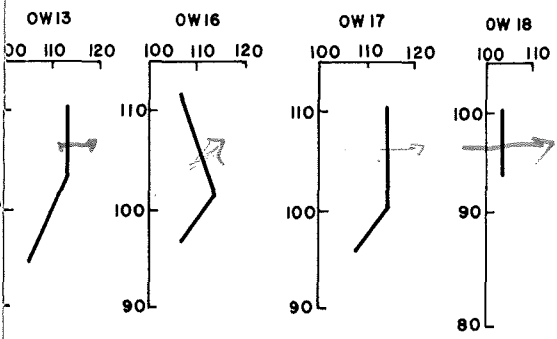
The water level data reveals a complex pattern of ^{*local*} groundwater flow consistent with a variably fractured shale bedrock environment.

The water level data generated by Hewetson (1985, Exhibit E-214) shows that on the portion of the site lying downgradient from the Bayview Landfill (Figure C.3.1, p. 95), groundwater flow is generally directed downwards.

EX. B 617-95



HYDRA



petre

*rest
Elamers X*

figure C.3.1

GROUNDWATER POTENTIAL DISTRIBUTION
'F'-Hydrogeology - City of Burlington

CRA

The hydraulic head profile at each piezometer nest is also shown on this figure. This figure shows that on a small scale, this distribution of hydraulic head is far from uniform. At WAT-4 for example, there is a low pressure zone at an elevation of 100 masl which is expected to define a discrete zone of active groundwater flow. A second low pressure zone is seen at an elevation of about 115 masl in this same multilevel piezometer nest. This pattern of water levels is not reflected in the adjacent piezometer nests, WAT-3 and WAT-5. The hydraulic head profile at WAT-5 is much more uniform than those at the adjacent multilevel piezometers, WAT-4 and WAT-7. It should be noted that the water levels in some piezometers in the WAT-4 nest lie at a lower elevation than that of the nearest downgradient nest, WAT-5.

More recent water level measurements taken by Trow (Appendix 5, Exhibit H-40A) and reproduced as Figure B.3.5 (p. 34) herein show, for WAT-3, WAT-5 and WAT-7, similarly complex pressure head profiles.

On the basis of these hydraulic head profiles, CRA concludes that the ^{local at the site} groundwater flow system is complex. While it is recognized that the pattern of groundwater flow is generally downwards and south-eastwards, there are significant local reversals in gradients. The flow path by which groundwater migrates through the fractured rock has not been defined.

C.3.3 HYDRAULIC CONDUCTIVITY

2 The reported hydraulic conductivity values range over eight orders of magnitude from a low of 10^{-10} cm/sec to a high of 10^{-2} cm/sec. The spatial distribution of these hydraulic conductivity values is erratic and is consistent with a variably fractured rock mass.

Hydraulic conductivity values determined from single well response tests range from 2.8×10^{-10} to 2.0×10^{-4} cm/sec. Tables C.3.1a, C.3.1b and C.3.1c (p. 99-105) present the hydraulic conductivity data generated by Hewetson (Exhibit E-214), Morrison Beatty (1980, 1981) and Gartner-Lee (Exhibits H-217 and H-83), respectively.

The data provided by Hewetson range from 2.3×10^{-10} cm/sec to a high of 1.4×10^{-4} cm/sec at WAT-5-1. A trend in the distribution of hydraulic conductivity values with depth is not apparent from these data. The pattern at WAT-5 shows a consistent reversal in the values ranging over five orders of magnitude. At WAT-4, the immediate upgradient well, the hydraulic conductivity data is more uniform than that obtained at WAT-5. It is to be noted that the hydraulic head profile is more uniform at WAT-5 than at WAT-4.

TABLE C.3.1a

HYDRAULIC CONDUCTIVITY AND GROUNDWATER VELOCITY
QUEENSTON SHALE - (HEWETSON, 1985)

<u>MONITORING LOCATION</u>	<u>DEPTH BELOW BEDROCK SURFACE (m)</u>	<u>BULK HYDRAULIC CONDUCTIVITY* (cm/sec)</u>	<u>FRACTURE POROSITY</u>	<u>AVERAGE LINEAR GROUNDWATER VELOCITY (m/yr)</u>
WAT2-2	13.8	2.0×10^{-9}	.00002	1.4
WAT2-5	22.7	3.0×10^{-9}	.00001	2.6
WAT2-8	31.5	4.0×10^{-9}	.000009	7.2
WAT3-9	28.0	1.3×10^{-9}	.00001	2.0
WAT3-11	33.3	4.0×10^{-9}	.00001	5.8
WAT4-3	8.3	3.5×10^{-10}	.000007	0.8
WAT4-7	18.2	2.7×10^{-10}	.000009	0.5
WAT4-10	26.9	2.3×10^{-10}	.000009	0.4
WAT4-12	32.2	9.8×10^{-8}	.00003	47.0
WAT5-1	2.1	1.8×10^{-4}	.001	3000.0
WAT5-5	12.6	3.7×10^{-9}	.00003	2.1
WAT5-8	20.6	1.4×10^{-6}	.0002	149.0
WAT5-10	25.8	1.4×10^{-8}	.00001	17.0
WAT7-5	11.4	1.1×10^{-8}	.00004	4.5
WAT7-9	22.1	3.1×10^{-10}	.00001	0.5
WAT7-11	27.4	2.1×10^{-8}	.00003	13.0

SOURCE: Hewetson (1985 - Exhibit E-214)

* Determined from single well response tests

TABLE C.3.1b

HYDRAULIC CONDUCTIVITY - QUEENSTON SHALE
 (MORRISON BEATTY - 1980, 1981)

OBSERVATION WELL NO.	BOREHOLE DEPTH B.G.L. (m)	HYDRAULIC CONDUCTIVITY* K_h (cm/sec.)
OW1-1	1.52	--
OW1-8	9.14	{ 5.0 x 10 ⁻⁵ 6.7 x 10 ⁻⁸ 2.4 x 10 ⁻⁸
OW1-14	15.24	
OW1-20	22.86	
OW2-1	1.52	--
OW2-8	8.84	1.0 x 10 ⁻⁵
OW2-14	15.24	1.4 x 10 ⁻⁸
OW2-21	22.86	3.8 x 10 ⁻⁸
OW3-1	1.52	--
OW3-8	8.84	7.8 x 10 ⁻⁷
OW3-14	15.24	2.0 x 10 ⁻⁶
OW3-19	22.55	5.6 x 10 ⁻⁶
OW3-20	24.38	< 1 x 10 ⁻⁸
OW4-9	10.52	2.0 x 10 ⁻⁴
OW4-15	15.24	2.0 x 10 ⁻⁹
OW5-8	9.14	1.2 x 10 ⁻⁴
OW6-27	30.48	9.1 x 10 ⁻⁶
OW7-21	24.38	6.2 x 10 ⁻⁶

continued....

TABLE C.3.1b

HYDRAULIC CONDUCTIVITY - QUEENSTON SHALE
 (MORRISON BEATTY - 1980, 1981)

OBSERVATION WELL NO.	BOREHOLE DEPTH B.G.L. (m)	HYDRAULIC CONDUCTIVITY* K_h (cm/sec.)
OW8-15	18.29	1.6×10^{-6}
OW9-8	12.19	1.9×10^{-5}
OW10-5	6.10	1.1×10^{-5}
OW11-3	4.57	1.0×10^{-4}
OW11-15	15.24	--
OW11-22	22.86	1.4×10^{-5}
OW12-4	4.57	--
OW12-10	10.67	2.0×10^{-6}
OW12-19	22.86	6.0×10^{-8}
OW13-4	4.57	6.4×10^{-7}
OW13-10	10.67	1.0×10^{-5}
OW13-19	22.86	4.8×10^{-6}
OW14-22	22.86	4.1×10^{-6}
OW15-9	13.71	2.5×10^{-5}
OW15-18	19.81	9.0×10^{-6}
OW15-27	28.96	--

continued....

TABLE C.3.1b

HYDRAULIC CONDUCTIVITY - QUEENSTON SHALE
(MORRISON BEATTY - 1980, 1981)

OBSERVATION WELL NO.	BOREHOLE DEPTH B.G.L. (m)	HYDRAULIC CONDUCTIVITY* K_h (cm/sec.)
OW16-7	7.62	2.14×10^{-6}
OW16-17	16.76	2.35×10^{-5}
OW16-21	22.86	5.9×10^{-7}
OW17-7	7.62	1.3×10^{-6}
OW17-16	16.76	6.1×10^{-7}
OW17-22	22.40	1.26×10^{-6}
OW18-7	7.62	$\left\{ \begin{array}{l} 9.27 \times 10^{-6} \\ 7.0 \times 10^{-7} \\ 2.24 \times 10^{-5} \end{array} \right.$
OW18-13	13.72	
OW18-20	20.42	

SOURCE: Morrison Beatty (1980, 1981)

Notes:

- These wells are numbered MB1-MB20 in later reports
- The suffix following the well number indicates the approximate depth below ground at which the observation well screen is set
- * Horizontal hydraulic conductivity determined from single well response tests

TABLE C.3.1c

HYDRAULIC CONDUCTIVITY - QUEENSTON SHALE
(GARTNER LEE 1984, 1986)

<u>BOREHOLE</u>	<u>DEPTH (m)</u>	<u>HYDRAULIC CONDUCTIVITY (cm/sec)</u>	
		<u>PRESSURE PACKER TESTS</u>	<u>SINGLE WELL RESPONSE TESTS</u>
1	12.8-13.4		1.22×10^{-6}
	16.8-18.3	1.49×10^{-5}	
	16.8-18.3	1.60×10^{-5}	
	17.7-18.3		2.45×10^{-6}
5	6.9-7.5		9.40×10^{-7}
	7.6-9.1	1.33×10^{-5}	
	7.6-9.1	1.45×10^{-5}	
	10.7-12.2	8.50×10^{-6}	
	10.7-12.2	9.10×10^{-6}	
	11.3-12.0		7.24×10^{-7}
12	6.1-7.6	5.20×10^{-6}	
	6.4-7.0		1.11×10^{-6}
	9.1-10.7	1.00×10^{-6}	
	9.1-10.7	1.10×10^{-6}	
	10.6-12.2	1.80×10^{-6}	
	10.6-12.2	1.10×10^{-6}	
	11.6-12.2		1.05×10^{-6}

continued....

TABLE C.3.1c

HYDRAULIC CONDUCTIVITY - QUEENSTON SHALE
(GARTNER LEE 1984, 1986)

<u>BOREHOLE</u>	<u>DEPTH (m)</u>	<u>HYDRAULIC CONDUCTIVITY (cm/sec)</u>	
		<u>PRESSURE PACKER TESTS</u>	<u>SINGLE WELL RESPONSE TESTS</u>
18	10.21-10.67		7.00×10^{-7}
	15.70-16.15		1.62×10^{-7}
19	10.52-10.97		9.45×10^{-8}
20	10.21-10.67		5.10×10^{-5}
	16.09-16.55		5.13×10^{-8}
21	17.83-18.29		1.40×10^{-7}
22	15.70-16.15		2.48×10^{-8}
23	17.60-18.20		2.00×10^{-6}
24	5.00- 6.20		4.00×10^{-7}
	11.80-12.40		9.00×10^{-6}
	17.70-18.30		4.00×10^{-6}
	17.70-18.30		2.00×10^{-6}
25	5.00- 6.20*		3.00×10^{-7}
	11.00-12.40		5.00×10^{-6}

continued....

TABLE C.3.1c

HYDRAULIC CONDUCTIVITY - QUEENSTON SHALE
(GARTNER LEE 1984, 1986)

<u>BOREHOLE</u>	<u>DEPTH (m)</u>	<u>HYDRAULIC CONDUCTIVITY (cm/sec)</u>	
		<u>PRESSURE PACKER TESTS</u>	<u>SINGLE WELL RESPONSE TESTS</u>
	17.90-18.50		2.00×10^{-6}
	17.90-18.50		4.00×10^{-4}
26	4.90- 6.10*		3.00×10^{-4}
	11.80-12.40		8.00×10^{-6}
	17.90-18.50		2.00×10^{-5}
27	5.80- 7.00*		1.00×10^{-5}
28	7.60- 8.80*		1.00×10^{-6}
29	7.80- 9.00*		1.00×10^{-6}
30	6.90- 8.10*		2.00×10^{-7}

Notes:

* - Shale/Till Interface

Sources:

Boreholes 1 to 22, Gartner Lee (1984, Table 9, Exhibit H-217)

Boreholes 23 to 31, Gartner Lee (1986, Table 1, Exhibit H-83)

It is not possible to describe the degree of openness of the fractures encountered. Estimates of the fracture porosity were produced by Hewetson (Exhibit E-214). The resulting fracture apertures ranged from 1 to 50 um which correspond to fracture porosities of 0.000007 to 0.001.

Using this fracture porosity data and the hydraulic conductivity data, Hewetson (Exhibit E-214) calculated the average linear groundwater velocities at a number of the observation wells. These data are also provided on Table C.3.1a (p. 98) and show a range of less than 1 m/yr to a high of 3,000 m/yr. This is an extreme range and reflects the complexity of the groundwater flow system at this site.

The data generated for the same area by Morrison Beatty (1980, 1981) range over five orders of magnitude from a high of 2.0×10^{-4} cm/sec to 2.0×10^{-9} cm/sec. At some boreholes (e.g. OW1, OW2 and OW3) there is a trend to decreasing hydraulic conductivity values with depth while at others (e.g. OW18) the trend is reversed.

The distribution of the higher hydraulic conductivity values is somewhat irregular and cannot be

correlated between boreholes. The higher values are generally found at shallower depths in the rock, although high values, in excess of 1×10^{-5} cm/sec are found at depths in excess of 20 m at OW11 and OW18.

As discussed in Part B of this report (Section B.3, p. 36) the bedrock hydraulic conductivity data provided by Trow (Exhibit H-40A) ranged from an assumed low of 1×10^{-8} cm/sec to a high of 1×10^{-2} cm/sec. The pressure packer tests resulted in hydraulic conductivities as high as 1.2×10^{-3} cm/sec (Trow, Appendix 11, Exhibit H-40A). A hydraulic conductivity of 1×10^{-2} cm/sec was calculated from the pumping test results by Trow for an assumed 0.3 m thickness of fractured rock.

Both pressure packer tests and single well response tests were undertaken to determine the hydraulic conductivity of the shale beneath the Burlington Landfill (GLA 1984, Exhibit H-217). A smaller range of values were obtained here (Table C.3.1c, p. 102). Hydraulic conductivity values determined from single well response tests ranged from 5.1×10^{-5} to 2.48×10^{-8} cm/sec while those for the pressure packer tests ranged from 1.6×10^{-5} to 1.0×10^{-6} cm/sec. Good agreement was achieved where both tests were conducted over the same portion of a borehole.

It is to be noted that the pressure packer test results provided by Trow (Appendix 11, Exhibit H-40A) range from '0' (i.e. less than 1×10^{-8} cm/sec to a high of 1.2×10^{-3} cm/sec).

More recent testing (GLA 1986b, Exhibit H-83) undertaken at this site indicates a low reproducibility of duplicate single well response tests conducted on some boreholes.

It was emphasized by Gartner Lee that bedrock permeability was of secondary importance in assessing the suitability of the expanded Burlington Landfill. Landfilling is now occurring on the till materials which provides natural attenuation for that portion of the leachate that may bypass the engineered leachate collection system.]

CRA concludes that the available hydraulic conductivity data show a considerable variation which reflects the complexity of the hydrogeology at this site. It should also be noted that higher hydraulic conductivities are reported for Site F than for the Burlington site.

C.3.4 TRITIUM DISTRIBUTION

The distribution of tritium downgradient from Bayview Park as determined by Hewetson (1985, Exhibit E-214) is shown on Figure C.3.2, (p. 109). A trend to decreasing values at depth is obvious at two boreholes (OW11 and OW18). The reverse pattern is apparent at OW12 and WAT-7. Tritiated groundwater is encountered at depth at WAT-3 and WAT-5 while at a similar elevation in WAT-4, which lies between these two piezometers, non-tritiated waters are encountered.

Low tritium values indicate that sections of the rock mass are hydraulically isolated from infiltrating rainwater and circulating groundwater. This interpretation is consistent with that inferred from the hydraulic head profiles and the distribution of hydraulic conductivity values: namely that discrete zones of active groundwater flow are set in a matrix of low permeable rock. It is not possible, based on the tritium distribution, to delineate these discrete zones of active groundwater flow.

The wide spread occurrence of tritiated groundwaters beneath the site indicates a hydraulically active flow regime, one in which groundwater, recharged by infiltrating precipitation since 1952, has migrated to a depth of at least 35 m. This indicates a significant degree of fracture connection between the surface and the deep rock.]

B 617-109

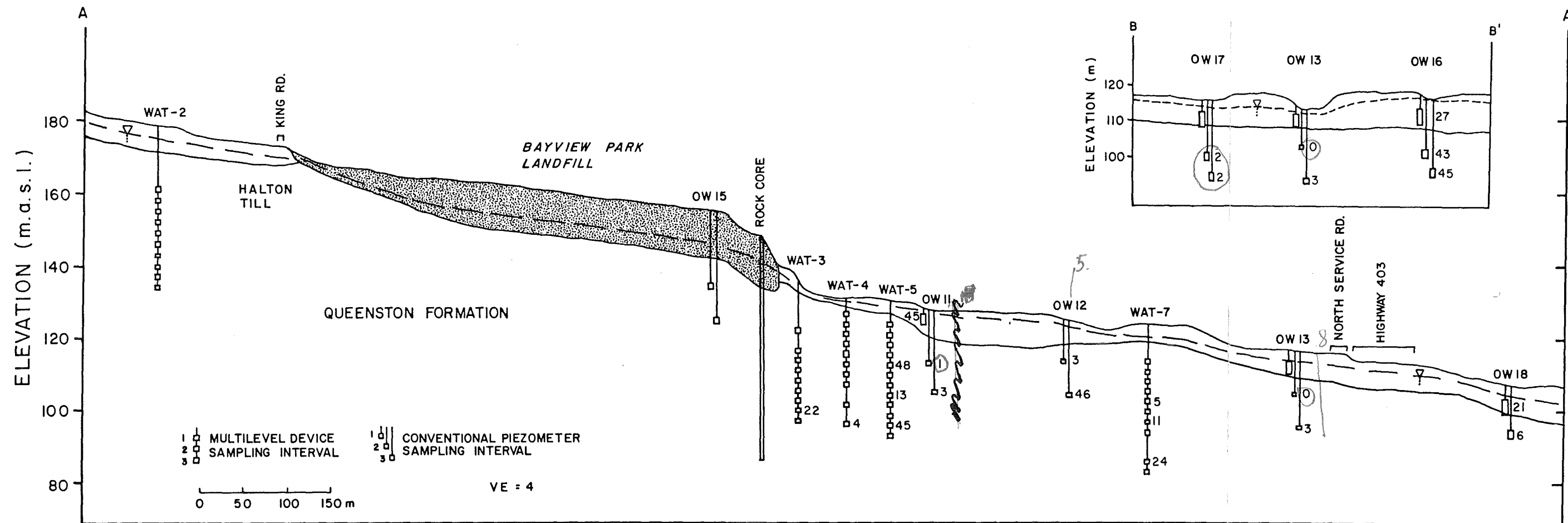
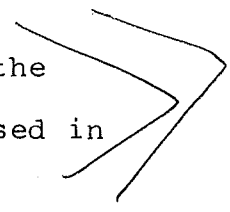


figure C.3.2
DISTRIBUTION OF TRITIUM IN GROUNDWATER 1983, 1984.
Site 'F'-Hydrogeology - City of Burlington

) This interpretation is consistent with the tritium data provided by Trow (Exhibit H-40) and discussed in Section B.3.6 (p.47) of this report. 

C.3.5 DISCUSSION

The hydraulic conductivity data range over more than five orders of magnitude. There is no clear  pattern to the distribution of hydraulic conductivity values. This range reflects the variable degree of fracturing within the bedrock and demonstrates the degree of complexity of the hydrogeology at this site.

) The groundwater flow system at this site is characterized by an irregular distribution of discrete zones of more active groundwater flow set in a low permeable fractured rock matrix as evidenced by the water level and tritium data. The distribution, orientation, aperture width and degree of connection of those fractures which provide for active groundwater flow are unknown. [To date it has not been possible to quantify with confidence those parameters which govern the rate and, on a local scale, the direction of groundwater flow within these fractures.]

) On the basis of the available data, the site cannot be subdivided into distinct hydrogeologic units each with its own hydraulic characterization.

A recent study conducted on fractured shales in the Oakville area highlights the complexity of the fractured shale environment. This investigation, conducted by Mase, Thompson, Cherry, Blackport and Smith, (1987) was aimed at characterizing the hydraulic head and fracture aperture within a discrete fracture zone with the view to determining the groundwater velocity within this zone. This paper is contained in Appendix ^C I of this report. The initial results of this study are summarized as follows:

- 1) Packer-tests conducted in the shale above and below the most permeable zone (at 10.4 m below ground surface) suggest a hydraulic conductivity of less than 1×10^{-8} cm/sec,
- 2) The direction of groundwater flow in the fracture zone is to the northeast which varies from the regional flow direction (southeast), and
- 3) Groundwater flow velocity within a 0.3 m fracture zone is approximately 20 m/day with a hydraulic gradient of 0.01.

CRA concludes from the above that an elaborate investigation is required to define the hydraulics of flow in this fractured rock environment. Such an investigation is consistent with the methodology suggested by

Mr. Feenstra for the installation and testing of the monitoring wells at this site (Section B.8, p. 86). As concluded by CRA (Section B.8, p. 86) such an investigation is aimed at a realistic delineation of the patterns of groundwater flow in the interconnected fracture network. Such an investigation should therefore be completed prior to undertaking an assessment of the suitability of this site for landfilling.

C.3.6 HYDROGEOLOGIC CONCEPTUALIZATION

Any explanation of the hydrogeology of this site must account for the generally steep downward hydraulic gradients encountered on the site. The steepest gradients are generally encountered at depth with lateral gradients in the upper portions of the rock. Downslope off site, the gradients are generally subhorizontal with some upward gradients encountered beneath the adjacent residential area of Aldershot.

Consideration of these gradients indicates that the site is "underdrained". In such a system, a higher permeability zone or zones exists at depth which acts to drain the overlying rock and provides a means for the lateral flow of this water away from beneath the site. The position of such a drain or drains lies at or beneath the deepest

may be above deepest level

on-site piezometers at which the steep downward hydraulic gradients are encountered; specifically, HC3, HC4, HC5, HC6, HC7 and HC8.

The uppermost section of the bedrock is generally weathered and fractured and, as such, is expected to have a higher hydraulic conductivity than the underlying less weathered rock. Less steep gradients were encountered in this zone, indicating that it comprises a pathway of lateral groundwater flow. However, the exact vertical extent of weathered and fractured bedrock mantle is unknown.

Off site, weak downward gradients are encountered immediately south of Highway 403. Further south in Aldershot, weak upward gradients are encountered at HC10, HC11 and HC12. At Lake Ontario, the gradients are weakly downwards.

Such evidence suggests that the underdrain system which exists beneath Site F extends off site to HC10, HC11 and HC12. At HC11, Trow reported possible fracture zones at 15.81-16.36 m; with clay/mud seams at 16.4-20.0 m, 19.45-20.68 m, 21.43-21.52 m; as well as possible slickenslides at 19.4-19.45 m, 19.86-20.0 m and 21.43-21.52 m below ground surface (Appendix 2, Exhibit H-40A). This fracturing may comprise the lateral extension of an underdrain.

The horizontal limits of this underdrain are undefined beyond HC11. Since HC16 appears noticeably less fractured and does not have a discharge gradient, it is probable that the underdrain does not extend this far downgradient.

The groundwater flow system at this site is characterized by:

- 1) A shallow groundwater flow system dominated by lateral flow in the weathered and fractured bedrock mantle.
- 2) A deep groundwater flow system dominated by strong downward gradients, with significant lateral flow at depth.

CRA has interpreted the hydrogeology of the site as follows:

- 1) A heterogeneous upper weathered and fractured bedrock zone which mantles the entire site.
- 2) A thick lower permeable zone with permeabilities generally ranging from 1×10^{-6} cm/sec to less than 1×10^{-8} cm/sec.
- 3) A moderately permeable drain or drains within this sequence of thick lower permeable strata.

It must be emphasized that this simplified hydrogeologic interpretation does not reflect the complexity of the site as evidenced by the water levels, hydraulic conductivity and tritium data.

C.4.0 GROUNDWATER FLOW MODELLING

C.4.1 INTRODUCTION

A computer model (FLONET) was employed by Trow to simulate the groundwater flow system at the site. The calibrated model, as interpreted by Trow, provided the following:

- 1) Confirmation of the conceptual hydrogeologic definition of the site (Figure B.3.1, p. 27).
- 2) An estimate of infiltration of 13.7 mm/yr which was supported by the value of 22.5 mm/yr calculated using the Darcy equation (Figure 7, p.75 Exhibit H-40).
- 3) The hydraulic conductivities used by Trow in the contaminant transport modelling.
- 4) The basis for developing some aspects of the landfill design, specifically;
 - a) the point of discharge for lateral groundwater flow from immediately beneath the proposed landfill (Figure 21, p. 125, Exhibit H-40),
 - b) the efficiency of the proposed leachate collection system, and

- c) the incremental flux in the deep groundwater flow system due to landfilling.

CRA concludes from the above that this groundwater flow simulation is highly dependent on the hydrogeologic interpretation of the site (as developed by Trow). As such, the applicability of the model was reviewed by CRA in order to establish the validity of using the model and the results.

As discussed in Section B.3 of this report, CRA concluded that the available hydrogeological data provided by Trow, namely the water levels, hydraulic conductivity tests, pumping test results and tritium distribution, do not support the hydrogeologic interpretation.

The value of infiltration developed from the model (Trow's Model 1 - 13.1 mm/yr) is low. CRA considers an infiltration rate in the range of 25 to 100 mm/yr to be more appropriate.

II. 1.2 mm.

The groundwater flow model does not reflect the groundwater flow through the fractures within the shale. As discussed in Section B.6, CRA considers that the contaminant transport modeling must be based on the

groundwater velocity within the fractures. This velocity cannot be determined from the groundwater flow modeling undertaken by Trow.

As concluded by CRA (Section B.7), this model does not provide adequate support for the landfill design proposed.

CRA concludes therefore, that the groundwater flow model does not support Trow's interpretation of the hydrogeology. The model therefore, cannot realistically provide the basis for the impact assessment nor for some components of the landfill design.

Since the model is integral to Trow's assessment of this site, CRA undertook a detailed evaluation of the model.

CRA has reservations regarding the applicability of using a porous media model to assess the hydraulics of fractured rock. Such a model may give a reasonable representation of the regional groundwater flow system however, it cannot adequately reproduce the complexity of the site nor provide the basis for contaminant transport modelling within the fractures. As such, the usefulness of the model for such purposes is questionable.

The groundwater flow modelling undertaken by CRA and described in this section consists of:

- 1) An evaluation of the sensitivity of the FLONET model to variations in the hydraulic conductivity of the hydrogeologic units as defined by Trow.
- 2) A simulation, using the FLONET model, of the hydraulic gradients and measured water levels independent of the Trow hydrostratigraphy.

C.4.2 SENSITIVITY ANALYSES

The sensitivity analyses were performed using the FLONET model. The finite element grid and the values of the hydraulic conductivity for each element were provided by Trow.

These additional simulations were performed by CRA in order to investigate the effects of:

- 1) Changing the upgradient boundary from a "no flow" to a "prescribed head" boundary.

2) Varying the hydraulic conductivity of the 'weakly fractured massive shale' (Unit 3) and the 'highly fractured shale' (Unit 4) by an order of magnitude. This sensitivity analysis was performed on the isotropic model (Trow's Model 1) and on the anisotropic model (Trow's Model 2).

Changing the upgradient boundary condition from a "no-flow" boundary (Trow's Model 1 and Model 2) to a "prescribed head" boundary, resulted in no significant variation in the simulated flow system and was not evaluated further.

For the sensitivity analysis 11 simulations were run by CRA, as detailed in Table C.4.1, (p. 121). It should be noted that both Model 1 and Model 2, as provided by Trow, were reproduced as part of this sensitivity analysis.

For each case run for the sensitivity analysis, the modelled results were compared with the April 1986 water level data at the appropriate node. The nodes which were used by Trow for comparison of the results appeared to be inconsistent between the two models as presented in Exhibit H-221. Hence a series of nodal values

TABLE C.4.1

SUMMARY OF FLONET SIMULATIONS

<u>Case</u>	<u>Highly Fractured Shale (Unit 4)</u>	<u>Weakly Fractured Shale (Unit 3)</u>	<u>Isotropic</u>	<u>Anisotropic</u>
① model 1	10^{-3}	10^{-7}	X	
2	10^{-4}	10^{-7}	X	
3	10^{-2}	10^{-7}	X	
4	10^{-3}	10^{-6}	X	
5	10^{-3}	10^{-8}	X	
6	10^{-4}	10^{-6}	X	
7	10^{-3}	10^{-7}		X
⑧ model 2	10^{-4}	10^{-7}		X
9	10^{-2}	10^{-7}		X
10	10^{-3}	10^{-6}		X
11	10^{-3}	10^{-8}		X

Notes:

Case 1 = Trow's Model 1

Case 8 = Trow's Model 2

For anisotropic cases values are given for K_H ; $K_V = .1 K_H$

) which closely represented those used by Trow in Model 2, was used by CRA for the comparison between the various simulations. For each simulation, the total difference between the simulated and measured water levels was calculated by summing the real value difference between the measured and simulated values. The infiltration rate was determined for each case by summing the stream tubes over the site.

C.4.2.1 RESULTS

) The calculated difference between the simulated and measured heads, and the infiltration rate for each simulation are presented in Table C.4.2 (p. 123). For Trow's Models 1 and 2, the resulting total difference between the simulated and measured heads was 45.54 and 26.88 meters, respectively. Trow's Model 2 has the better overall fit. However, a number of other cases run by CRA had similarly good head fits.

The results of the various simulations are detailed in Appendix II.

TABLE C.4.2

SENSITIVITY ANALYSIS RESULTS

Case	Highly Fractured Shale (Unit 4) (cm/sec)	Weakly Fractured Shale (Unit 3) (cm/sec)	Isotropic	Anisotropic	Infiltration (mm/yr)	Difference (m)
1	10^{-3}	10^{-7}	X		13.7	45.54
2	10^{-4}	10^{-7}	X		1.9	48.88
3	10^{-2}	10^{-7}	X		24.78	106.47
4	10^{-3}	10^{-6}	X		31.1	45.62
5	10^{-3}	10^{-8}	X		1.1	86.09
6	10^{-4}	10^{-6}	X		3.0	54.79
7	10^{-3}	10^{-7}		X	1.43	79.91
8	10^{-4}	10^{-7}		X	1.2	26.88
9	10^{-2}	10^{-7}		X	1.8	96.73
10	10^{-3}	10^{-6}		X	10.2	30.27
11	10^{-3}	10^{-8}		X	1.2	106.84

Notes:

Case 1 = Trow's Model 1

Case 8 = Trow's Model 2

For anisotropic cases values are given for K_H ; $K_V = .1 K_H$

Isotropic Cases

Of the various simulations, cases 1, 2, 4 and 6 provide better head fits. Case 1 is identical to Trow's Model 1. The above mentioned simulations correspond a total head difference ranging from 45.54 to 54.79 meters.

For these four cases the infiltration varied more than an order of magnitude from 1.9 mm/yr to 31.1 mm/yr. The hydraulic conductivity of the 'weakly fractured shale' (Unit 3) varied from 10^{-6} to 10^{-7} cm/sec while that for the highly fractured shale (Unit 4) varied from 10^{-3} to 10^{-4} cm/sec.

Anisotropic Cases

Several simulations produced better fits than those obtained from the isotropic cases. Cases 8 and 10 produced a total head difference of 26.88 and 30.27, respectively. Case 8 is identical to Trow's Model 2. Again, the infiltration estimate for these two simulations varied significantly from 1.2 mm/yr to 10.2 mm/yr while the hydraulic conductivities varied from 10^{-3} to 10^{-4} cm/sec for the 'highly fractured shale' (Unit 4) and from 10^{-6} to 10^{-7} cm/sec for the 'weakly fractured shale' (Unit 3) (Table C.4.2, p. 123).

C.4.2.2 Discussion

The sensitivity analysis as undertaken by CRA has demonstrated that a similar fit of the water levels can be achieved with significantly different hydraulic conductivities. The values for the hydraulic conductivities which were used in some of the better fit simulations are physically realizable. For these same simulations, however, the infiltration varied by an order of magnitude.

Thus, for the flow field under consideration, the potentiometric heads simulated by FLONET are not particularly sensitive to variations in the hydraulic conductivities, while the magnitude of the infiltration is. CRA concludes that a better fit of the water levels does not confirm the modelled hydrogeologic conceptualization.

This is consistent with the conclusions of Gartner-Lee (Exhibit H-83) when applying this same model to the fractured shale at the Burlington Landfill, namely that the infiltration is sensitive to the "average" shale hydraulic conductivity used.

A number of additional concerns were identified in the process of running these simulations, specifically:

1. Poor head fit below the 'highly fractured zone' (Unit 4) at HC5 and HC8.
2. Absence of modelled upward gradients at HC11.
3. The presence of slight upward gradients in the modelled results at HC14.

Cases 8 and 10 had the better head fit. For these cases, the largest head differences occurred at HC4-2, HC5-1, and HC8-1. The difference associated with these points is tabulated in Table C.4.3 (p. 127), and is also reflected in the calculated gradients presented on Table C.4.4 (p. 128). The large difference in gradient at HC4-2, HC5-1 and HC8-1 suggests a poor hydrogeological definition by Trow at these points.

In all of the simulations performed by CRA, as detailed in Table C.4.1 (p. 121), none of the results produced an upward gradient at HC-11. Again, this suggests a poor hydrogeological definition in this area.

As mentioned previously, the hydrogeological interpretation by Trow directs groundwater to discharge just beyond HC14. The modelled gradients as detailed in Table C.4.4 (p. 128), reflect slight upward movement of the groundwater in this area. However, the measured field values

TABLE C.4.3

CALIBRATED WATER LEVELS

Well	Piezometer	Field Measurements (April 1986)	Case 1 (Trow Model) #1	Difference	Case 8 (Trow Model) #2	Difference
HC-1	1	177.44	177.63	.19	177.72	.28
	2	180.14	177.66	2.48	178.04	2.1
	3	183.98	183.98 (F)	0	183.98 (F)	0
HC-2	1	167.99	167.99 (F)	0	167.99 (F)	0
HC-4	1	122.96	~~122.96~~ 117.44	5.49	122.15	.81
	2	133.18	135.25	2.07	135.25	2.07
	3	133.60	135.35	1.75	135.38	1.78
	4	135.44	135.44 (F)	0	135.44 (F)	0
HC-5	1	114.97	112.84	2.13	116.02	1.05
	② highly frac	<u>118.97</u>	<u>112.84</u>	<u>6.13</u>	<u>116.03</u>	<u>2.94</u>
	3	124.70	121.51	3.19	125.35	.65
	4	125.83	125.83 (F)	0	125.83 (F)	0
HC-8	① highly frac	109.46	111.58	<u>2.12</u>	114.38	<u>4.92</u>
	2	115.59	112.62	<u>2.97</u>	115.20	.39
	3	117.87	113.66	4.21	116.01	1.86
	4	117.84	117.84 (F)	0	117.84 (F)	0
HC-14	1	108.31	105.77	2.54	107.26	1.05
	2	108.32	105.77	2.55	107.23	1.09
	3	108.63	105.77	2.86	107.22	1.41
	4	109.14	109.14 (F)	0	109.14 (F)	0
HC-11	1	92.09	89.42	<u>2.67</u>	89.14	<u>2.95</u>
	2	89.48	89.48 (F)	0	89.48 (F)	0
HC-16	1	74.49	75.98	1.49	75.61	1.12
	2	75.39	76.09	.70	75.80	.41
	3	76.21	76.21 (F)	0	76.21 (F)	0
				<u>45.54</u>	26.88	

NOTE: (F) denotes prescribed head.

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of 4770

TABLE C.4.4
CALIBRATED VERTICAL HYDRAULIC GRADIENTS

*see p. 59
 (different.)*

<u>Well Nest</u>	<u>Field Measurements¹</u> (April 1986)	<u>Model 1²</u> <u>Isotropic</u>	<u>Model 2²</u> <u>Anisotropic</u>
HC1-85(1-2)	-0.233	-0.003	-0.034
(2-3)	-0.240	-0.340	-0.319
HC4-85(1-2)	-0.780	-1.235	-0.909
(2-3)	-0.068	-0.014	-0.018
(3-4)	-0.209	-0.013	-0.008
HC5-85(1-2)	-0.328	0.000	-0.001
(2-3)	-0.610	-0.639	-0.685
(3-4)	~~-0.135~~ .224	-0.637	-0.071
HC8-85(1-2)	~~-0.444~~ -.447	-0.065	-0.051
(2-3)	~~-0.356~~ -.351	-0.323	-0.253
(3-4)	~~+0.003~~ +.006	-0.325	-0.143
HC14-86(1-2)	-0.001	0.000	+0.005
(2-3)	-0.084	0.000	-0.002
(3-4)	-0.035	-0.279	-0.159
HC11-85(1-2)	+0.153	-0.003	-0.014
HC16-86(1-2)	-0.098	-0.012	-0.021
(2-3)	~~-0.081~~ -0.928	-0.009	-0.029

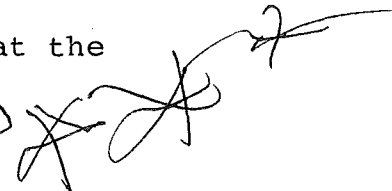
Notes:

Negative values indicate downward flow.

- Gradients calculated by CRA using mid-screen separations except for water table.
- Gradients calculated by CRA using nodal separations.

do not provide evidence of upward gradients at this point. This evidence again suggests poor hydrogeological definition of the discharge zone.

Based on this sensitivity analysis of Trow's models undertaken by CRA, the following is concluded:

- ✓ 1. The hydraulic conductivities of the calibrated models are non-unique.
2. Calibration of the model does not confirm that the hydraulic conductivities are representative. 
3. Calibrated models yield infiltration estimates that vary over an order of magnitude.
4. The hydrostratigraphy of the site is more complex than that interpreted by Trow as evidenced by a consistently poor head fit encountered in the 'highly fractured shale' (Unit 4) and a poor fit of hydraulic gradients at HC11 and HC14.

C.4.3 CRA'S FLOW MODEL

C.4.3.1 Introduction

This computer modelling was performed with the view to reproduce the observed hydraulic gradients at both on-site and off-site well nests. This simulation was performed using FLONET and the finite element grid used by Trow. The hydraulic conductivity was varied at specific groups of elements in order to provide a better fit to the observed water levels and gradients.

It must be emphasized that the calibrated CRA flow model does not confirm the hydrogeologic system modeled, but does give a reasonable approximation of the regional groundwater flow system at this site.

C.4.3.2 Results

The resultant hydrogeology of the revised CRA model is depicted in Figure C.4.1 (p. 131). Values of hydraulic conductivity used range from 1.0×10^{-7} to 8.1×10^{-4} cm/sec. The model is isotropic. This figure shows a more extensive drainage layer than that interpreted by Trow. The flow pattern, as produced by FLONET, is shown in Figure C.4.2 (p. 132). The calculated heads and the

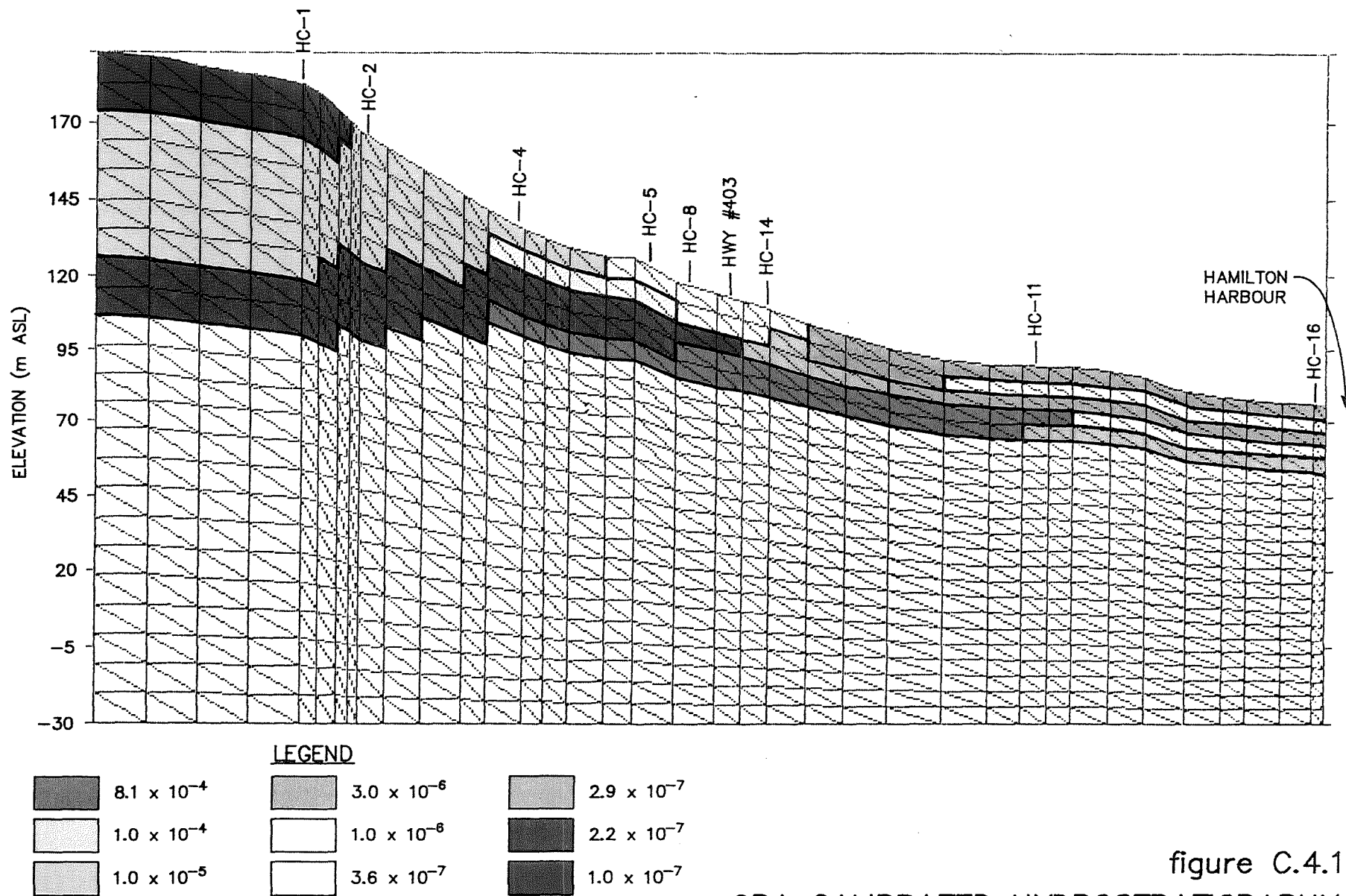
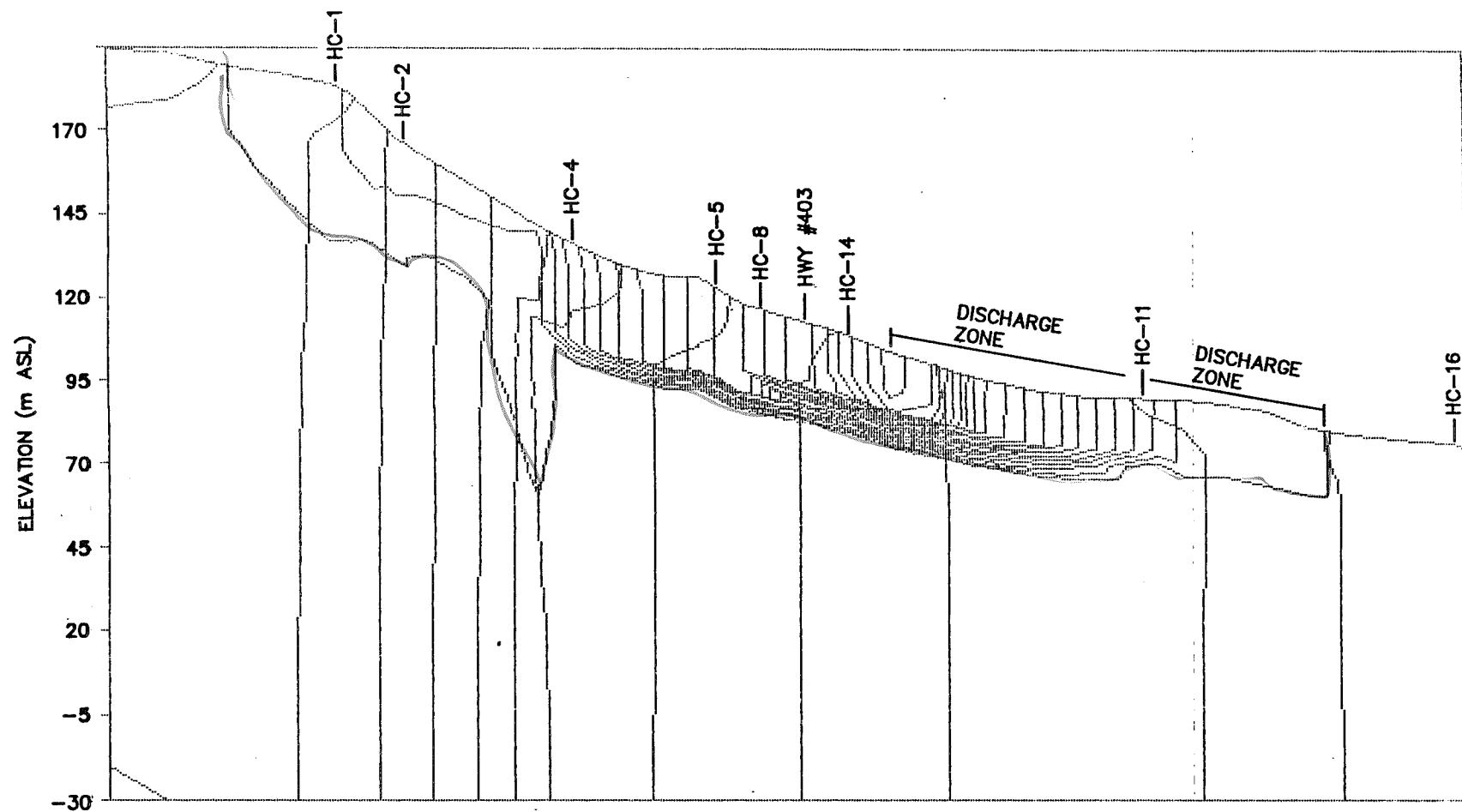


figure C.4.1

CRA CALIBRATED HYDROSTRATIGRAPHY
 Site 'F'—Hydrogeology — City of Burlington



CRA

figure C.4.2
 CALIBRATED FLOW MODEL
 Site 'F'-Hydrogeology - City of Burlington

measured field values are detailed in Table C.4.5 (p. 134). The results of this new model are compared with the results of Trow's Model 1 and Model 2. In addition, the vertical hydraulic gradients determined from this model are provided on Table C.4.6 (p. 135).

C.4.3.3 Discussion

By comparing the results as detailed in Table C.4.5 (p. 134), it is apparent that this model (CRA's calibrated model) produces better results than those provided in Section C.4.2 (p. 119). The better fit does not, however, indicate a better understanding of the hydrogeology of the site. It does indicate that by matching the measured hydraulic heads, various non-unique solutions of the hydraulic conductivity and infiltration rate can be produced.

Since it is possible to produce a variety of plausible solutions, the resulting effects must also be addressed. One effect of the latest simulation is the alteration of the discharge area. No longer is the discharge restricted to areas just south of piezometer HC14 (as postulated by Trow). (The discharge area is shown to lie within the residential area of Aldershot.) Another consequence of the latest model simulation is the increased

TABLE C.4.5

SUMMARY OF MEASURED AND MODELLED POTENTIOMETRIC HEADS

Well	Piezometer	Field Measurements (April 1986)	Case 1 (Trow Model) #1	Differ- ence	Case 8 (Trow Model) #2	Differ- ence	CRA's Calibrated Model	Differ- ence
HC-1	1	177.44	177.63	.19	177.72	.28	177.80	0.36
	2	180.14	177.66	2.48	178.04	2.1	177.83	2.31
	3	183.98	183.98 (F)	0	183.98 (F)	0	183.98 (F)	0
HC-2	1	167.99	167.99 (F)	0	167.99 (F)	0	167.99 (F)	0
HC-4	1	122.96	117.47	5.49	122.15	.81	124.46	1.5
	2	133.18	135.25	2.07	135.25	2.07	132.77	.41
	3	133.60	135.35	1.75	135.38	1.78	135.34	1.74
	4	135.44	135.44 (F)	0	135.44 (F)	0	135.44 (F)	0
HC-5	1	114.97	112.84	2.13	116.02	1.05	116.67	1.7
	2	118.97	112.84	6.13	116.03	2.94	119.87	.9
	3	124.70	121.51	3.19	125.35	.65	125.09	0.39
	4	125.83	125.83 (F)	0	125.83 (F)	0	125.83 (F)	0
HC-8	1	109.46	111.58	2.12	114.38	4.92	112.95	3.49
	2	115.59	112.62	2.97	115.20	.39	114.52	1.07
	3	117.87	113.66	4.21	116.01	1.86	116.09	1.78
	4	117.84	117.84 (F)	0	117.84 (F)	0	117.84 (F)	0
HC-14	1	108.31	105.77	2.54	107.26	1.05	107.28	1.03
	2	108.32	105.77	2.55	107.23	1.09	107.28	1.04
	3	108.63	105.77	2.86	107.22	1.41	107.30	1.33
	4	109.14	109.14 (F)	0	109.14 (F)	0	109.14 (F)	0
HC-11	1	92.09	89.42	2.67	89.14	2.95	91.80	.29
	2	89.48	89.48 (F)	0	89.48 (F)	0	89.48 (F)	0
HC-16	1	74.49	75.98	1.49	75.61	1.12	75.50	1.01
	2	75.39	76.09	.70	75.80	.41	75.62	.23
	3	76.21	76.21 (F)	0	76.21 (F)	0	76.21 (F)	0
				45.54			26.88	20.58

NOTE: (F) denotes prescribed head.

TABLE C.4.6

SUMMARY OF VERTICAL HYDRAULIC GRADIENTS
FROM CALIBRATED MODELS

<u>Well Nest</u>	<u>Field Measurements¹</u> <u>(April 1986)</u>	<u>Model 1²</u> <u>Isotropic</u>	<u>Model 2²</u> <u>Anisotropic</u>	<u>CRA's²</u> <u>Calibrated</u> <u>Model</u>
HC1-85(1-2)	-0.233	-0.003	-0.034	-0.003
(2-3)	-0.240	-0.340	-0.319	-0.331
HC4-85(1-2)	-0.780	-1.235	-0.909	-0.577
(2-3)	-0.068	-0.014	-0.018	-0.356
(3-4)	-0.209	-0.013	0.00	-0.014
HC5-85(1-2)	-0.328	0.000	0.00	-0.471
(2-3)	-0.610	-0.639	-0.685	-0.384
(3-4)	~~-0.135~~ -0.224	-0.637	-0.07	-0.108
HC8-85(1-2)	~~-0.444~~ -0.447	-0.065	-0.04	-0.098
(2-3)	~~-0.356~~ -0.351	-0.323	-0.206	-0.491
(3-4)	~~+0.003~~ +0.006	-0.325	-0.208	-0.136
HC14-86(1-2)	-0.001	0.000	0.00	0.000
(2-3)	-0.084	0.000	0.00	-0.003
(3-4)	-0.035	-0.279	0.00	-0.152
HC11-85(1-2)	+0.153	-0.003	-0.014	+0.112
HC16-86(1-2)	-0.098	-0.012	-0.021	-0.013
(2-3)	~~-0.001~~ -0.328	-0.009	-0.029	-0.043

Notes:

Negative values indicate downward flow.

1. Gradients calculated by CRA using mid-screen separations except for water table.
2. Gradients calculated by CRA using nodal separations.

amount of infiltration to the site. The estimate of infiltration for the latest revised model is 21.5 mm. This is compared to 13.7 mm/yr and 1.2 mm/yr for Trow's Models 1 and 2, respectively.

Therefore, in the view of CRA, the use of an equivalent porous media model such as FLONET is not adequate to accurately model local groundwater flow at Site F. Since various reasonable values for hydraulic conductivity can produce a variety of infiltration values as well as significantly different flow patterns while producing a good head fit, one definitive solution cannot be obtained. Consequently, determinations of infiltration and hydraulic conductivity which are derived from this model are not applicable for definitive contaminant transport modelling, nor do they provide the basis for designing an engineered landfill.

Regardless of the good fit of the hydraulic heads via any of the previous simulations, there can always be errors in some of the calculated nodal values. This is not uncommon in a fractured rock media. Hydraulic head distributions within a fractured rock environment are usually very complex, as demonstrated at the WAT multilevel piezometers (Hewetson, Exhibit E-214). It is, therefore, not unusual to find groundwater moving in a different direction from the regional gradient (Mase, Thompson, Cherry, Blackport

and Smith, 1987, Appendix III). Therefore, various calculated hydraulic heads, regardless of the associated difference between simulated and measured values, may not at all reflect a representative value within the local hydrogeological flow regime.

An equivalent porous media model such as FLONET does not incorporate the effects of a complex fracture network. The flow characteristics, as produced by FLONET, cannot accommodate the local complex variations of groundwater flow as seen at this site.

C.4.4 CONCLUSIONS

CRA concludes that the modelling of Site F will not produce a unique solution due to the various possible hydrogeological interpretations. The design of an engineered landfill is very dependent on a definitive solution.

A generalized model such as FLONET cannot be used to describe a definitive groundwater flow pattern in the fractured rock at this site. The porous media model cannot accurately reproduce local fluctuations in the groundwater flow pattern due to the complex nature of groundwater flow in a fractured media. Therefore, it is inappropriate to use the

FLONET results for contaminant transport modelling or in the design of an engineered landfill which requires a definitive solution on the local scale without consideration of the uncertainties associated with the results of FLONET.

C.5.0 SUITABILITY FOR LANDFILLING

C.5.1 INTRODUCTION

As described in Part A of this report, for a site to be considered suitable for landfilling it must meet three criteria. These are:

- 1) Migration of contaminants from the landfill must be predictable and the impact from this migration must be determined to be acceptable.
- 2) The site must be able to be adequately monitored in order to verify the predictions on the behavior of the contaminant migration.
- 3) Feasible contingency plans must be developed such that if unacceptable contaminant migration results, as determined from the monitoring, this migration can be mitigated.

Each of these criteria are now examined in light of the interpretation of the groundwater flow system presented herein by CRA.

C.5.2 PREDICTABILITY

In order to ensure that unacceptable groundwater quality impacts will not occur, potential contaminant transport times are evaluated for the given landfill design and hydrogeologic conceptualization. This evaluation requires that the velocity and direction of groundwater flow be quantified. CRA's interpretation of the groundwater flow system at this site demonstrates that the flow paths along which contaminants may potentially migrate are not realistically defined. The groundwater velocities in the zones of active groundwater flow are also unknown. Since the pathway(s) of potential contaminant migration at this site have not been clearly defined, it is not possible to reliably predict the extent and degree of groundwater quality degradation that may occur as a consequence of landfilling.

A landfill at this site as presently designed does not meet the criteria of predictability.

Given that some degree of off-site groundwater contamination can be expected from the proposed landfill at Site F, the acceptability of this contamination must be examined.

Unfortunately, a strictly technical test of acceptability is not possible. This follows necessarily from

the argument that the local pattern of contaminant migration cannot be predicted. That is, if the direction, concentration, and extent of contamination cannot be predicted, a determination of the effect this contamination will have on neighbouring property cannot be made.

However, the studies undertaken to date at the Burlington and Bayview Park Landfill Sites have not identified a major contaminant plume. Identified contaminant levels are low, sporadic and local to the landfills, and some of the contaminants probably result from extraneous sources.

In addition, the bedrock groundwater is of poor quality and it cannot reasonably be considered as an exploitable resource.

From a strictly technical perspective, an argument can be made that the contaminant release from Site F to the natural environment would not cause any adverse impact as it would be diluted/dispersed/attenuated/masked within the naturally poor quality groundwater. As such, using the Ministry of the Environment, Reasonable Use Guidelines (Exhibit E-154) this argument could be extended to indicate that "the discharge of contaminants (leachate) to adjacent property would have no more than a negligible effect on the present or potential reasonable use of that property" (page 13). However, given the difficulty in predicting this

contaminant migration, this statement can only be made on the basis that the groundwater is of such poor natural quality that there is no reasonable use of this. While this argument may be compelling, the following must also be recognized.

- 1) There is some limited use of the groundwater at the present time.
- 2) The public, as a whole, is very concerned about any contamination of the groundwater.

It should also be noted that, if it is determined that off-site contamination from the proposed landfill is a serious problem, the contaminants may be widespread in the natural environment to a degree that remediation may prove to be impractical.

Given that some degree of off-site groundwater contamination can be expected from the proposed landfill at Site F, the acceptability of this contamination must be examined.

The "Reasonable Use Guidelines" (Exhibit E-154) also notes that the Ministry of the Environment has the management authority to "prevent the owner of a waste disposal site from using the subsurface

beneath his property for waste disposal purposes if this use is not felt to be in the public interest" (p. 10, emphasis added).

In the present case, the public interest must be defined in terms of the affected public sectors. It is also necessary to consider how the interests of the affected public sectors are formed and expressed.

The first sector to consider is the public that is most directly affected by the development of Site F, namely, the residents of Aldershot. The concerns and fears of the Aldershot residents have been thoroughly documented elsewhere. For present purposes one does not have to judge the validity of these fears and concerns. It is sufficient simply to acknowledge the wide range of serious effects to which these fears and concerns predispose the Aldershot residents. The inescapable conclusion is that inevitable but indeterminate contaminant migration is not in their public interest.

CRA believes that the construction of a low permeable liner at Site F would permit the prediction with relative confidence of the relatively minute but inevitable leakage through that liner, and that such leakage predictions can be verified through monitor devices. Only if such a liner and monitoring system were to be incorporated into the

development of a landfill at Site F, could CRA consider Site F as hydrogeologically suitable. Such safeguards would clearly serve the public interest of residents of Aldershot. In recommending these safeguards, CRA has considered that they are supported by concerned Aldershot residents, who are members of a broader public which will bear the considerable additional costs. //



The relation of the question of safeguards at Site F to the public interest of the broader Ontario public can now be considered.

Over the last decade or so environmental public opinion in Ontario has been radically transformed in the direction of more stringent environmental protection. This transformation can be demonstrated by a long list of environmental concerns and initiatives in both the private and the public sectors. The Ministry of the Environment and environmental boards and commissions have both reflected and shaped the changing public opinion. Initiatives in the areas of mobile PCB destruction, Great Lakes/Hamilton Harbour revitalization, spills of hazardous contaminants, and the management of both municipal and industrial wastes (Municipal-Industrial Strategy for Abatement) and hazardous wastes (Ontario Waste Management Corporation), have established one guiding premise for public policy: the benefit of any doubt must go to the protection of the

environment. In defence of this premise, governments and private industries are now prepared to pay sums that were formerly considered prohibitive.

A parallel transformation of Ontario public opinion with respect to landfilling of wastes, has also taken place. The disposing of waste in a relatively haphazard manner with little concern for the natural environment has given way to engineered landfill sites. The consensus that all landfill sites leak in the long run, has even led certain segments of informed opinion to recommend the replacement of landfilling by some alternative method of disposal.

It is CRA's opinion that all major landfill sites within the Province of Ontario should be designed and constructed using the best practical technology to control discharge of contaminants. It is CRA's opinion that for the proposed landfill design, it cannot be demonstrated that the best practical technology has been employed since it is not possible to predict nor monitor with precision the release of contaminants from the site. To not employ the best practical technology for controlling contamination at Site F runs against the present strong consensus of professional and public opinion.

As such the test for acceptability of the potential leachate migration has not been met by the proposed design.

C.5.3 MONITORING

In order to demonstrate that the landfill is performing as predicted, a monitoring program must be implemented.

The success of a monitoring program requires that:

- 1) The potential contaminant pathway be delineated.
- 2) Background water quality be defined against which quality changes can be assessed.
- 3) A suitable landfill derived indicator parameter must be identified with which to establish the extent of groundwater contamination.

With respect to the first item, CRA concludes that the potential contaminant pathways at this site cannot be adequately defined.

Numerous investigations conducted at both the Burlington and Bayview Park Landfills have identified the difficulty of defining the background water quality. The groundwater at the site becomes more mineralized with depth. The chloride concentration in the uppermost portions of the rock is less than 250 mg/L while below 20 m, values in excess of 10,000 mg/L can be found.

Such highly mineralized waters mask the presence of typical indicators of landfilled derived contamination. Thus, the proposed landfill at this site does not meet the criteria of ability to monitor.

However, given that landfilling has taken place and further, that leachate has migrated into the fractured rock beneath the site, the groundwater quality data base at both the Burlington and Bayview Park Landfills was reviewed by CRA.

This review shows that the following:

- 1) The naturally very high levels of inorganic compounds preclude the use of the more common leachate constituents as indicators of groundwater contamination.
- 2) Synthetic organic compounds occur sporadically and at low levels downgradient from both the Burlington and Bayview Park Landfills. Many of these compounds probably result from extraneous sources.
- 3) It has not been demonstrated that the existing observation wells monitor the deep contaminant pathway as delineated by Trow.

- 4) It cannot be demonstrated that the monitor well screens straddle the fracture zones that provide for the bulk of the groundwater flow from the landfill.

The monitoring programs conducted at both the Burlington and Bayview Park Landfill sites highlight the difficulty in delineating a contaminant plume in the fractured shales at the site.

CRA concludes that the criteria of ability to monitor the site as proposed cannot be adequately met.

C.5.4 CONTINGENCY PLAN

A contingency plan is required for implementation in the event that an unacceptable degradation of the groundwater quality occurs. The effective implementation of a contingency plan requires that the monitoring program provide advance indication of unacceptable groundwater quality degradation.

As was concluded by CRA in the previous section (C.5.3), the monitoring of this site can not adequately delineate the pattern of groundwater contamination. As well, the rate of contaminant migration has not been realistically quantified.

Purge wells are proposed by Trow as a contingency option for this site. Tests conducted by Trow at this site showed that some of the observation wells monitored responded to pumping. Trow concluded (p. 137, Exhibit H-40) that a purge well contingency option could be considered.

A purge well contingency may provide hydraulic control of contaminants migrating in groundwater at this site. However, CRA concludes that there are a number of uncertainties associated with this contingency option; specifically

- 1) The monitoring to date does not provide assurance that a trigger level at which to implement the contingency plan can be defined.
- 2) The effectiveness of a purge well(s) in the shallow flow system may not be readily demonstrated.
- 3) The results of test pumping to date do not demonstrate that contaminants in the deep pathway, as defined by Trow, can be controlled by pumping.

C.5.5 SUMMARY

CRA concludes that the site does not meet the criteria of ability to predict or monitor future contaminant distribution. The escape of leachate from the proposed landfill in unknown amounts is not in the public interest.

While contingency purge wells can be considered, the contaminant level at which this option would be implemented may not readily be determined from the monitoring program.

The site as proposed is not hydrogeologically suitable. However, an alternative design can be considered in order to meet the criteria of ability to predict and monitor future contaminant distribution. This alternative landfill design is discussed in the next section.

C.6.0 HYDROGEOLOGIC CONSIDERATIONS FOR LANDFILL DESIGN

A landfill design at this site must consider the following; that

- 1) The weathered and fractured bedrock mantle provides for lateral groundwater flow.
- 2) This mantle is underlain by fractured rock in which hydraulic gradients are generally directed downwards but are locally quite variable.
- 3) The groundwater flux downward through this fractured rock has not been reliably quantified.
- 4) Groundwater flow rates within the fracture network could be in the order of meters/day but, as of present, are incompletely quantified.

The volume of leachate that will leak from this site is unknown. Trow's estimate of leakage is based on the infiltration rate determined from the flow model. As discussed previously (Section C.4), models of a similarly good fit resulted in variations in the infiltration rate of an order of magnitude. CRA concludes that the infiltration at this site cannot be reliably determined from the flow model and, therefore, the efficiency of the leachate collection system cannot be realistically determined.

The landfill proposed by Trow considers placing waste onto the 'moderately weathered and fractured shale' (Unit 2). As demonstrated by CRA, this unit has an average bulk hydraulic conductivity of 2×10^{-4} cm/sec. This unit does not provide a low permeable base upon which to landfill. The efficiency of a leachate collection system underlain by 10^{-4} cm/sec material will be low. *

average

Given the above information and comments presented earlier in this report, CRA concludes that the landfill, as designed, is ~~not hydrogeologically suitable~~. An alternate design can, however, be considered. The principal elements of this design are as follows;

- 1) A leachate collection system.
- 2) A liner.
- 3) An underdrain system beneath the liner.

Installation of a liner allows for a substantially reduced and predictable leachate loss from the landfill. The underdrain system provides the capacity to monitor the performance of the liner. This underdrain also functions as a contingency system in the event of a liner failure. This alternate design places less reliance on the natural geological system. The performance of the engineered components of the landfill system can be monitored. The breakthrough times and peak concentrations of leachate

migrating through the liner can be predicted and monitored. The contaminant flux collected in this drainage layer can be determined by monitoring.

The need for a liner rests in part on demonstrating that a significant reduction in exfiltration from the landfill can be achieved. The exfiltration estimates provided by Trow range from 2.5 to 22.4 mm/yr, while those developed by Rowe (Exhibits M-440 and 444) range to as low as 1.0 mm/yr. CRA consider that this range is unreasonably low. Estimates for infiltration in Ontario generally are considered to range from 100 to 250 mm/yr (4 to 10 inches/yr). Sibul (et. al.) provide an estimate of 114.3 to 139.7 mm/yr (4.5 to 5.5 inches/yr) for the Duffins Creek-Rouge River drainage basins. These values have generally been developed from regional water budget studies and not from site specific investigations.

CRA considers that the range of ~~infiltration~~ ^{exfiltration} estimates developed for this site (1.0 to 22.4 mm/yr) demonstrates the significant degree of uncertainty associated with characterizing the hydrogeology at Site F. In addition, CRA considers the highest of the above ~~infiltration~~ ^{exfiltration} rates (22.4 mm/yr) as low. Given that a landfill provides a continuous source of water due to the ponding of leachate, CRA expects that exfiltration from the landfill will exceed the natural rate of ~~infiltration~~ from precipitation.

Unfortunately there is very little available documentation concerning infiltration rates in fractured material.

In the area of fractured surfaces, it has been demonstrated that fracturing can significantly increase the infiltration rates of a soil (e.g. Thomas and Phillips, 1979). In another study in clay, Bouma et. al. (1978) introduced the term "short-circuiting" in soils and documented infiltration rates as high as 33 mm/hr with a high intensity rainfall event.

In a study more relevant to the present site, Luxmoore et al (1981) documented an average infiltration rate of 2 cm/day into fractured shale by means of an infiltrometer experiment. A copy of this paper is included in this report (Appendix IV). The site where Luxmoore et al conducted the experiment is underlain by weathered shales. This investigation was aimed at determining the infiltration rate into weathered shales with the view to quantifying the rate of leachate migration from a local radionuclide waste disposal site into the underlying shales. The weathered shales at the study site have hydraulic conductivities ranging from 10^{-3} to 10^{-7} cm/sec, while for unweathered shale the values varied from 10^{-5} to 10^{-8} cm/sec. The study reports a geometric mean infiltration rate of 2 cm/day. CRA considers this value does not reflect an annual average condition. However, an

infiltration rate of 2 cm/day does indicate that infiltration into fractured shale can be substantial.

In order to provide an estimate of the exfiltration from the landfill into the bedrock, a simplified water balance was developed. This system is shown schematically on Figure C.6.1 (p. 156) For this system, only lateral flow in the bedrock is considered. CRA has demonstrated from an analysis of the pumping test data an average bulk hydraulic conductivity of 2×10^{-4} cm/sec for the upper weathered and fractured bedrock. The horizontal hydraulic gradient in this unit is 0.03. For a 15 m and 30 m thickness, a flux of 28.4 m³/yr to 56.8 m³/yr, respectively, is calculated. Dividing by a landfill length of 1250 m gives a potential exfiltration into the bedrock of 22.7 to 45.4 mm/yr, respectively, as shown on Figure C.6.1a (p. 156).

This calculation assumes that all of the exfiltration entering the system migrates laterally off site. Flow into the system from upgradient is not considered nor is flow downward from the system (arrows C and D on Figure C.6.1). This leads to a conservative estimate of exfiltration. Upgradient of the proposed landfill, the shale is mantled by a low permeable till. Exfiltration through this till and into the underlying shale is expected to be less than into the exposed weathered and fractured shale. The steep vertical gradients at the site indicate that a

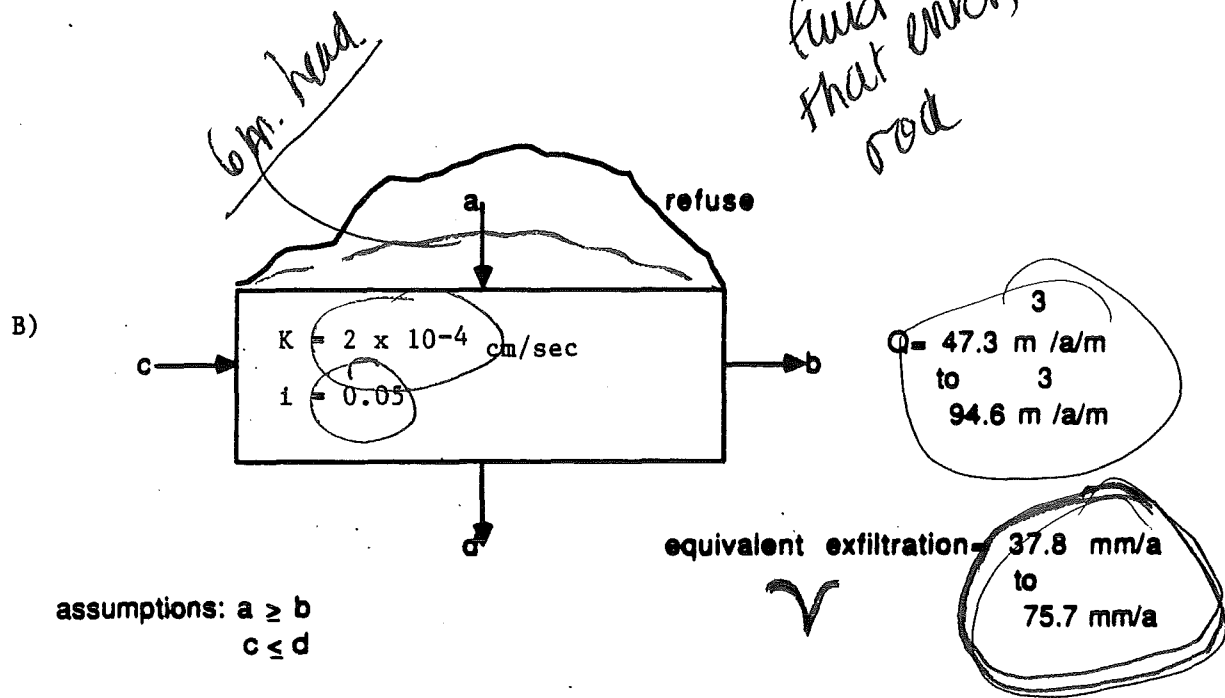
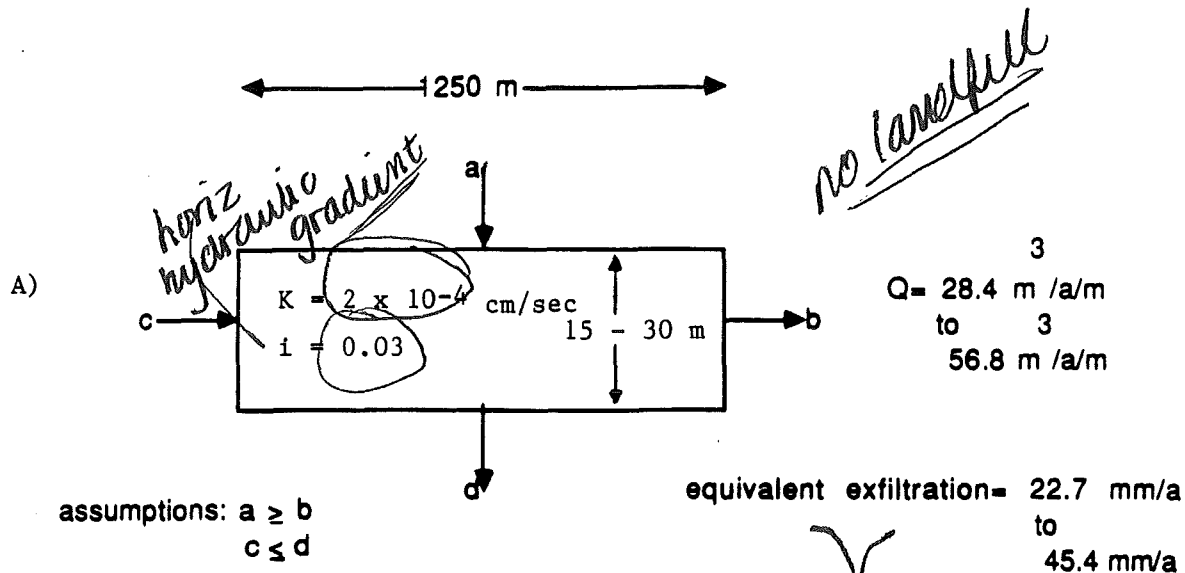


figure C.6.1

EXFILTRATION SCHEMATIC

Site F' - Hydrogeology - City of Burlington

indicate that a portion of the exfiltration migrates to depth and not laterally in the weathered and fractured bedrock mantle.

It is expected therefore that upgradient groundwater flow into this system will be equal to or less than the groundwater flux migrating to depth below this system. Therefore CRA considers that the above estimates of exfiltration into the shale (22.7 to 45.4 mm/yr) are conservative.

Mounding of leachate within the landfill will effectively increase the horizontal hydraulic gradient in the underlying shale. For an assumed leachate mound of approximately 6 m and assuming the hydraulic head in the shale at a point 300 m downgradient does not change, the horizontal hydraulic gradient increases to approximately 0.05. The groundwater flux in the underlying shale increases to 47.3 to 94.6 m³/yr respectively, for the thicknesses considered above. Again, assuming that all the exfiltration from the landfill exits the system laterally, a potential exfiltration rate of 37.8 to 75.7 mm/yr, respectively is calculated (Figure C.6.1b).

This scenario does not provide exact measure of exfiltration but does demonstrate that a conservative exfiltration estimate from the base of the landfill in the range of 50 to 100 mm/yr is reasonable.

The exfiltration through a compacted clay liner can be calculated readily. For a design hydraulic conductivity of 1×10^{-8} cm/sec for the liner and a hydraulic gradient of 1.0 over the thickness of the liner an exfiltration rate of 3.15 mm/yr results. (It is to be noted that this value exceeds the exfiltration estimates of Trow and Rowe and yet the surface materials at Site F do not consist of compacted clays with a hydraulic conductivity of 1×10^{-8} cm/sec but rather weathered and fractured bedrock with hydraulic conductivity of 2×10^{-4} cm/sec.).

A conservative estimate of the exfiltration from the landfill into fractured permeable shale with a hydraulic conductivity of 2×10^{-4} cm/sec is shown above to potentially vary from 37.8 to 75.7 mm/yr. CRA argues that an exfiltration rate of 50 to 100 mm/yr from beneath landfill is therefore a reasonable conservative estimate. For rates of 50 to 100 mm/yr, a compacted clay liner would therefore reduce the exfiltration from the landfill by 46.85 to

96.85 mm/yr.

An estimate of the efficiency of the leachate collection system can now be made for assumed infiltration rates into the landfill of 152.4 to 304.8 mm/yr (6 to 12 inches/yr).

Without a liner, the collection system efficiency could range from 34.4 to 83.6 percent. With a liner the collection system efficiency is increased to 97.9 to 99.0 percent for an infiltration into the landfill of 152.4 to 304.8 mm/yr, respectively.

CRA does recognize that the exfiltration rate at Site F is unknown and that ~~available best estimates vary~~ dramatically. Thus the effects of a liner cannot be demonstrated conclusively. However, given the uncertainties associated with quantifying the exfiltration rate and groundwater velocities at this Site; and further, that the bulk hydraulic conductivity of the weathered and fractured shale underlying the proposed landfill is calculated to be 2×10^{-4} cm/sec; CRA concludes that an alternate design incorporating a compacted clay liner will provide a significant degree of assurance as to the ability to collect leachate and control leachate egress from the landfill.

CRA concludes that the alternate landfill design is hydrogeologically suitable because;

- 1) The migration of leachate through the liner can be predicted.

- 2) The performance of the liner and the underlying drainage layer can be monitored.
- 3) In the event of a failure of the liner system, the underlying drainage layer serves as the primary contingency to control leachate migration.
- 4) The uncertainties associated with the hydrogeological definition of the site and interpretation of the existing monitoring data are offset by the more elaborate engineering considered for this alternate design.

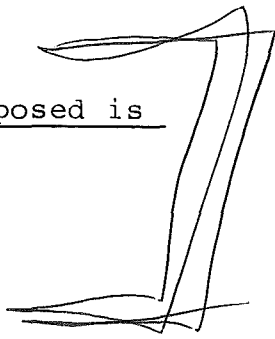
C.7.0 SUMMARY

The ^{local} groundwater flow system at the site is shown to be complex. Discrete fracture sets provide the pathway for active groundwater flow at this site. The distribution, orientation, aperture width and degree of connection of these fracture sets is unknown. As indicated by the hydraulic head profiles and the tritium data, this distribution is complex and cannot be adequately delineated. The site cannot be subdivided into various hydrostratigraphic units each with a specific hydraulic characterization.

CRA concludes that the pathway by which contaminants potentially may migrate off site has not been defined.

In addition, this site does not meet the criteria of predictability and ability to monitor, which CRA considers as fundamental for a hydrogeologically suitable landfill site.

It is concluded that the site as proposed is not hydrogeologically suitable for landfilling.

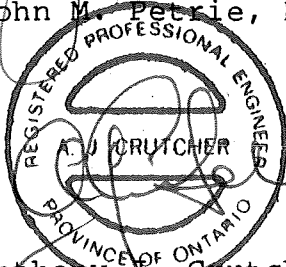


For an alternative landfill design consisting of a suitable liner and an underdrain system, the site is hydrogeologically suitable for landfilling.

All of which is respectfully submitted,
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REFERENCES

Bouma, J., L.W. Dekker, and J.H.M. Wosten (1978). A Case Study on Infiltration into a Dry Clay Soil II. Physical Measurements. *Geoderma*, 20, 41-51.

Cherry, J.A. and Lush, D.L. 1980, Hydrological Suitability of the NSPL Landfill Site, Burlington, Ontario. Prepared for The Regional Municipality of Halton, Oakville, Ontario, 54pp.

Conestoga-Rovers & Associates (1987), Review of 1986 Monitoring Report - Halton Regional Landfill - Burlington. Prepared for the City of Burlington.

Conestoga-Rovers & Associates (1987), Updated 1986 Monitoring Report, Bayview Landfill Site, City of Burlington. Prepared for the City of Burlington.

Gartner Lee Associates Limited (1984), Hydrogeological Study - Burlington Landfill Site Continued Use - Prepared for the Regional Municipality of Halton. Exhibit H-217.

Gartner Lee Associates Limited (1986a), Burlington Landfill - 1985 Monitoring Report for the Regional Municipality of Halton, (March 1986).

Gartner Lee Limited (1986b), Burlington Regional Landfill Plume Delineation Program Report for the Regional Municipality of Halton (December 1986). Exhibit H-83.

Heweston, J.P. (1985), An Investigation of the Groundwater Zone in Fractured Shale at a Landfill. M.Sc. Thesis Prepared for the University of Waterloo. Exhibit E-214.

Hydrology Consultants (1986), Hydrogeologic Report for Site F - Burlington Halton Region Landfill Technical Report. Text-Volume 1, Appendices - Volume 2. Exhibits H-40, H-40A.

Luxmore R.J., B.P. Spalding and I.M. Munroe (1981), Areal Variation and Chemical Modification of Weathered Shale Infiltration Characteristics. *Soil Sci. Soc. Am. J.* Vol. 45 pp687-691, 1981.

Morrison Beatty Limited (1979). Progress Report on Phase I Hydrogeologic Study National Sewer Pipe Ltd. Proposed Landfill, Burlington.

Mase, C.W., G. Thompson, J.A. Cherry, R. Blackport, L. Smith (1987). Evaluation of Contaminant Velocity in Low-Permeability Fractured Shale. From 1987 Technology Transfer Conference Proceedings. Part C: Liquid and Solid Waste Research.

Morrison Beatty Limited (1980), Phase II - Hydrogeologic Investigation National Sewer Pipe Ltd. Landfill Site, Burlington. Exhibit N-186

Morrison Beatty Limited (1981), Phase III - Hydrogeologic Investigation National Sewer Pipe Ltd. Landfill Site, Burlington. Exhibit N-185

Sibul, U., K.T. Wang, D. Vallery (1977). Ground-Water Resources of the Duffins Creek - Rouge River Drainage Basins. Ministry of the Environment, Water Resources Branch, Water Resources Report 8.

Thomas, G.W. and R.E. Phillips (1979). Consequences of Water Movement in Macropores. Journal of Environmental Quality, 2, 149-153.

APPENDIX I

VERTICAL HYDRAULIC GRADIENTS

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels												
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86	17-Apr-86
HC1-1-85		157.10	155.60	156.35	174.39	174.31			175.92		177.06		177.17		177.41	177.92	177.44
HC1-2-85		168.70	167.20	167.95	180.28	180.34			180.14		179.65		179.70		179.79	179.99	180.14
HC1-3-85		184.60	180.60	w.t.	184.18	184.10			183.79		183.91		183.88		183.68	183.92	183.98
		Vertical Gradient: (1-2)			-0.508	-0.520			-0.364		-0.223		-0.218		-0.205	-0.178	-0.233
		(1-3)			-0.352	-0.353			-0.287		-0.249		-0.244		-0.229	-0.218	-0.237
		(2-3)			-0.240	-0.233			-0.230		-0.267		-0.262		-0.247	-0.246	-0.240
HC3-1-85		111.30	109.80	110.55	138.22	137.71			137.28		137.82		137.74		137.73	126.92	126.79
HC3-2-85		126.60	125.10	125.85	135.50	135.32			135.26		135.45		135.58		135.64	134.97	135.69
HC3-3-85		130.20	128.70	129.45	135.61	135.46			135.30		135.61		135.60		135.57	135.02	135.98
HC3-4-85		134.20	132.70	133.45													
HC3-5-85		138.20	136.60	137.40													139.42
		Vertical Gradient: (1-2)			0.178	0.156			0.132		0.155		0.141		0.137	-0.526	-0.582
		(1-3)			0.138	0.119			0.105		0.117		0.113		0.114	-0.429	-0.486
		(1-4)															-0.470
		(1-5)															-0.081
		(2-3)			-0.031	-0.039			-0.011		-0.044		-0.006		0.019	-0.014	-0.081
		(2-4)															
		(2-5)															
		(3-4)															-0.323
		(3-5)															
		(4-5)															-0.433
HC4-1-85		107.80	106.90	107.35	122.60	122.50			122.61		122.89		122.88		122.98	122.05	122.96
HC4-2-85		120.90	120.00	120.45	133.02	132.75			132.80		133.01		132.99		132.95	132.44	133.18
HC4-3-85		127.10	126.20	126.65	133.14	132.86			132.92		133.14		133.15		133.18	132.80	133.60
HC4-4-85		136.30	131.40	w.t.	134.99	134.76			134.78		134.74		134.84		134.99	134.69	135.44
		Vertical Gradient: (1-2)			-0.795	-0.782			-0.778		-0.773		-0.772		-0.761	-0.793	-0.780
		(1-3)			-0.546	-0.537			-0.534		-0.531		-0.532		-0.528	-0.557	-0.551
		(1-4)			-0.448	-0.447			-0.444		-0.433		-0.435		-0.435	-0.462	-0.444
		(2-3)			-0.019	-0.018			-0.019		-0.021		-0.026		-0.037	-0.058	-0.068
		(2-4)			-0.135	-0.140			-0.138		-0.121		-0.129		-0.140	-0.158	-0.151
		(3-4)			-0.222	-0.234			-0.229		-0.198		-0.206		-0.217	-0.235	-0.209

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels													
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86	17-Apr-86	
HC5-1-85		96.60	95.10	95.85													114.97	
HC5-2-85		108.80	107.30	108.05													118.97	
HC5-3-85		118.20	116.70	117.45													124.70	
HC5-4-85		124.80	120.20	122.50	125.49	125.26		125.06		125.43		125.33		125.28	124.98		125.83	
		Vertical Gradient: (1-2)															-0.328	
		(1-3)															-0.450	
		(1-4)															-0.408	
		(2-3)															-0.610	
		(2-4)															-0.475	
		(3-4)															-0.224	
HC6-1-85		98.20	97.30	97.75	117.49	117.55		117.54		118.01		118.16		118.27	118.41		118.37	
HC6-2-85		107.40	106.50	106.95	120.74	120.74		120.84		120.96		120.96		120.98	121.10		120.86	
HC6-3-85		111.00	110.10	110.55	120.67	120.38		120.55		120.74		120.78		120.85	120.89		121.11	
HC6-4-85		120.50	119.60	120.05	120.68	120.55		120.55		120.79		120.68		120.59	120.64		121.16	
HC6-5-85		125.70	124.80	w.t.						125.19		125.45		125.65	125.84		126.88	
		Vertical Gradient: (1-2)			-0.353	-0.347		-0.359		-0.321		-0.304		-0.295	-0.292		-0.271	
		(1-3)			-0.248	-0.221		-0.235		-0.213		-0.205		-0.202	-0.194		-0.214	
		(1-4)			-0.143	-0.135		-0.135		-0.125		-0.113		-0.104	-0.100		-0.125	
		(1-5)								-0.262		-0.263		-0.265	-0.265		-0.292	
		(2-3)			0.019	0.100		0.081		0.061		0.050		0.036	0.058		-0.069	
		(2-4)			0.005	0.015		0.022		0.013		0.021		0.030	0.035		-0.023	
		(2-5)								-0.232		-0.243		-0.250	-0.251		-0.302	
		(3-4)				-0.001	-0.018		0.000		-0.005		0.011		0.027	0.026		-0.005
		(3-5)								-0.304		-0.313		-0.318	-0.324		-0.353	
		(4-5)								-0.856		-0.883		-0.904	-0.898		-0.837	
HC7-1-85			104.70	103.20	103.95	115.57	115.36		115.48		115.57		115.66		115.74	115.85		112.91
HC7-2-85			112.50	111.00	111.75	121.45	121.34		121.31		121.54		121.66		121.80	121.86		121.46
HC7-2A-85						121.41	121.31		121.28		121.51		121.70		121.94	121.97		121.49
HC7-3-85			127.30	123.30	w.t.	126.85	126.58		126.29		126.96		126.96		126.96	127.01		127.95
	Vertical Gradient: (1-2)				-0.754	-0.767		-0.747		-0.765		-0.769		-0.777	-0.771		-1.096	
	(1-2A)																	
	(1-3)				-0.493	-0.496		-0.484		-0.495		-0.491		-0.488	-0.484		-0.627	
	(2-2A)																	
	(2-3)				-0.358	-0.353		-0.343		-0.356		-0.348		-0.339	-0.337		-0.401	
	(2A-4)																	

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels												
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86	17-Apr-86
HC8-1-85		88.90	87.10	88.00	110.99	110.84			104.07		111.14		111.19		111.26	111.28	109.46
HC8-2-85		102.50	100.90	101.70	115.84	115.71			115.97		116.03		116.09		116.18	116.32	115.59
HC8-3-85		109.10	107.30	108.20	118.24	118.21											117.87
HC8-4-85		115.60	111.60	113.60	118.14	118.25											117.84
		Vertical Gradient: (1-2)			-0.354	-0.355		-0.869		-0.357		-0.358		-0.359	-0.368	-0.447	
		(1-3)			-0.359	-0.365										-0.416	
		(1-4)			-0.279	-0.289										-0.327	
		(2-3)			-0.369	-0.385										-0.351	
		(2-4)			-0.193	-0.213										-0.189	
		(3-4)			0.019	-0.007										0.006	
HC9-1-85		76.60	75.10	75.85	102.08	101.94			86.45		96.43		99.73		101.02	101.09	102.91
HC9-2-85		90.00	89.10	89.55	103.48	103.75			103.38		103.61		103.83		104.30	104.35	104.31
HC9-3-85		98.00	96.50	97.25	104.45	104.35			104.22		104.32		104.36		104.39	104.39	104.39
HC9-4-85		103.80	100.70	102.25	104.45	104.19			104.19		104.16		104.36		104.47	104.57	104.39
		Vertical Gradient: (1-2)			-0.102	-0.132		-1.236		-0.524		-0.299		-0.239	-0.238	-0.102	
		(1-3)			-0.111	-0.113		-0.830		-0.369		-0.216		-0.157	-0.154	-0.069	
		(1-4)			-0.090	-0.085		-0.672		-0.293		-0.175		-0.131	-0.132	-0.056	
		(2-3)			-0.126	-0.078		-0.109		-0.092		-0.069		-0.012	-0.005	-0.010	
		(2-4)			-0.076	-0.035		-0.064		-0.043		-0.042		-0.013	-0.017	-0.006	
		(3-4)			0.000	0.032		0.006		0.032		0.000		-0.016	-0.036	0.000	
HC10-1-85		76.60	75.10	75.85	93.07	93.03			93.04		93.02		92.97		92.92	92.97	93.02
HC10-2-85		85.20	84.30	84.75	93.08	93.09			93.09		93.11		93.12		93.10	93.13	93.02
HC10-3-85		93.40	88.70	w.t.	92.28	92.18			91.98		92.11		92.17		92.25	92.43	92.28
		Vertical Gradient: (1-2)			-0.001	-0.007		-0.006		-0.010		-0.017		-0.020	-0.018	0.000	
		(1-3)			0.048	0.052		0.066		0.056		0.049		0.041	0.033	0.045	
		(2-3)			0.106	0.122		0.154		0.136		0.128		0.113	0.091	0.098	

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels											
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86
HC11-1-85		73.20	71.70	72.45	92.34	92.22		92.24		92.31		92.29		92.27	92.45	92.09
HC11-2-85		91.70	87.10	w.t.	89.57	89.49		89.50		89.51		89.48		89.55	89.70	89.48
		Vertical Gradient: (1-2)			0.162	0.160		0.161		0.164		0.165		0.159	0.159	0.153
HC12-1-85		62.89	61.00	61.95	84.36	84.31		84.05		84.01		84.09		84.21	84.38	84.57
HC12-2-85		78.60	77.50	78.05	84.35	84.22		84.10		84.08		84.06		84.07	84.25	84.07
HC12-3-85		83.90	80.80	82.35												84.06
		Vertical Gradient: (1-2)			0.001	0.006		-0.003		-0.004		0.002		0.009	0.008	0.031
		(1-3)														0.025
		(2-3)														0.002
HC13-1-85		48.30	47.40	47.85	75.15	75.03		74.93		75.02		75.00		75.06	75.41	75.14
HC13-2-85		69.10	68.20	68.65	75.26	75.13		75.12		75.17		75.05		75.29	75.30	75.30
HC13-3-85		75.50	71.50	w.t.	75.20	75.07		75.10		75.11		75.03		75.23	75.26	75.24
		Vertical Gradient: (1-2)			-0.005	-0.005		-0.009		-0.007		-0.002		-0.011	0.005	-0.008
		(1-3)			-0.002	-0.001		-0.006		-0.003		-0.001		-0.006	0.005	-0.004
		(2-3)			0.009	0.009		0.003		0.009		0.003		0.009	0.006	0.009
HC14-1-85		84.20	83.30	83.75											108.03	108.31
HC14-2-85		91.80	90.30	91.05											108.32	108.32
HC14-3-85		95.50	94.00	94.75											108.57	108.63
HC14-4-85		102.20	99.00	w.t.											108.71	109.14
		Vertical Gradient: (1-2)													-0.040	-0.001
		(1-3)													-0.049	-0.029
		(1-4)													-0.027	-0.033
		(2-3)													-0.068	-0.084
		(2-4)													-0.022	-0.045
		(3-4)													-0.010	-0.035

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels												
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86	17-Apr-86
HC15-1-85		87.30	85.80	86.55												110.06	109.35
HC15-2-85		94.30	93.70	94.00												109.31	109.42
HC15-3-85		100.40	99.80	100.10												109.97	109.99
HC15-4-85		107.10	105.90	w.t.												110.85	110.96
		Vertical Gradient: (1-2)														0.101	-0.009
				(1-3)												0.007	-0.047
				(1-4)												-0.033	-0.066
				(2-3)												-0.108	-0.093
				(2-4)												-0.091	-0.091
				(3-4)												-0.082	-0.089
HC16-1-85		57.30	56.40	56.85												76.13	74.49
HC16-2-85		66.50	65.60	66.05												75.25	75.39
HC16-3-85		69.00	68.10	68.55												74.36	76.21
		Vertical Gradient: (1-2)														0.096	-0.098
				(1-3)												0.151	-0.147
				(2-3)												0.356	-0.328

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels												
					02-Apr-85	16-Apr-85	07-May-85	17-Jun-85	12-Jul-85	20-Aug-85	29-Aug-85	05-Sep-85	25-Sep-85	18-Oct-85	01-Nov-85	18-Nov-85	04-Dec-85
MB-12-3		103.10	102.20	102.65	119.50	119.43	119.44	119.24	119.17	118.96	118.98	118.98	119.06	118.99	118.99	119.18	119.12
MB-12-2		112.25	111.33	111.79	120.90	120.89	120.35	120.52	120.43	120.22	120.03	120.01	119.93	119.99	120.45	120.64	120.64
MB-12-1		120.48	117.43	118.96	121.50	121.42	121.61	121.65	121.54	121.36	121.11	121.06	120.96	120.92	121.61	121.82	121.80
		Vertical Gradient: (3-2)			-0.153	-0.160	-0.100	-0.140	-0.138	-0.138	-0.115	-0.113	-0.095	-0.109	-0.160	-0.160	-0.166
		(3-1)			-0.123	-0.122	-0.133	-0.148	-0.145	-0.147	-0.131	-0.128	-0.117	-0.118	-0.161	-0.162	-0.164
		(2-1)			-0.084	-0.074	-0.176	-0.158	-0.155	-0.159	-0.151	-0.147	-0.144	-0.130	-0.162	-0.165	-0.162
MB-13-3		94.70	93.80	94.25	105.07	105.52	103.87	104.68	104.35	104.59	104.63	104.72	104.85	104.76	104.79	104.70	104.77
MB-13-2		103.75	102.83	103.29	113.30	113.38	113.47	113.33	113.21	112.99	113.01	113.00	113.00	113.05	113.08	113.13	113.38
MB-13-1		111.88	108.83	110.36	113.40	113.26	113.07	111.55	111.35	110.11	109.51	109.40	109.20	109.12	109.43	112.44	113.17
		Vertical Gradient: (3-2)			-0.910	-0.869	-1.062	-0.957	-0.980	-0.929	-0.927	-0.916	-0.902	-0.917	-0.917	-0.933	-0.952
		(3-1)			-0.517	-0.481	-0.571	-0.427	-0.435	-0.343	-0.303	-0.291	-0.270	-0.271	-0.288	-0.481	-0.522
		(2-1)			-0.014	0.017	0.057	0.252	0.263	0.408	0.495	0.510	0.538	0.556	0.517	0.098	0.030
MB-16-3		95.86	97.86	96.86	107.40	107.07	106.85	105.56	105.21	105.71	105.61	106.51	106.59	106.59	106.67	106.83	107.30
MB-16-2		100.34	102.44	101.39	115.33	115.16	114.02	115.29	115.03	113.79	113.58	113.55	113.50	113.53	113.66	115.40	114.94
MB-16-1		109.48	114.00	111.74	117.10	116.97	116.44	115.60	115.27	114.96	114.72	114.42	114.37	114.15	114.39	116.77	116.97
		Vertical Gradient: (3-2)			-1.751	-1.786	-1.583	-2.148	-2.168	-1.784	-1.759	-1.554	-1.525	-1.532	-1.543	-1.892	-1.687
		(3-1)			-0.652	-0.665	-0.644	-0.675	-0.676	-0.622	-0.612	-0.532	-0.523	-0.508	-0.519	-0.668	-0.650
		(2-1)			-0.171	-0.175	-0.234	-0.030	-0.023	-0.113	-0.110	-0.084	-0.084	-0.060	-0.071	-0.132	-0.196

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels											
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86
MB-12-3		103.10	102.20	102.65	119.20	119.27	119.30		119.50		119.45		119.47		119.68	118.94
MB-12-2		112.25	111.33	111.79	120.60	120.62	120.60		120.71		120.60		120.58		120.82	120.63
MB-12-1		120.48	117.43	118.96	121.77	121.72	121.74		121.81		121.75		121.76		121.83	121.85
		Vertical Gradient: (3-2)			-0.153	-0.148	-0.142		-0.132		-0.126		-0.121		-0.125	-0.185
		(3-1)			-0.158	-0.150	-0.150		-0.142		-0.141		-0.140		-0.132	-0.178
		(2-1)			-0.163	-0.154	-0.159		-0.154		-0.161		-0.165		-0.141	-0.170

MB-13-3		94.70	93.80	94.25	104.95	104.85	104.99		105.08		104.99		104.54		104.57	109.12
MB-13-2		103.75	102.83	103.29	113.50	113.50	113.50		113.70		113.50		112.26		112.38	110.33
MB-13-1		111.88	108.83	110.36	113.19	113.05	112.99		113.25		113.19		113.22		113.28	113.30
		Vertical Gradient: (3-2)			-0.946	-0.957	-0.941		-0.954		-0.941		-0.854		-0.864	-0.134
		(3-1)			-0.512	-0.509	-0.497		-0.507		-0.509		-0.539		-0.541	-0.260
		(2-1)			0.044	0.064	0.072		0.064		0.044		-0.136		-0.127	-0.420

MB-16-3		95.86	97.86	96.86	107.56	107.41	107.25		107.51		107.84		108.00		108.03	109.93
MB-16-2		100.34	102.44	101.39	113.98	113.89	114.18		114.33		114.22		114.45		114.94	113.39
MB-16-1		109.48	114.00	111.74	116.70	116.35	116.48		116.48		116.51		116.45		116.63	116.81
		Vertical Gradient: (3-2)			-1.417	-1.430	-1.530		-1.506		-1.408		-1.424		-1.525	-0.764
		(3-1)			-0.614	-0.601	-0.620		-0.603		-0.583		-0.568		-0.578	-0.462
		(2-1)			-0.263	-0.238	-0.222		-0.208		-0.221		-0.193		-0.163	-0.330

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels											
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86 17-Apr-86
WAT-3-12		93.438	92.298	92.868												101.88
WAT-3-11		96.198	95.048	95.623												115.80
WAT-3-10		98.948	97.808	98.378												123.49
WAT-3-9		101.708	101.238	101.473												124.10
WAT-3-8		104.468	103.328	103.898												126.07
WAT-3-7		107.228	106.088	106.658												122.91
WAT-3-6		109.988	108.848	109.418												123.46
WAT-3-5		112.748	111.608	112.178												124.57
WAT-3-4		115.508	114.358	114.933												127.91
WAT-3-2		121.018	119.878	120.448												123.29
WAT-3-1		123.778	122.638	123.208												128.21
		Vertical Gradient: (12-11)														-5.053
				(11-10)												-2.791
				(10-9)												-0.197
				(9-8)												-0.812
				(8-7)												1.145
				(7-6)												-0.199
				(6-5)												-0.402
				(5-4)												-1.212
				(4-2)												0.838
				(2-1)												-1.783
WAT-5-12		92.000	90.930	91.465												123.90
WAT-5-11		94.840	93.690	94.265												123.78
WAT-5-10		97.590	96.450	97.020												123.97
WAT-5-9		100.350	99.210	99.780												124.13
WAT-5-8		102.710	102.270	102.490												123.87
WAT-5-7		105.870	104.730	105.300												124.11
WAT-5-6		108.630	107.430	108.030												124.10
WAT-5-5		111.570	110.240	110.905												124.31
WAT-5-4		114.150	113.000	113.575												124.99
WAT-5-3		116.900	115.760	116.330												125.71
WAT-5-2		119.660	118.520	119.090												126.83
WAT-5-1		127.510	127.510	127.510												127.09
		Vertical Gradient: (12-11)														0.043
				(11-10)												-0.069
				(10-9)												-0.058
				(9-8)												0.096
				(8-7)												-0.085
				(7-6)												0.004
				(6-5)												-0.073
				(5-4)												-0.255
				(4-3)												-0.261
				(3-2)												-0.406
				(2-1)												-0.031

VERTICAL HYDRAULIC GRADIENTS

Well Number	Hydrostratigraphic Unit	Top of Screen	Bottom Of Screen	Mid Point	Water Levels												
					21-Dec-85	05-Jan-86	17-Jan-86	18-Jan-86	05-Feb-86	06-Feb-86	18-Feb-86	19-Feb-86	05-Mar-86	09-Mar-86	24-Mar-86	15-Apr-86	17-Apr-86
WAT-7-12		82.237	81.167	81.702	108.45	108.16		101.51		108.43		108.50		108.73	108.72	109.26	109.26
WAT-7-11		85.077	83.927	84.502	101.35	101.23		101.51		101.60		101.72		101.86	101.88	102.98	102.98
WAT-7-10		87.827	86.687	87.257	92.25	92.20				92.08		92.34		92.42	92.44	92.58	92.58
WAT-7-9		90.587	89.447	90.017	98.63	98.75		98.85		99.03		99.10		99.22	99.21	99.53	99.53
WAT-7-8		93.347	92.207	92.777	108.55	108.51		108.79		108.59		109.13		109.33	109.36	109.95	109.95
WAT-7-7		96.107	94.967	95.537	109.90	110.10		110.21		110.33		110.53		110.73	110.77	112.55	110.95
WAT-7-6		98.867	97.727	98.297	114.05	114.71		114.65		114.67		114.10		113.61	113.64	110.46	110.46
WAT-7-5		101.807	100.477	101.142	114.37	114.92		114.88		114.85		114.25		113.66	113.72	110.54	110.54
WAT-7-4		104.387	103.237	103.812	115.52	115.57		115.45		115.49		115.60		114.90	114.94	112.79	112.79
WAT-7-3		107.137	105.997	106.567	115.92	116.11		116.10		116.23		116.30		115.60	115.66	114.22	114.22
WAT-7-2		109.897	108.757	109.327	116.12	116.22		116.18		116.35		116.37		115.96	116.07	114.94	114.94
WAT-7-1		111.517	111.517	111.517	117.47	116.58		116.57		116.70		116.73		116.17	116.32	115.12	115.12
		Vertical Gradient: (12-11)			2.536	2.475		0.000		2.439		2.421		2.454	2.443	2.243	2.243
				(11-10)	3.303	3.278				3.456		3.405		3.426	3.426	3.775	3.775
				(10-9)	-2.312	-2.373				-2.518		-2.449		-2.464	-2.453	-2.518	-2.518
				(9-8)	-3.594	-3.536		-3.601		-3.464		-3.634		-3.663	-3.678	-3.775	-3.775
				(8-7)	-0.489	-0.576		-0.514		-0.630		-0.507		-0.507	-0.511	-0.942	-0.362
				(7-6)	-1.504	-1.670		-1.609		-1.572		-1.293		-1.043	-1.040	0.757	0.178
				(6-5)	-0.112	-0.074		-0.081		-0.063		-0.053		-0.018	-0.028	-0.028	-0.028
				(5-4)	-0.431	-0.243		-0.213		-0.240		-0.506		-0.464	-0.457	-0.843	-0.843
				(4-3)	-0.145	-0.196		-0.236		-0.269		-0.254		-0.254	-0.261	-0.519	-0.519
				(3-2)	-0.072	-0.040		-0.029		-0.043		-0.025		-0.130	-0.149	-0.261	-0.261
				(2-1)	-0.616	-0.164		-0.178		-0.160		-0.164		-0.096	-0.114	-0.082	-0.082

APPENDIX II

RESULTS OF FLONET SENSITIVITY ANALYSIS

NOTES:

1. Water levels determined from flow model
2. Measured water levels - April 1986
(Trow, Appendix 5, Exhibit H-40A)

FLONET SENSITIVITY ANALYSIS

		2	1		1	
		Measured	Case 1	Difference	Case 2	Difference
HC-1	1	177.44	177.63	0.19	177.81	0.37
	2	180.14	177.66	2.48	177.85	2.29
	3	183.98	183.98	0.00	183.98	0.00
HC-2	1	167.99	167.99	0.00	167.99	0.00
HC-4	1	122.96	117.47	5.49	133.47	10.51
	2	133.18	135.25	2.07	135.41	2.23
	3	133.60	135.35	1.75	135.43	1.83
	4	135.44	135.44	0.00	135.44	0.00
HC-5	1	114.97	112.84	2.13	122.33	7.36
	2	118.97	112.84	6.13	122.33	3.36
	3	124.70	121.51	3.19	124.66	0.04
	4	125.83	125.83	0.00	125.83	0.00
HC-8	1	109.46	111.58	2.12	119.41	9.95
	2	115.59	112.62	2.97	119.15	3.56
	3	117.87	113.66	4.21	118.89	1.02
	4	117.84	117.84	0.00	117.84	0.00
HC-14	1	108.31	105.77	2.54	107.92	0.39
	2	108.32	105.77	2.55	107.92	0.40
	3	108.63	105.77	2.86	107.92	0.71
	4	109.14	109.14	0.00	109.14	0.00
HC-11	1	92.09	89.42	2.67	89.42	2.67
	2	89.48	89.48	0.00	89.48	0.00
HC-16	1	74.49	75.98	1.49	75.98	1.49
	2	75.39	76.09	0.70	76.09	0.70
	3	76.21	76.21	0.00	76.21	0.00

Sum of Differences:

45.54

48.88

FLONET SENSITIVITY ANALYSIS

		2	1		1	
		Measured	Case 3	Difference	Case 4	Difference
HC-1	1	177.44	177.49	0.05	177.63	0.19
	2	180.14	177.53	2.61	177.67	2.47
	3	183.98	183.98	0.00	183.98	0.00
HC-2	1	167.99	167.99	0.00	167.99	0.00
	2	167.99	167.99	0.00	167.99	0.00
HC-4	1	122.96	104.32	18.64	131.64	8.68
	2	133.18	135.12	1.94	135.09	1.91
	3	133.60	135.29	1.69	135.27	1.67
	4	135.44	135.44	0.00	135.44	0.00
HC-5	1	114.97	103.58	11.39	121.51	6.54
	2	118.97	103.58	15.39	121.51	2.54
	3	124.70	118.44	6.26	122.24	2.46
	4	125.83	125.83	0.00	125.83	0.00
HC-8	1	109.46	103.37	6.09	119.01	9.55
	2	115.59	105.78	9.81	118.96	3.37
	3	117.87	108.19	9.68	118.91	1.04
	4	117.84	117.84	0.00	117.84	0.00
HC-14	1	108.31	102.40	5.91	108.34	0.03
	2	108.32	102.40	5.92	108.34	0.02
	3	108.63	102.40	6.23	108.34	0.29
	4	109.14	109.14	0.00	109.14	0.00
HC-11	1	92.09	89.42	2.67	89.42	2.67
	2	89.48	89.48	0.00	89.48	0.00
HC-16	1	74.49	75.98	1.49	75.98	1.49
	2	75.39	76.09	0.70	76.09	0.70
	3	76.21	76.21	0.00	76.21	0.00

Sum of Differences:

106.47

45.62

FLONET SENSITIVITY ANALYSIS

		2	1		1	
		Measured	Case 5	Difference	Case 6	Difference
HC-1	1	177.44	177.60	0.16	177.66	0.22
	2	180.14	177.64	2.50	177.69	2.45
	3	183.98	183.98	0.00	183.98	0.00
HC-2	1	167.99	167.99	0.00	167.99	0.00
HC-4	1	122.96	106.45	16.51	135.98	13.02
	2	133.18	135.40	2.22	135.48	2.30
	3	133.60	135.43	1.83	135.47	1.87
	4	135.44	135.44	0.00	135.44	0.00
HC-5	1	114.97	105.19	9.78	122.87	7.90
	2	118.97	105.19	13.78	122.87	3.90
	3	124.70	124.84	0.14	123.37	1.33
	4	125.83	125.83	0.00	125.83	0.00
HC-8	1	109.46	104.86	4.60	119.65	10.19
	2	115.59	107.95	7.64	119.58	3.99
	3	117.87	111.04	6.83	119.50	1.63
	4	117.84	117.84	0.00	117.84	0.00
HC-14	1	108.31	103.34	4.97	108.05	0.26
	2	108.32	103.34	4.98	108.04	0.28
	3	108.63	103.34	5.29	108.04	0.59
	4	109.14	109.14	0.00	109.14	0.00
HC-11	1	92.09	89.42	2.67	89.42	2.67
	2	89.48	89.48	0.00	89.48	0.00
HC-16	1	74.49	75.98	1.49	75.98	1.49
	2	75.39	76.09	0.70	76.09	0.70
	3	76.21	76.21	0.00	76.21	0.00
Sum of Differences:				----- 86.09	----- 54.79	

FLONET SENSITIVITY ANALYSIS

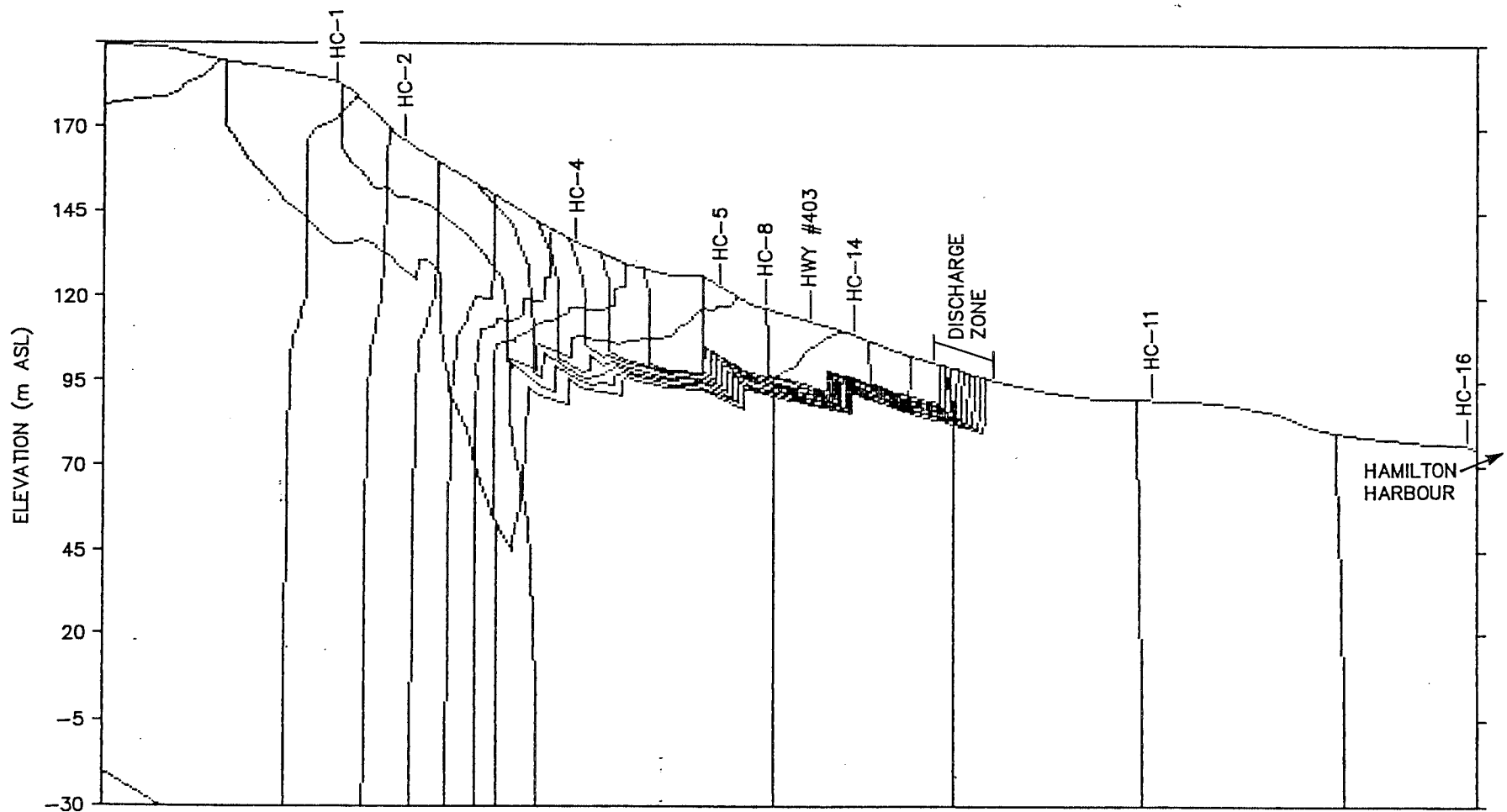
		2 Measured	1 Case 7 Difference		1 Case 8 Difference	
HC-1	1	177.44	177.50	0.06	177.72	0.28
	2	180.14	177.83	2.31	178.04	2.10
	3	183.98	183.98	0.00	183.98	0.00
HC-2	1	167.99	167.99	0.00	167.99	0.00
HC-4	1	122.96	106.87	16.09	122.15	0.81
	2	133.18	135.10	1.92	135.25	2.07
	3	133.60	135.31	1.71	135.38	1.78
	4	135.44	135.44	0.00	135.44	0.00
HC-5	1	114.97	105.77	9.20	116.02	1.05
	2	118.97	105.77	13.20	116.03	2.94
	3	124.70	124.86	0.16	125.35	0.65
	4	125.83	125.83	0.00	125.83	0.00
HC-8	1	109.46	105.47	3.99	114.38	4.92
	2	115.59	108.39	7.20	115.20	0.39
	3	117.87	111.31	6.56	116.01	1.86
	4	117.84	117.84	0.00	117.84	0.00
HC-14	1	108.31	104.08	4.23	107.26	1.05
	2	108.32	104.07	4.25	107.23	1.09
	3	108.63	104.07	4.56	107.22	1.41
	4	109.14	109.14	0.00	109.14	0.00
HC-11	1	92.09	89.15	2.94	89.14	2.95
	2	89.48	89.48	0.00	89.48	0.00
HC-16	1	74.49	75.61	1.12	75.61	1.12
	2	75.39	75.80	0.41	75.80	0.41
	3	76.21	76.21	0.00	76.21	0.00
Sum of Differences:				----- 79.91	----- 26.88	

FLONET SENSITIVITY ANALYSIS

		2	1		1	
		Measured	Case 9	Difference	Case 10	Difference
HC-1	1	177.44	177.44	0.00	177.29	0.15
	2	180.14	177.78	2.36	177.63	2.51
	3	183.98	183.98	0.00	183.98	0.00
HC-2	1	167.99	167.99	0.00	167.99	0.00
HC-4	1	122.96	103.34	19.62	118.64	4.32
	2	133.18	135.06	1.88	133.87	0.69
	3	133.60	135.29	1.69	134.69	1.09
	4	135.44	135.44	0.00	135.44	0.00
HC-5	1	114.97	103.22	11.75	114.89	0.08
	2	118.97	103.22	15.75	114.90	4.07
	3	124.70	124.74	0.04	122.21	2.49
	4	125.83	125.83	0.00	125.83	0.00
HC-8	1	109.46	103.18	6.28	113.85	4.39
	2	115.59	106.64	8.95	114.51	1.08
	3	117.87	110.10	7.77	115.17	2.70
	4	117.84	117.84	0.00	117.84	0.00
HC-14	1	108.31	103.03	5.28	109.19	0.88
	2	108.32	103.03	5.29	109.17	0.85
	3	108.63	103.03	5.60	109.16	0.53
	4	109.14	109.14	0.00	109.14	0.00
HC-11	1	92.09	89.15	2.94	89.18	2.91
	2	89.48	89.48	0.00	89.48	0.00
HC-16	1	74.49	75.61	1.12	75.61	1.12
	2	75.39	75.80	0.41	75.80	0.41
	3	76.21	76.21	0.00	76.21	0.00
Sum of Differences:				----- 96.73	----- 30.27	

FLONET SENSITIVITY ANALYSIS

		2	1	
		Measured	Case 11	Difference
HC-1	1	177.44	178.18	0.74
	2	180.14	178.47	1.67
	3	183.98	183.98	0.00
HC-2	1	167.99	167.99	0.00
HC-4	1	122.96	102.87	20.09
	2	133.18	135.35	2.17
	3	133.60	135.43	1.83
	4	135.44	135.44	0.00
HC-5	1	114.97	102.62	12.35
	2	118.97	102.62	16.35
	3	124.70	125.70	1.00
	4	125.83	125.83	0.00
HC-8	1	109.46	102.56	6.90
	2	115.59	102.55	13.04
	3	117.87	110.10	7.77
	4	117.84	117.84	0.00
HC-14	1	108.31	102.27	6.04
	2	108.32	102.27	6.05
	3	108.63	102.27	6.36
	4	109.14	109.14	0.00
HC-11	1	92.09	89.14	2.95
	2	89.48	89.48	0.00
HC-16	1	74.49	75.61	1.12
	2	75.39	75.80	0.41
	3	76.21	76.21	0.00
Sum of Differences:				----- 106.84

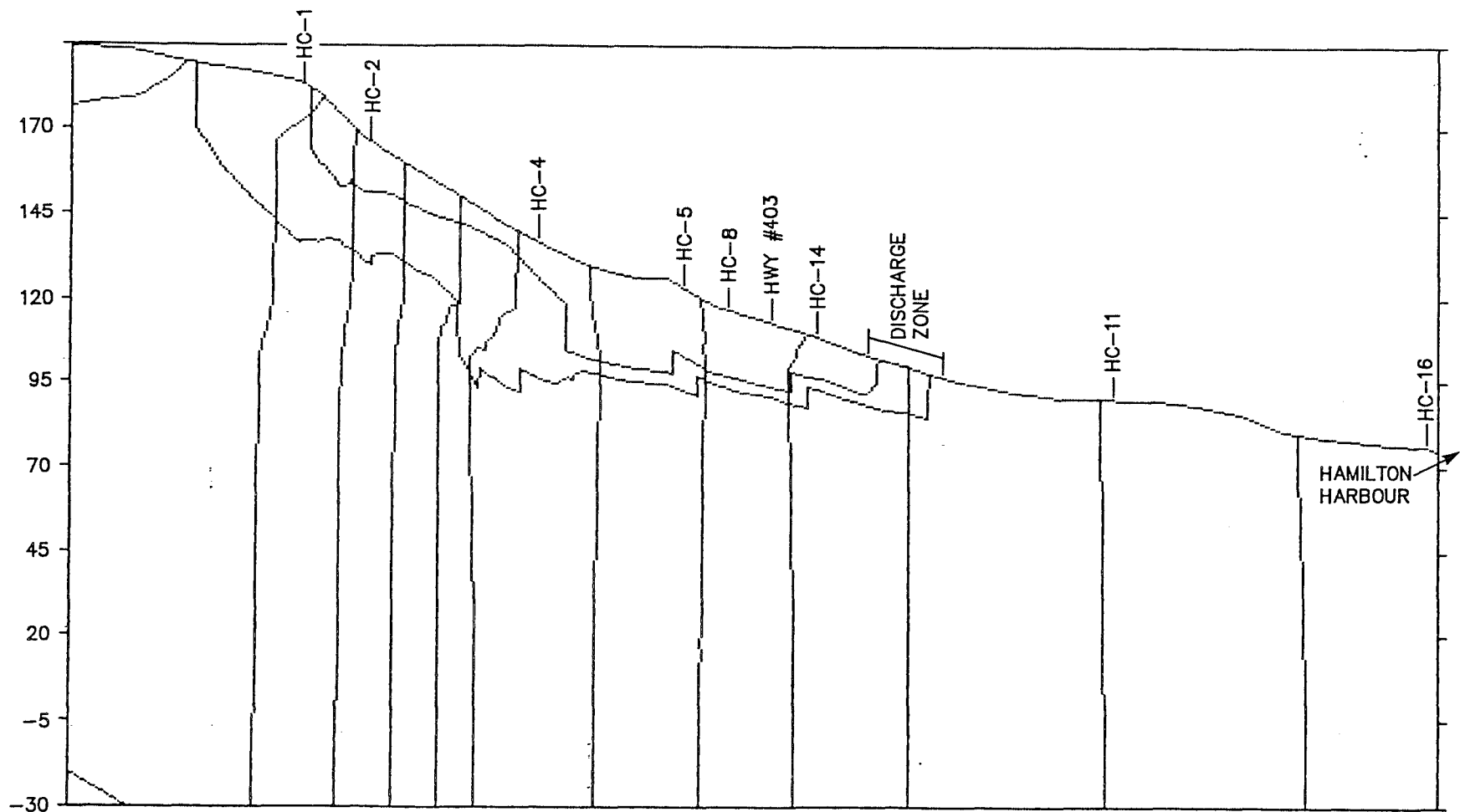


HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

Appendix II - 1
 SENSITIVITY ANALYSIS CASE 1
 Site 'F' - Hydrogeology - City of Burlington

CRA

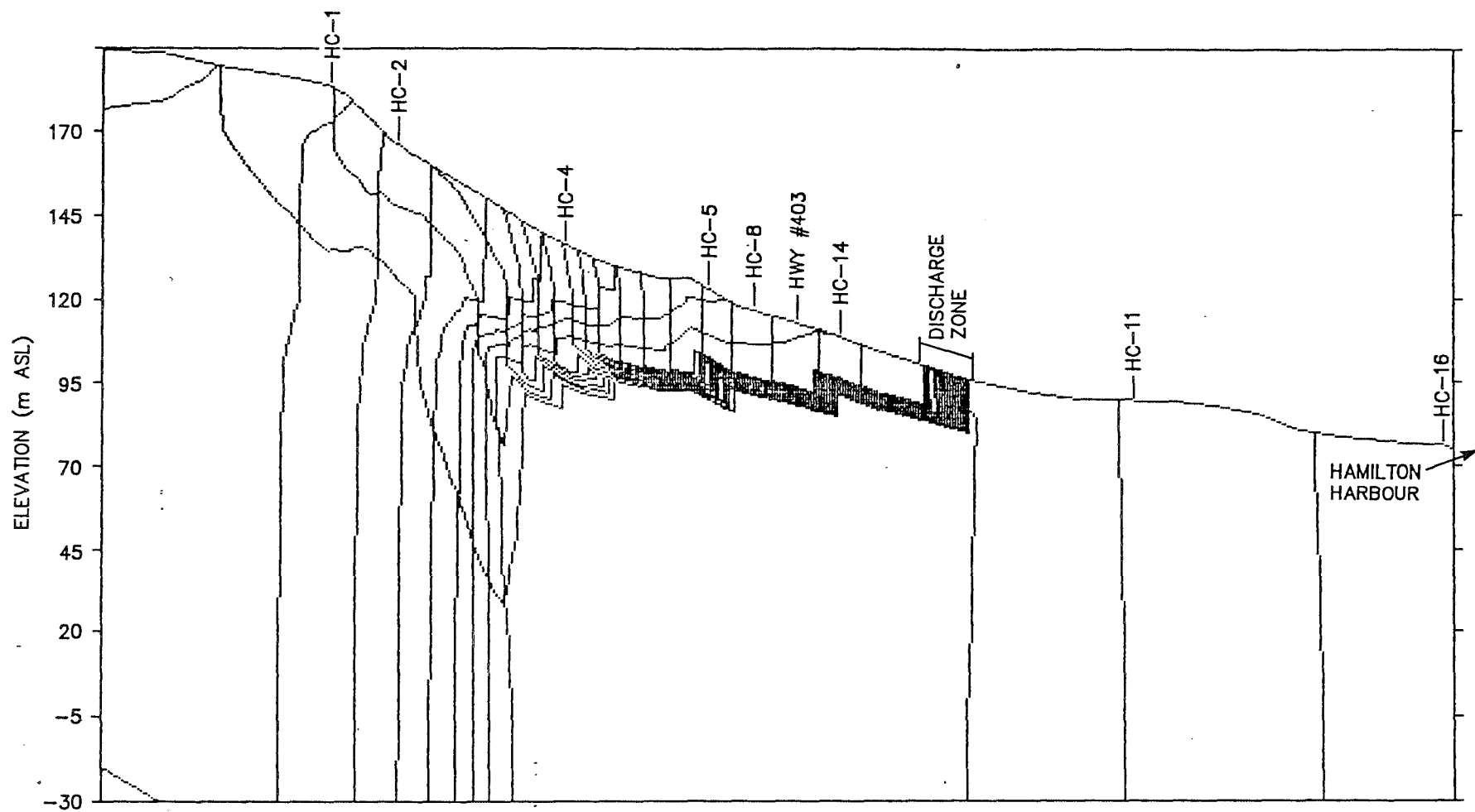
1354-10-8/01/88-34-D-0



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

CRA

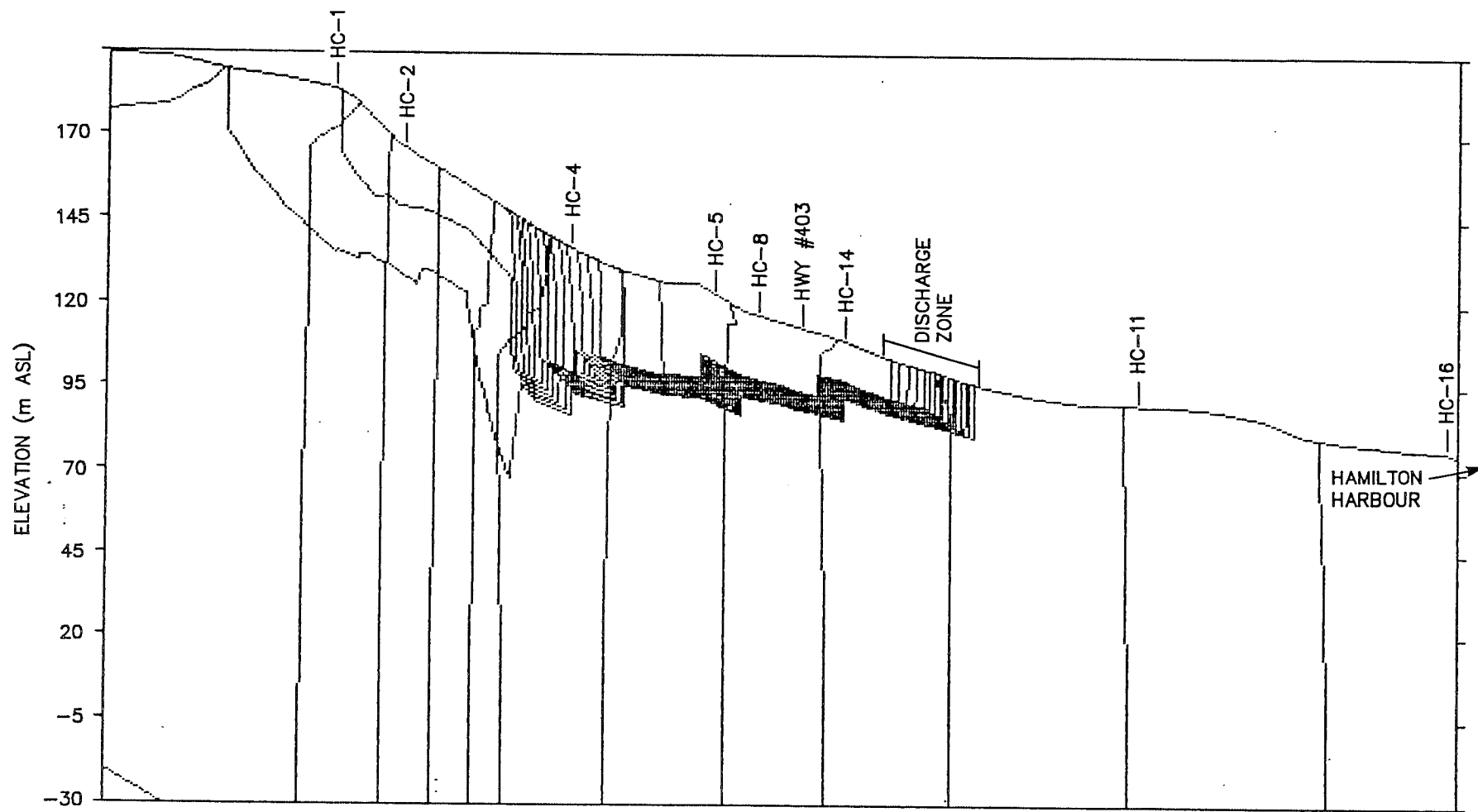
Appendix II - 2
 SENSITIVITY ANALYSIS CASE 2
 Site 'F'—Hydrogeology — City of Burlington



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

CRA

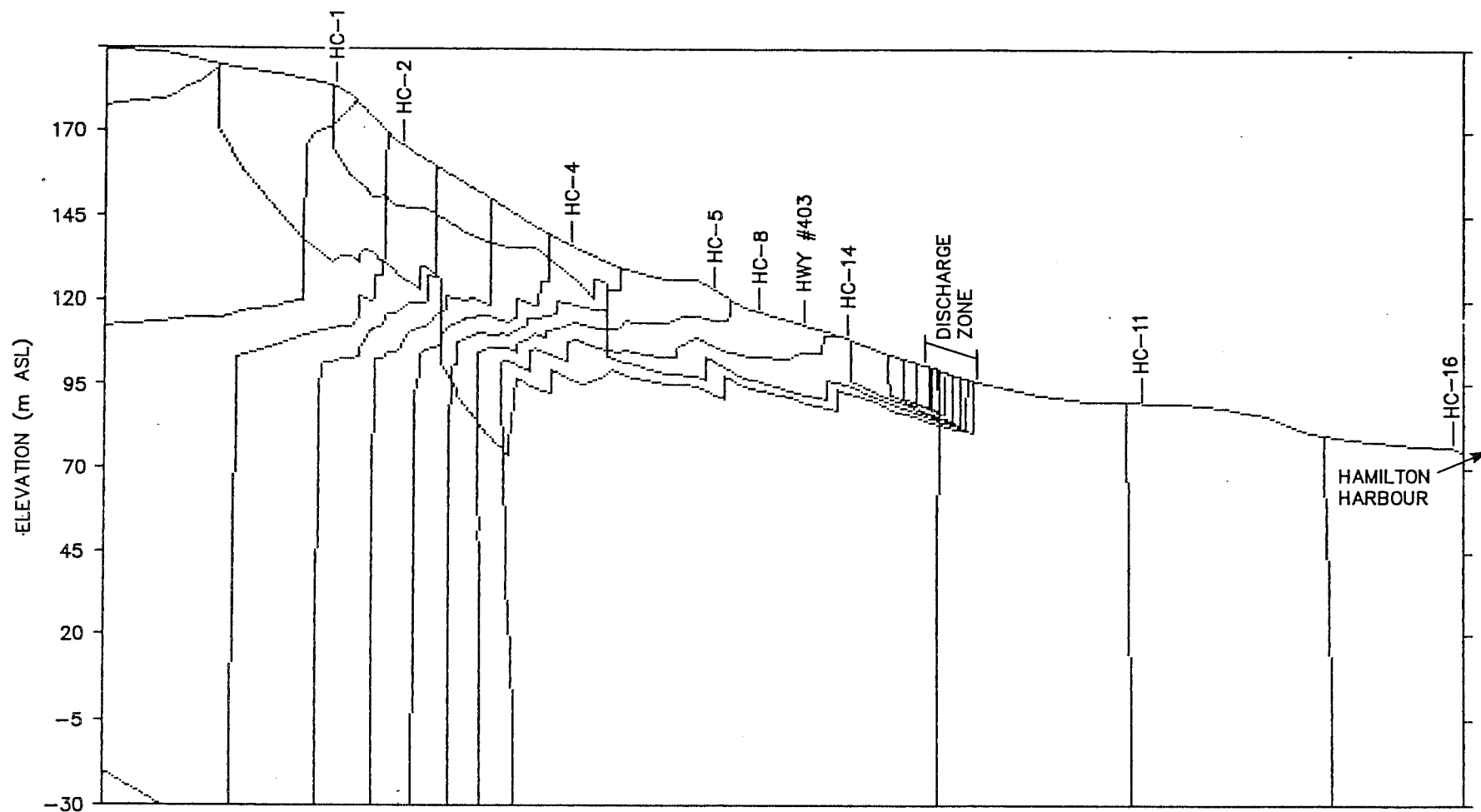
Appendix II - 3
 SENSITIVITY ANALYSIS CASE 3
 Site 'F' - Hydrogeology - City of Burlington



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

Appendix II - 4
 SENSITIVITY ANALYSIS CASE 4
 Site 'F'—Hydrogeology — City of Burlington

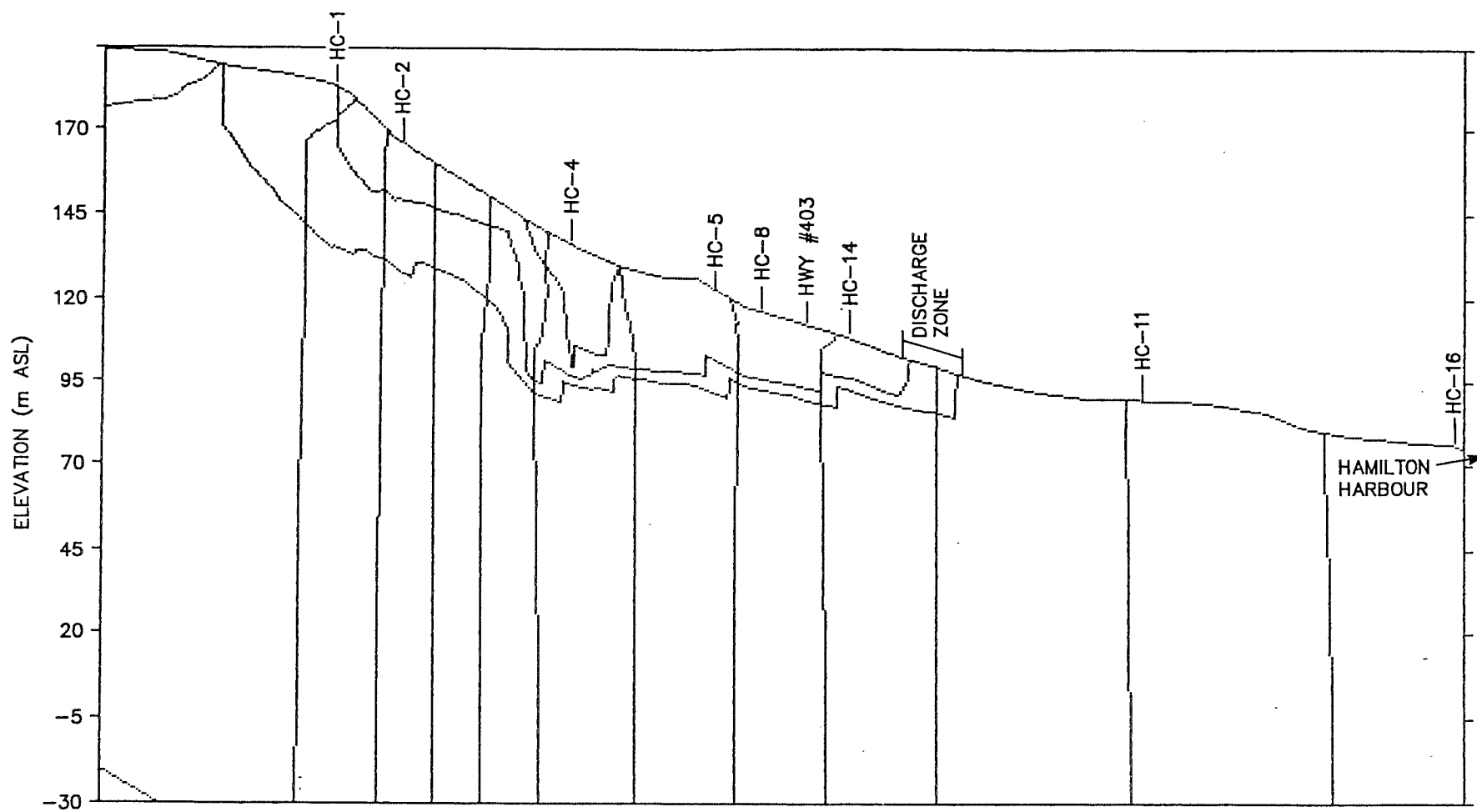
CRA



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

Appendix II - 5
 SENSITIVITY ANALYSIS CASE 5
 Site 'F'—Hydrogeology — City of Burlington

CRA



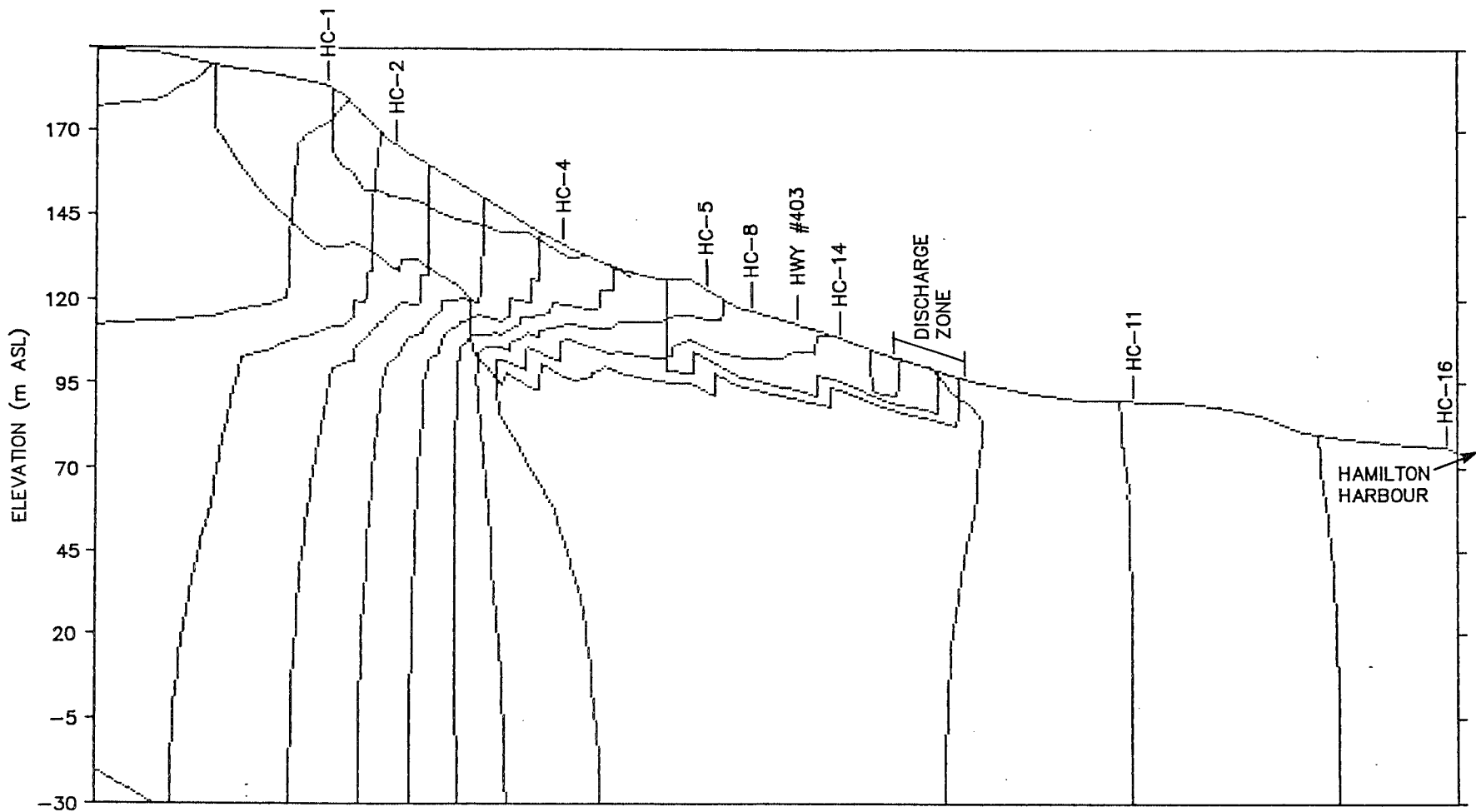
HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: 30m³/yr

Appendix II - 6

SENSITIVITY ANALYSIS CASE 6
 Site 'F'—Hydrogeology — City of Burlington

CRA

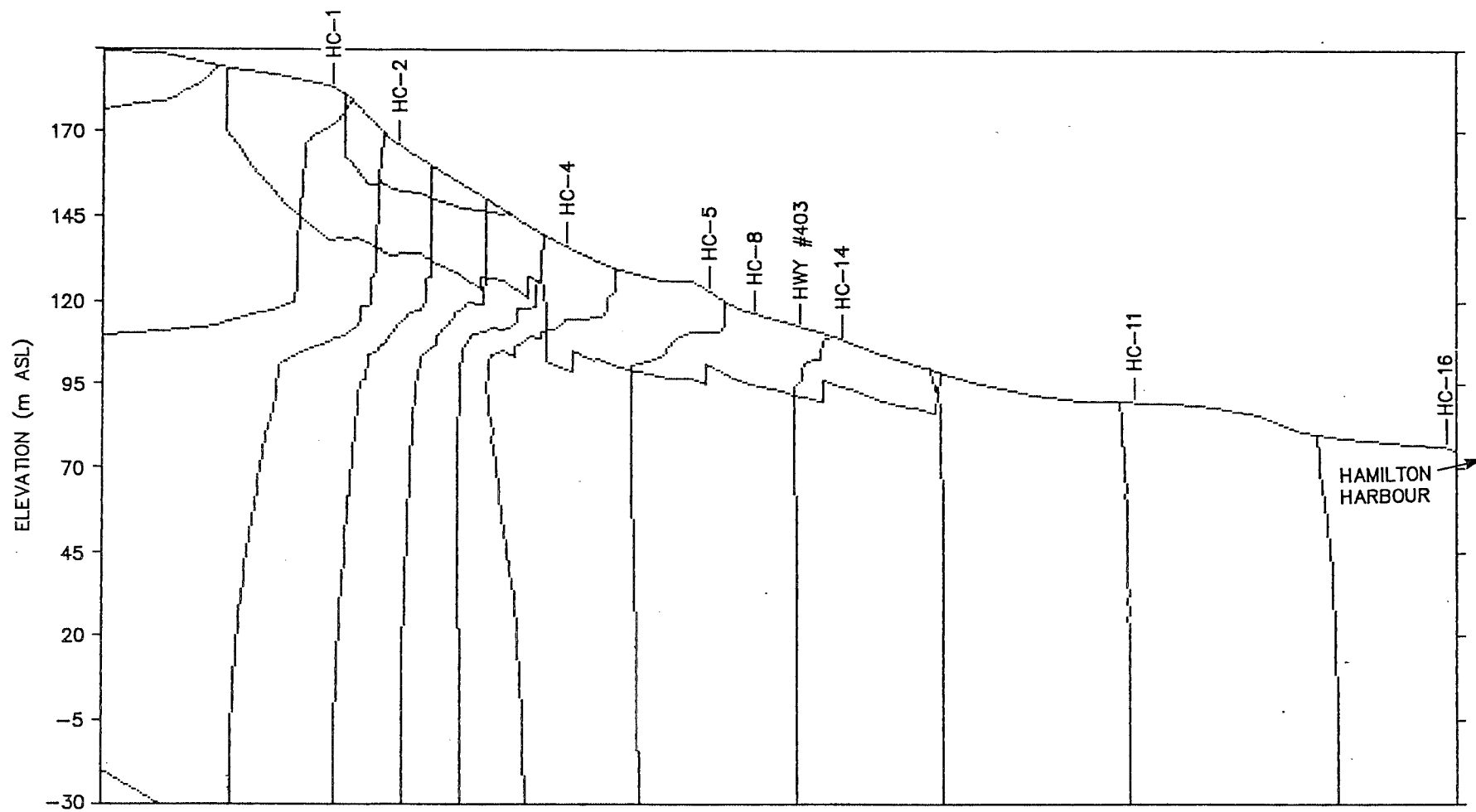
1354-10-8/01/88-34-D-0



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

CRA

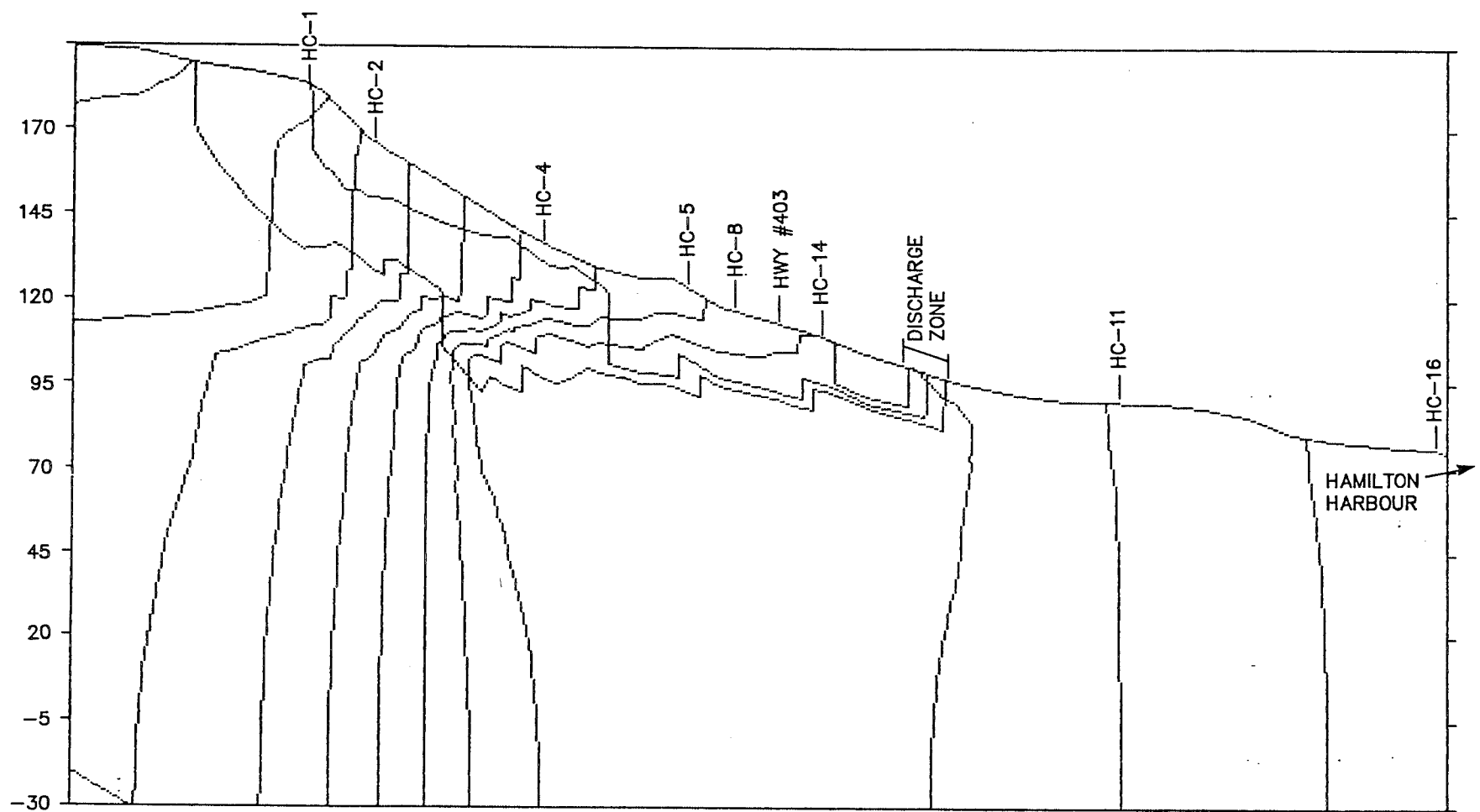
Appendix II - 7
 SENSITIVITY ANALYSIS CASE 7
 Site 'F'—Hydrogeology — City of Burlington



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

CRA

Appendix II - 8
 SENSITIVITY ANALYSIS CASE 8
Site 'F'—Hydrogeology — City of Burlington



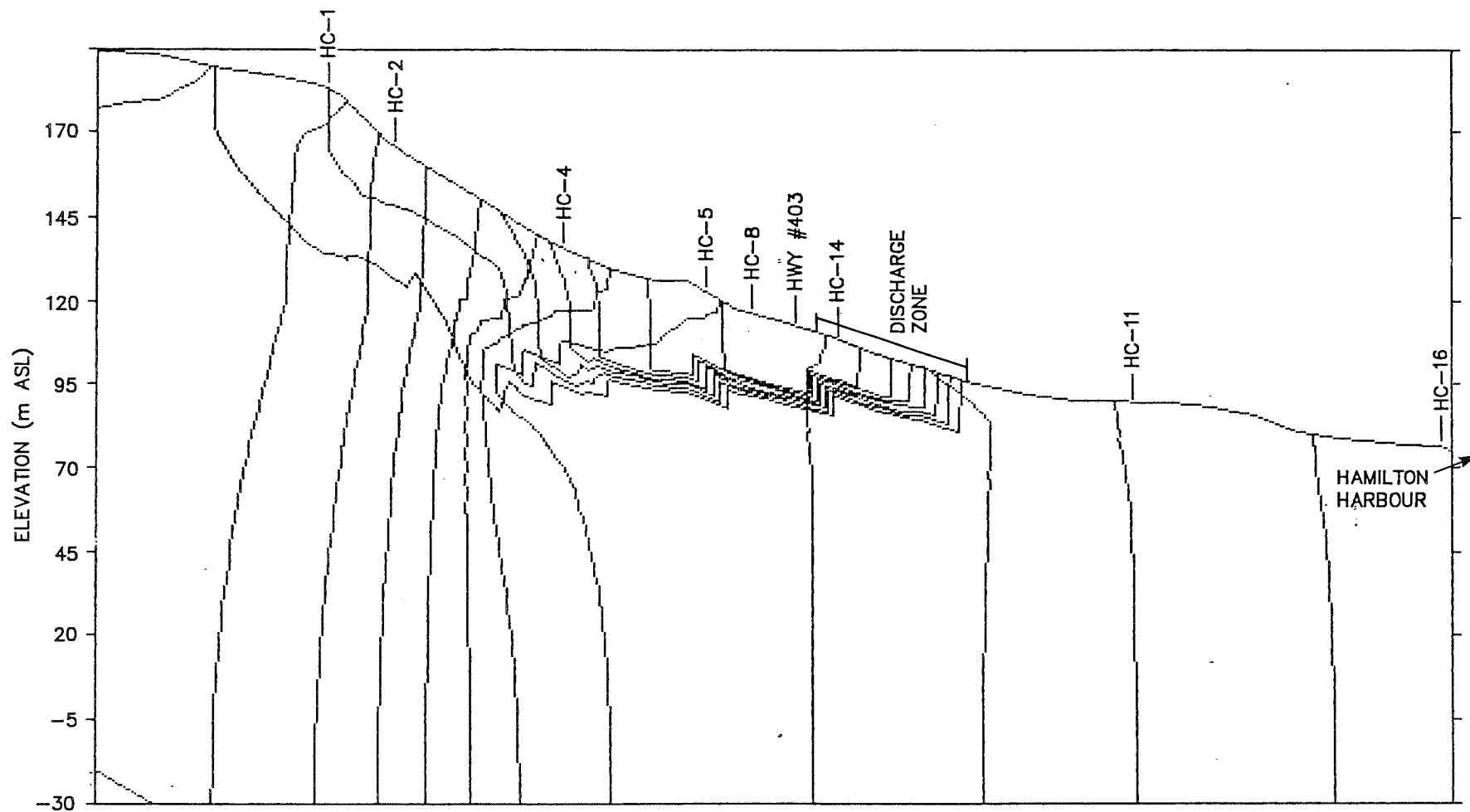
HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

Appendix II-9

SENSITIVITY ANALYSIS CASE 9
Site 'F'—Hydrogeology — City of Burlington

CRA

1354-10-8/01/88-34-D-0

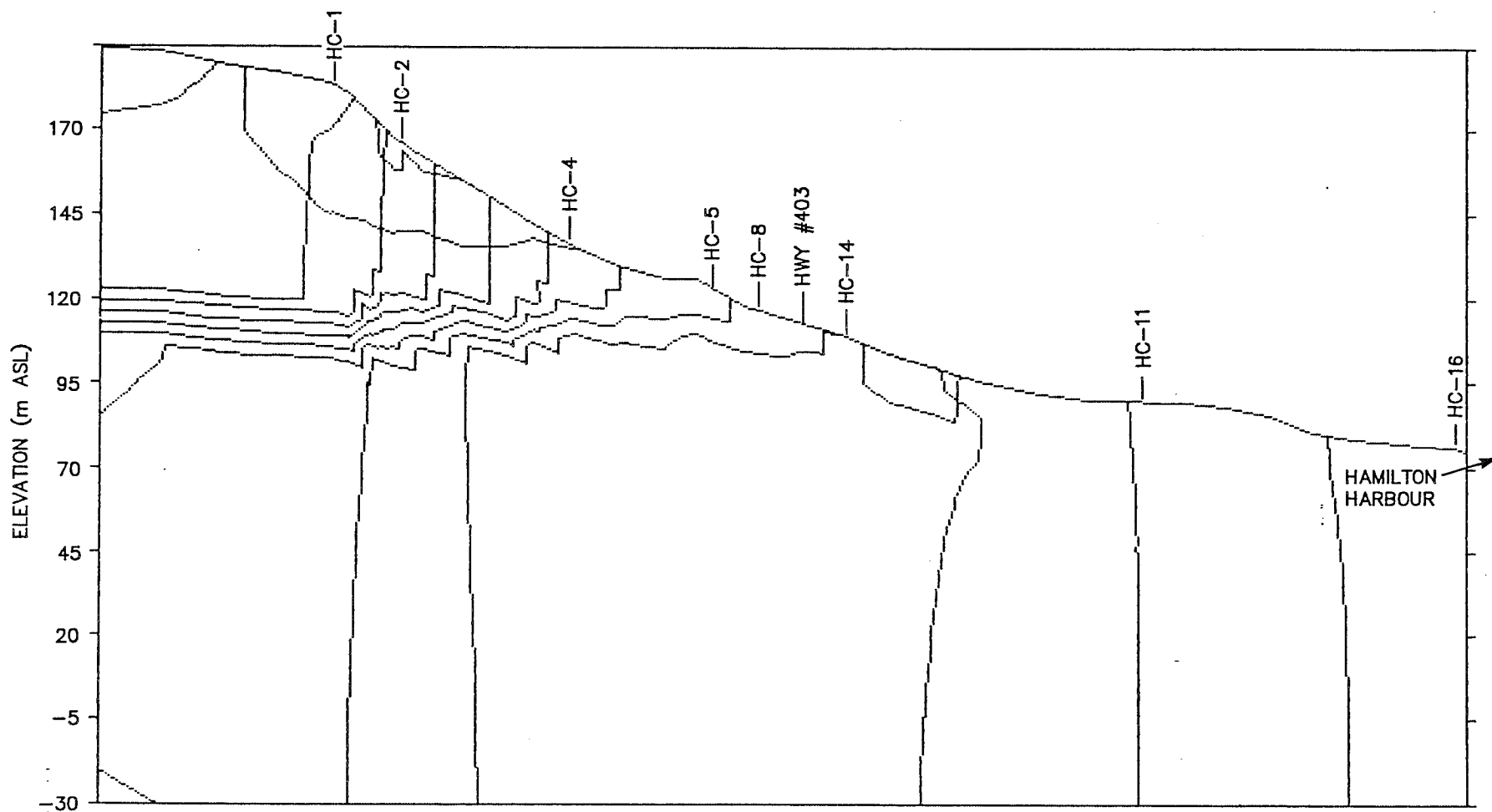


HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: $30\text{m}^3/\text{yr}$

Appendix II-10
 SENSITIVITY ANALYSIS CASE 10
Site 'F'—Hydrogeology — City of Burlington

CRA

1354-10-8/01/88-34-D-0



HEAD CONTOUR INTERVAL: 10m
 STREAM FUNCTION CONTOUR INTERVAL: 30m³/yr

Appendix II - 11

SENSITIVITY ANALYSIS CASE 11
Site 'F' - Hydrogeology - City of Burlington

CRA

1354-10-8/01/88-34-D-0

APPENDIX III

EVALUATION OF CONTAMINANT VELOCITY IN
LOW PERMEABILITY FRACTURED SHALE

EVALUATION OF CONTAMINANT VELOCITY IN
LOW-PERMEABILITY FRACTURED SHALE

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INTRODUCTION

The prediction of subsurface migration of toxic contaminants from landfills disposal or industrial sites underlain by fractured shale has become an important consideration in the assessment of water quality in the heavily-industrialized regions of southern Ontario. For example, the near-surface sediments of the Oakville-Mississauga region are underlain by the fractured Dundas shale, the Burlington-Hamilton region has the Queenston and Rochester shales, and the Niagara Falls region has various fractured shales including the Queenston shale that may transmit contaminants through the subsurface to the Niagara River. Because fractured shale with low matrix permeability underlies much of these industrialized regions, it is probable that fractures will play an important, if not dominant role, in providing pathways for subsurface migration of toxic contaminants. This leads to the expectation that very large contaminant plumes can develop within fractured shale under conditions of relatively low contaminant mass flux into the shale. Consequently, landfills and industrial sites situated on fractured shale can produce large plumes of contaminated groundwater that extend over considerable distances. Risk assessments for the design, siting, or clean up of such sites require that hydrogeologic models be used to estimate the extent and rates at which contaminants may migrate from the site to the accessible biosphere. The most important step in predicting the extent to which groundwater contamination could occur is the determination of the average velocity of the non-reactive contaminants. In groundwater investigations, however, the greatest difficulties and uncertainties in measuring and modelling groundwater velocities occur in fractured rock.

There are two modelling approaches to the simulation of fluid flow and contaminant transport in fractured media. The continuum approach treats a fractured rock mass as if it were equivalent to a porous medium, while network models are based upon an explicit representation of the fractures. If fracture densities are much greater than those that mark the onset of percolation, then a continuum approach can be adopted. In this case, parameters must be formulated to characterize advective, dispersive, and chemical processes occurring within representative elemental volumes (REV) of the fractured shale mass. At sites underlain by sparsely fractured shale, or those where a significant proportion of the fractures are sealed by mineral precipitates, considerable transport distance may be required before contaminants encounter a representative sample of fluid velocities. Until contaminants have migrated over that distance, continuum models are not applicable. It is also conceivable that there may be several scales of fracturing within the flow region, with a relatively few, areally extensive fractures exerting a disproportionately large influence on the groundwater flow velocities. These situations can be identified as sub-REV problems where discrete network models are appropriate. A characteristic of such sites is the potential for contaminant transport to be significantly influenced by individual fractures and their interconnections with other fractures forming the network.

Migration of contaminants in a network of discrete fractures that cannot be represented using a continuum approach, including the region close to the site where contaminants enter the flow system, can be simulated using stochastic modelling techniques. The starting point for a stochastic model is a statistical description of fracture geometry. Probability descriptions of the fracture locations, dimensions, and apertures are required to transform a geometrical model of the fracture network into a hydraulic model. Although stochastic fracture models are powerful analytical tools, if they are to find practical applications, then it is necessary to demonstrate that reliable estimates of model parameters can be obtained within the normal constraints of field investigations. One key parameter is fracture aperture. Mechanical measurements of aperture in the disturbed region around a borehole are of questionable reliability. Current practice relies on hydraulic and, or, tracer tests to obtain in situ estimates of fracture aperture. From measurements of aperture and hydraulic head, it is then possible to calculate the groundwater velocity in the fracture. The objective of this study is threefold. First, to examine numerically the reliability of several hydraulic and induced-gradient tracer tests used to obtain in situ estimates of fracture aperture. Second, to apply these tests in the determination of the aperture of a fracture zone within low-permeability shale at a specific field site. Third, in light of the errors inherent in the interpretation of hydraulic and tracer tests, to assess whether the groundwater velocity determined from the estimated aperture and measurements of hydraulic head correctly represents the in situ velocity in the fracture. In this paper we discuss results from the first objective, numerical simulation of hydraulic and tracer tests. Preliminary results for the in situ determination of aperture and groundwater velocity for a specific field site are presented in the later part of the paper.

ESTIMATION OF FRACTURE APERTURE

Estimates of fracture aperture can be derived from both hydraulic and tracer tests. While a variety of testing configurations are possible, the examples presented here focus on steady state withdrawal tests. In this approach, a section of borehole adjacent to the fracture identified for testing is isolated and water is withdrawn from the fracture by pumping at a constant rate Q . Flow patterns and the groundwater velocity field within the fracture will depend upon how fluid moves in from the far-field to meet the pumping demand. Once steady state conditions are established, the fracture aperture can be estimated from difference in hydraulic head between an observation well and the withdrawal well. If a tracer is released from an observation well, then the aperture can also be estimated from its travel time to the withdrawal well. Examples of a withdrawal tests are given by Hodgkinson and Lever (1983), Abelin et al. (1985), Novakowski et al. (1985), and Anderson and Thunvik (1986).

To estimate fracture aperture, a model must be adopted to describe the flow field within the fracture plane. Models based on an assumption of axisymmetric flow are commonly used. Three such steady-state models are examined here. All three approaches are deterministic and have been adapted from methods originally used for interpreting hydraulic and tracer tests carried out in confined aquifers.

Assuming axisymmetric flow toward the withdrawal well, the volume flux across a surface a distance r from the withdrawal well is related to the pumping rate by

$$(1) \quad Q = 2\pi r K_f b \frac{dh}{dr}$$

K_f is the equivalent hydraulic conductivity of the fracture plane and b is the fracture aperture. Adopting Poiseuille's equation for laminar flow in a parallel plate opening of aperture b , and given values of hydraulic head at radial distances r_o and r_w , the fracture aperture is given by

$$(2) \quad b_h = \left[\frac{Q}{(h_o - h_w)} \frac{6\mu}{\pi \rho g} \ln(r_o/r_w) \right]^{1/3}$$

where b_h is referred to as the 'hydraulic aperture', r_w is the well radius and $(h_o - h_w)$ is the measured difference in hydraulic head between the observation and withdrawal wells.

A second model to estimate fracture aperture is based on mass balance considerations. For a tracer to move by advection from r_o to r_w , the fluid volume contained between the two radii r_w and r_o must be removed by the pumping well. If t_w denotes the travel time of a tracer from r_o to r_w , then the volume of water pumped in time t_w is given by

$$(3) \quad Qt_w = \pi b (r_o^2 - r_w^2)$$

The fracture aperture is estimated from (2) by

$$(4) \quad b_v = \frac{Qt_w}{\pi(r_o^2 - r_w^2)}$$

where b_v is referred to as the 'volume balance' aperture. In making this calculation, the area from which fluid is withdrawn is given implicitly by the radial distance from the injection point (r_o) to the outer radius of the withdrawal well (r_w).

A third approach for estimating fracture aperture also uses results from tracer tests. The travel time from an injection point to the withdrawal well is given by

$$(5) \quad t_w = \int_{r_o}^{r_w} \frac{dl}{V_1}$$

where the integration is along the streamline connecting the injection point with the withdrawal well and V_1 is the groundwater velocity along the streamline. In general, both the path length and the variation in the advective velocity along the streamline are unknown. By expressing the travel time as a function of an assumed axisymmetric flow in a parallel-plate fracture, both the path length and the radial velocity are known to within a constant that is given by the fracture aperture. The aperture is calculated by

$$(6) \quad b_e = \left(\frac{6H}{pg} \frac{\ln(r_o/r_w)}{t_w(h_o - h_w)} (r_o^2 - r_w^2) \right)^{1/2}$$

where b_e is referred to as the 'tracer aperture'.

All three models are appropriate for the idealized case of laminar flow in a smooth, planar fracture of large areal extent. It is important to recognize that these simple axisymmetric models are adopted in practice because data are seldom, if ever, available to refine the specification of boundaries on the fracture plane and on source or sink positions. However, the potential exists for deviations from these simple models because of distortion of the flow field by the roughness of the fracture walls, fractures with finite dimensions, and fracture intersections that act as line sources or sinks of fluid. In the following sections, we describe a numerical experiment that examines the magnitude of potential errors in the interpretation of hydraulic and tracer tests to obtain estimates of fracture aperture. The numerical model used here is described by Smith et al. (1985). This model simulates fluid flow and contaminant transport in three-dimensional networks of discontinuous, rough-walled fractures. Variations in aperture within each planar fracture could depend upon a number of factors including the roughness of fracture walls, the distribution of fracture-filling material, and fracture closure due to an applied stress. Differences between the in situ numerical aperture, and the estimated aperture based on hydraulic and travel time data are highlighted. In the following example, it is assumed that two boreholes penetrate a fracture which is identified for field

testing. The numerical experiment focuses on the factors that distort the radial flow field. To focus on these factors, matrix diffusion is neglected and the validity of the cubic law is not examined. We view these as important but separate issues.

Example of an Injection-Withdrawal Test

Figure 1 illustrates a fracture network positioned within an impermeable rock block. This section represents a vertical slice through the center of the rock mass. The network consists of three horizontal fractures, connected by ten cross fractures that intersect the horizontal fractures at 60° . Tests are simulated assuming the injection and withdrawal boreholes are open only to fracture plane two. The bounding edges of this fracture are impermeable. The withdrawal well is located at the center of fracture plane with a withdrawal rate of 0.8 l/min. Fracture planes one and three extend to external boundaries and are assigned equal values of hydraulic head on the X face. Water moves in from these constant head boundaries to meet the pumping demand of the withdrawal well. The Y face of the block is impermeable. Although highly idealized, this network contains features characteristic of the selected field site that can be expected to perturb an axisymmetric flow field. Solutions for the hydraulic head distribution and contaminant travel time through this three-dimensional network are obtained using a model described by Smith et al. (1985, 1986). In general, other examples show that the volume balance aperture is not a reliable estimate because of its sensitivity to errors in the assumed geometry of the region where water is withdrawn. Consequently, we will not consider it for this example.

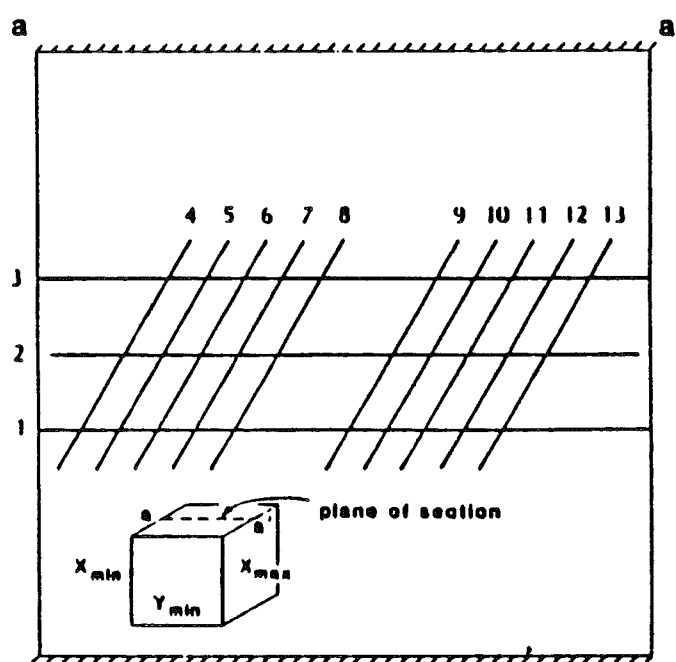


Figure 1. Discrete fracture network used in numerical simulations. Domain is 16 x 16 x 16 M.

Field evidence suggests that even for "open" fractures, there is considerable variation in the aperture of a planar fractures, due to combined effects of undulating fracture surfaces, mineral precipitates, and closure under in situ stresses. This aperture variation leads naturally to concepts of "channelling" within a fracture network. To model rough-walled, open fractures, aperture values can be represented as a spatial stochastic process. Results for a single realization are described here. For the horizontal fractures and cross fracture twelve, a lognormal distribution is specified with a log mean of -3.30 and a log standard deviation of 0.25 (base ten logarithms for aperture in meters). An isotropic correlation length of 2 m is assumed. Aperture values for the remaining cross fractures are generated from $\text{LOG}[-5.50, 0.25]$. Thus fracture twelve forms the dominant link between fracture plane two and the other fractures within the network. Values of the standard deviation and correlation length are selected here only to illustrate concepts, and do not represent a particular geologic environment. Figure 2a is a contour plot of the aperture variation within fracture plane two. Note that there is a crescent-shaped channel of higher aperture located within the central region of the fracture plane.

Figure 2b is a plot of the steady state hydraulic head field within fracture plane two. The values shown have been normalized, with the hydraulic head at the withdrawal well assigned a value of zero, and the maximum hydraulic head within the fracture plane assigned a value of one. The plot demonstrates that the general pattern of groundwater flow is determined by the geometry of the fracture network and the connections to the constant head boundaries, with the aperture variation acting to perturb the magnitude of the local hydraulic gradient. Due to the influence of the geometry of the network and the roughness of the fracture walls the flow field is decidedly non-symmetric.

A plot of hydraulic aperture is shown in Figure 2c. In this figure, the aperture values at a specific point on the fracture plane represent the aperture which would be calculated from a hydraulic tracer test if the observation well were located at that point. The plot is constructed by specifying a regularly-spaced grid of well locations, noting the hydraulic head at each location, and using a tracking particle to trace the path and travel time from each observation well point to the withdrawal well. From this data, both the hydraulic and tracer apertures can be calculated. Two features can be noted. First, the hydraulic aperture is insensitive to the spatial variability in aperture within the fracture plane. Second, the magnitude of the hydraulic aperture is primarily influenced by aperture values in the close vicinity of the withdrawal well. This happens because it is also the region where the the greatest drop in hydraulic head occurs. To illustrate this effect, Figure 2e is a plot of the hydraulic aperture calculated for the same fracture. The withdrawal well, however, is now located in a region where the fracture aperture is approximately 200 microns. For this case, the hydraulic head is much lower, again reflecting the real value close to the withdrawal well. Other examples, not shown here, suggest the hydraulic aperture is only mildly sensitive to the distortion of a radial flow field due to network effects, fracture intersections.

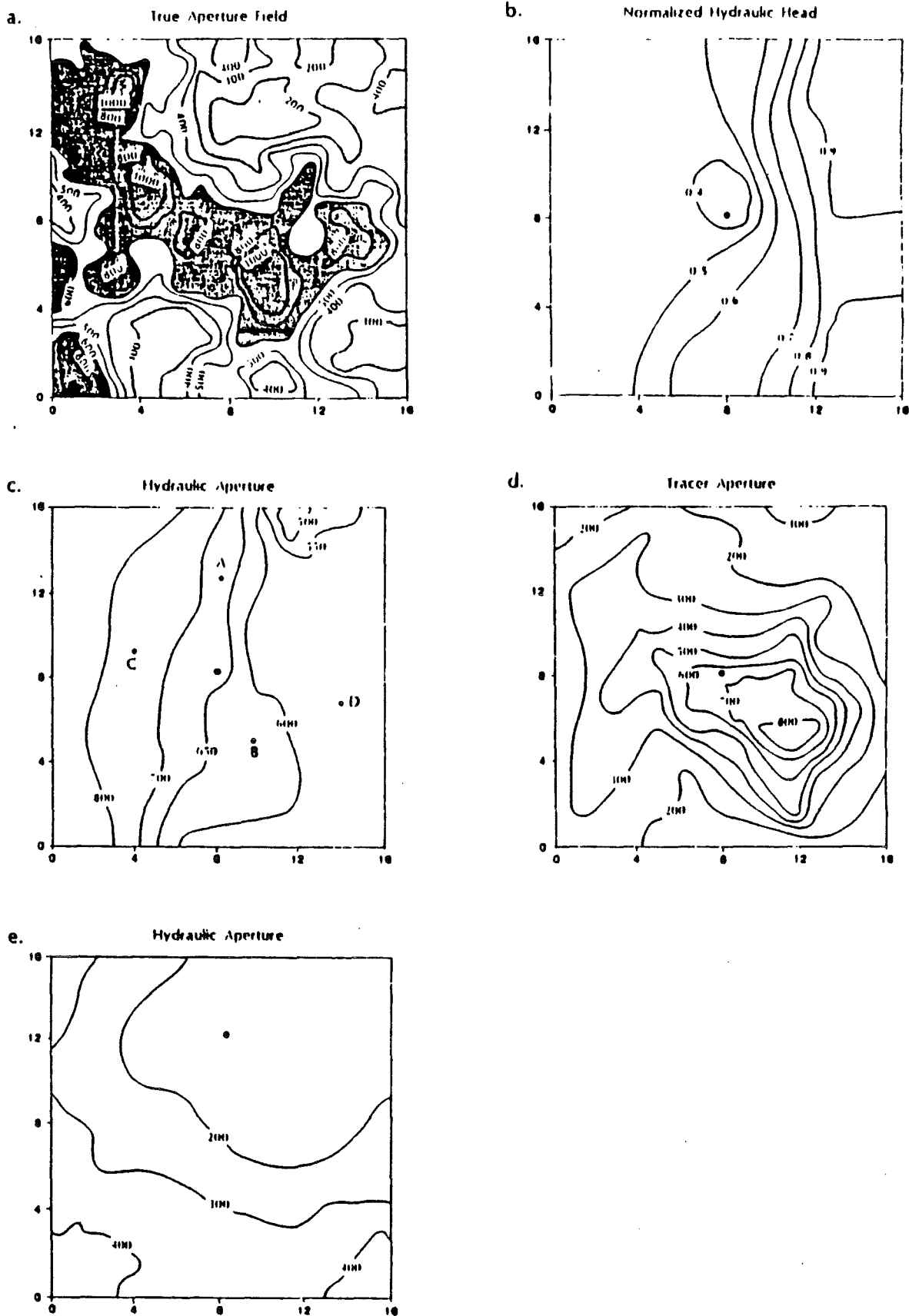


Figure 2. Results of aperture calculations for fracture plane two, network of rough-walled fractures. Aperture values are in microns.

Values of the tracer aperture for the same fracture are shown in Figure 2d. The tracer aperture is sensitive to the location of fracture intersections that provide water to meet the pumping demands, and to the relative position of the lower and higher aperture zones within the fracture plane. Calculated apertures are a smooth version of real values. The higher aperture region between the withdrawal well and the fracture intersection is identified, although estimated values are significantly less than peak values. The extension of this 'channel' into the region behind the fracture intersection or on the opposite side of the withdrawal well is not identified because of the limited groundwater flow which occurs there.

Figures 2c and 2d can be compared to identify relationships between aperture estimates based on hydraulic and tracer data. Consider the position of observation wells at points labelled A, B, C, and D. Point A is located within a region of the fracture plane with a lower aperture than the mean value. Here the estimate of hydraulic aperture exceeds the tracer aperture by a factor of three. The hydraulic head does not record the lower aperture zone because this model primarily reflects conditions only in the immediate vicinity of the withdrawal well. In general, the hydraulic aperture will exceed the tracer aperture if the withdrawal well is located in a more permeable zone of the fracture plane than the observation well. A tracer released at point A traverses the low aperture zone and thereby records a longer travel time, leading to the lower value of tracer aperture. At point B, estimates of the tracer aperture would exceed the hydraulic aperture by approximately 200 microns because the observation well is located in a zone with a greater aperture than the withdrawal well, and its position in the main 'channel' that supplies groundwater to the withdrawal well. At points C and D, the hydraulic aperture exceeds the value estimated from a tracer test. At point C, the tracer aperture is less than one third the real value because most of the water withdrawn for the well comes from the fracture intersection located on the other side of the well. Consequently, the slow 'back circulation' in this half of the fracture leads to large travel times and aperture estimates. For point D, the tracer aperture underestimates the real value because limited circulation within the dead-end segments of the fracture plane leads to large travel times and, consequently, lower estimated apertures. The hydraulic aperture, however, reflects the hydraulic head along the fracture intersection of the dead-end fracture and thereby does not feel the effects of large travel times from this region.

The example presented here demonstrates the potential for errors in estimates of fracture aperture using conventional testing and interpretational procedures. Large differences between hydraulic and tracer apertures are possible because of differing sensitivities to the effects of spatial variations in aperture within a fracture plane, and the presence of fracture intersections due to network geometry. In field operations, however, such information to aid in the interpretation of field data is seldom, if ever, available. If reliable estimates of fracture apertures are to be derived from the field results of hydraulic and tracer tests, it appears that an approach is needed to provide for additional constraints on the model adopted to describe the groundwater

flow system. Such techniques conceivably require multiple borehole responses, more sophisticated interpretational models than those based on simple axisymmetric flow, integration with geophysical techniques that sense the migration of a tracer within a fracture plane or network, or measurement of the in situ groundwater velocity by means of borehole dilution or natural-gradient tracer tests.

STUDY SITE

A site on Petro-Canada property in Clarkson, Ontario was selected to evaluate and compare several methods for determination of the groundwater flow velocity in fractured shale. The site was selected for three main reasons. First, it is located on the Dundas shale, a fractured low-permeability shale hydrologically similar to other shale deposits of southern Ontario. Second, background information on the hydrogeology was available indicating a shallow water table, thin overburden (2 m), and flat lying bedrock. Finally, the site has easy access and is a secure site with respect to long-term monitoring and limited potential for vandalism. The ultimate aim for the site is to predict the migration of an injected tracer in a single fracture, and compare the predicted concentrations with the actual measure values in the fracture.

A preliminary drilling program was conducted in May 1986, and five boreholes were drilled 15 m apart in two perpendicular rows of three as shown in Figure 3. The boreholes were continuously cored using an NV3 core barrel, and were drilled in the following manner. Hollow stem augers were advanced until refusal in the weathered bedrock at approximately 3 m. The borehole were then drilled by triconing to a depth of approximately 5.5 m, and then cored to 16.5 m below ground surface at each location. The core was logged in the field and photographed in 60 cm sections. Once the drilling was completed, casing was set to a depth of 5 m and the borehole was left open to allow for downhole testing.

The core consisted primarily of shale with interbedded dolomite layers approximately 5-20 cm thick. The fracture pattern is primarily horizontal (along bedding planes) in the shale and vertical in the dolomite. The degree of fracturing generally decreased with depth. Examination of the core and subsequent packer tests delineated several high-permeability zones. The location of these main fracture zones were correlated between each borehole to determine which fractures zones could be continuous across the site. Four main fracture zones were delineated at depths of 6.5-6.8, 7.2-7.6, 10.4-10.7, and 14.0-14.3 m below the ground surface. The fracture zones were isolated using a double packer assembly which was pneumatically inflated with nitrogen. Water was then injected into the isolated zones at a constant pressure and the injection rate monitored. Results of packer testing indicated that the zone of highest permeability is located at depths of 10.4-10.7 m. The other zones tested exhibited a lower apparent permeability, and exhibited a decrease in the injection rate as the test continued. This decrease suggest that these fractures zones are localized and not continuous in lateral extent across the field site. Packer tests in non-fractured shale above and below the most permeable zone suggest a hydraulic conductivity of less than 10^{-10} ms⁻¹.

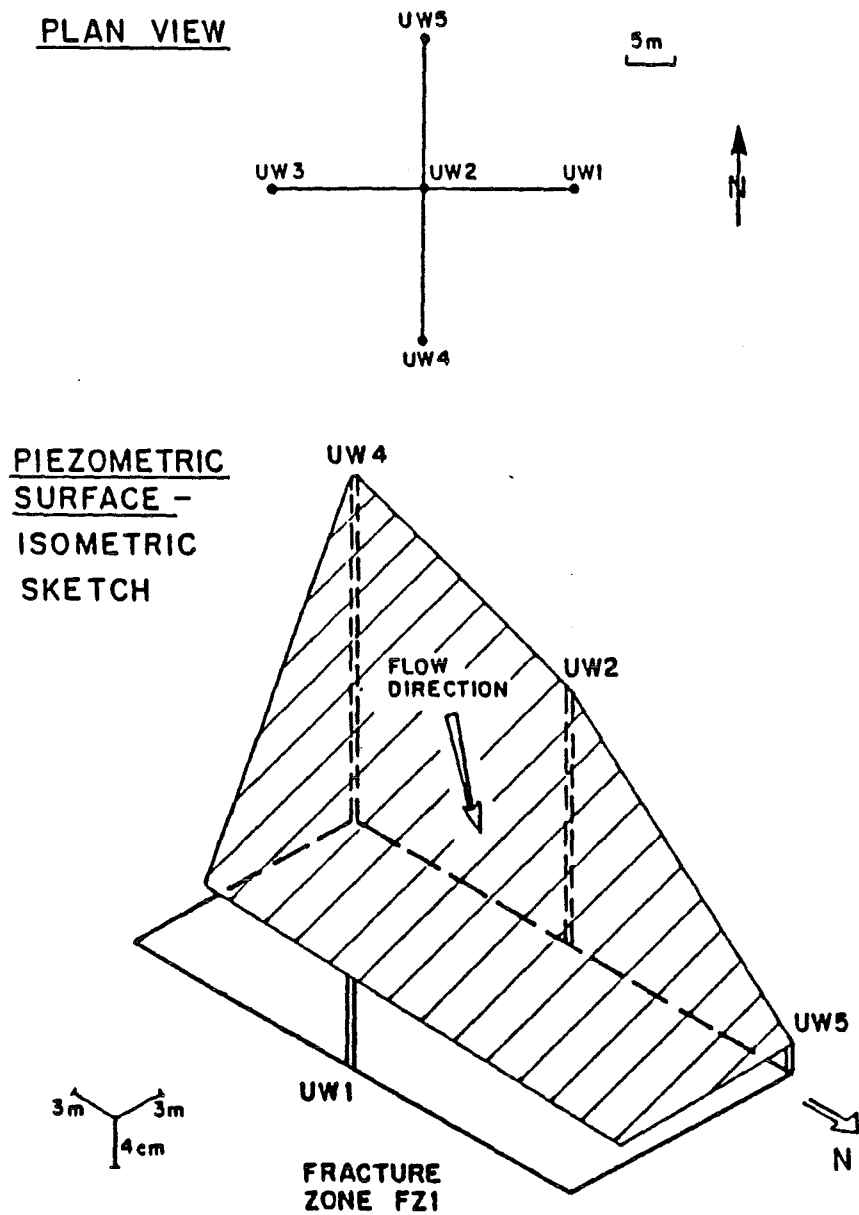


Figure 3. Plan view and hydraulic head surface of FZ1 at the Clarkson site.

Subsequent field work in the summer of 1987 focused on the most permeable zone (FZ1) located between 10.4-10.7 m depths. The objective of this work was to characterize the hydraulic head and fracture aperture within FZ1, and then use this data to predict groundwater velocities in FZ1. To measure the hydraulic head in FZ1, the zone was isolated in all boreholes by inflation of four triple packer piezometers. Problems were encountered in obtaining a stable set of measurements since deflation of a single packer would hydraulically connected FZ1 with other fracture zones. The packers were initially prone to slow leaks either through the rubber packer seals or poorly sealed valves and fittings. Due to these difficulties only one reliable set of measurements were obtained. Figure 3 shows a plan view of the site, together with an isometric diagram of the hydraulic head within FZ1 as viewed from the northeast. The hydraulic head in UW1, UW2, UW4, and UW5 ranged between 5.53 and 5.61 m below ground surface. Measurements in UW3, however, indicated a head of 3.23 m. Because of suspected packer problems, the measured hydraulic head at UW3 was not used for calculating the hydraulic gradient. The calculated hydraulic gradient based on the remaining wells is 0.01 and is towards the Northeast. The regional gradient is almost certainly to the Southeast towards Lake Ontario. Hydraulic head distributions in fractured rock, however, are usually complex. It is, therefore, not unusual to find groundwater flowing in a direction different from the regional gradient at such a shallow depth and small scale of observation (30 m).

A five hour pump test with a withdrawal rate of 460 ml min^{-1} from borehole UW5 produced drawdown in the remaining four observation wells. The drawdown data show a typical Theis behaviour with no apparent boundary effects and suggest that FZ1 is one continuous fracture zone with no lateral boundaries close to the site. In addition, none of upper packed off intervals above FZ1 registered any change in hydraulic head during the test which suggests that no vertical connections exists with other high-permeability zones. Fracture apertures calculated using equation (2) indicated an average hydraulic aperture of $160 \text{ }\mu\text{m}$ with a standard deviation of 38 ($\text{LOG}(2.19, .11)$). Similar aperture values are calculated from the hydraulic transmissivity obtained from a Theis analysis of the drawdown data. Such values, however, should be viewed with caution because well bore storage effects can cause the Theis method to underestimate the fracture transmissivity (Raven, 1986).

As seen from the previous numerical example (Figure 2), changing the position of the pumping well would yield substantially more information on the fracture aperture within FZ1. The average aperture of $160 \text{ }\mu\text{m}$ may reflect the aperture immediately surrounding well UW5, or it may reflect a relatively uniform aperture within FZ1. Later this fall we expect to move the position of the withdrawal well to each of the remaining four wells in turn and thereby obtain a set of four hydraulic-aperture measurements for each well. Variations in aperture within these sets should give us a clue to the variation in aperture within FZ1. In addition, preliminary induced-gradient tracer tests are expected to be carried out. From these tests, the volume balance and tracer apertures (equation (4) and (6), respectively) can be estimated. Comparison of all three methods will indicate whether FZ1 has a relatively uniform aperture, or a high degree spatial variability.

GROUNDWATER VELOCITY DETERMINATION

Hydraulic and forced-gradient tracer tests are used primarily to obtain estimates of the fracture aperture. Once the aperture and hydraulic gradient are known the groundwater velocity is readily calculated from Poiseuille's equation for laminar flow in a parallel plate opening and is given by

$$(7) \quad V = - \frac{pg}{12\mu} b^2 \frac{dh}{dl}$$

where μ is the dynamic viscosity of water, p is the density of water, and the hydraulic gradient is taken along the flowline. Substitution for a fracture aperture of 160 μm and a hydraulic gradient of 0.01 yields a groundwater flow velocity of approximately 20 m day^{-1} through FZ1. If this value is accurate, then groundwater contamination could quickly become widespread should any contaminant enter this fracture. Of course, numerous assumptions and uncertainties linger behind this value. For example, the withdrawal well for the hydraulic test may have been located in a large-aperture zone which may not reflect the average aperture in FZ1. This would lead to an overestimation of the velocity.

The groundwater velocity calculated from equation (7), however, can be checked by two additional independent methods: borehole dilution and natural-gradient tests. The borehole dilution method provides a direct measurement of the localized velocity rather than indirect calculation by equation (7). The test involves first isolating a fracture zone by means of packers and then monitoring the time rate of change of a suitable tracer contained within the packed off interval. From the concentration history and the volume of the packed off interval, a simple mass balance equation allows calculation of the mass flux flowing through the cross section of the borehole (Drost et al., 1968). The calculated mass flux can then be compared with the value estimated from equation (7)

$$(8) \quad q = db V$$

where d is the diameter of the borehole and db is the cross sectional area for flow from the fracture into the packed off interval. The natural-gradient test calls for the release of a tracer slug followed by the monitoring of the plume as it moves through the fracture under the in situ hydraulic gradient. From such data, effective velocities could be ascertained directly. To our knowledge, no record of a successful natural-gradient tracer test in fractured rock exists in the literature. Such a test would offer a unique opportunity to evaluate the reliability of groundwater velocities predicted from in situ measurement of fracture aperture and hydraulic head. In addition, it would also allow for the monitoring of a contaminant as it moves through fracture under a natural gradient and thereby make it possible to separate out advection, dispersion, and matrix diffusion effects. Both borehole dilution and natural-gradient tracer tests are expected to be conducted at the Clarkson field site early this fall. These tests will provide additional

constraints for the fracture aperture and groundwater flow velocities calculated from hydraulic and induced gradient tests.

SUMMARY

In this paper we have numerically examined potential errors in the interpretation of hydraulic and induced-gradient tracer tests that are carried out to obtain in situ values of fracture aperture. The volume balance aperture is not a reliable estimate because of its sensitivity to errors in the assumed geometry of the region where water is withdrawn. Large differences between hydraulic and tracer apertures are possible because of differing sensitivities to the effects of variability in fracture aperture and network geometry. To derive estimates of aperture and groundwater velocity using results of hydraulic and tracer tests, it appears that an approach is needed that provides for additional constraints on the flow field. Borehole dilution and natural-gradient tracer tests may provide direct measurements of the groundwater velocity, and thereby serve as additional checks and constraints for hydraulic or induced-gradient tracer tests.

Preliminary work at the Clarkson site yield a hydraulic aperture of 160 μm , a hydraulic gradient of 0.01, and a groundwater velocity of 20 m day^{-1} . Based on the encouraging results obtained from hydraulic tests of FZ1, four additional boreholes were drilled at the site in late summer 1987. The four holes were drilled equidistance from the points of the cross such that the central well, UW2, is surrounded by a square composed of eight wells. In late fall, we expect to conduct induced-gradient tracer tests to measure the in situ aperture and thereby provide a second estimate of the velocity in FZ1. In early 1988, borehole dilution and natural-gradient tests to measure the velocity directly are planned. Upon completion of these test, it will then be possible to assess whether the groundwater velocity determined from measurements of hydraulic head and fracture aperture correctly represents the in situ velocity in fracture zone FZ1. If our preliminary finding of large groundwater velocities in the Dundas shale at the Clarkson site is correct, then its is reasonable to expect that groundwater contamination could quickly become widespread should any contaminant enter the fracture zone.

REFERENCES

Abelin, H., I. Neretnieks, S. Tunbrant, and L. Moreno, Final report of the migration in a single fracture - Experimental results and evaluation, SKB Tech. Report 85-03, Stockholm, 1985.

Anderson, J., and R. Thunvik, Predicting mass transport in discrete fracture networks with the aid of geometrical field data, Water Resour. Res., 22, 1986.

Drost, W., Point dilution method of investigating groundwater flow by means of radioisotopes, Water Resour. Res., 4, 125-146, 1968.

Hodgkinson, D.P., and D.A. Lever, Interpretation of a field experiment on the transport of sorbed and nonsorbed tracers through a fracture in crystalline rock, Radioactive Waste management and the Nuclear Fuel Cycle, 4, 1983.

Novakowski, K.S., G.V. Evans, D.A. Lever, and K. Raven, A field example of measuring hydrodynamic dispersion in a single fracture, Water Resour. Res., 21, 1985.

Smith, L., C.W. Mase, and F.W. Schwartz, A numerical model for transport in networks of planar fractures, Inter. Assoc. Hydrogeologist, Sumpos. on Hydrogeology of Rocks of Low Permeability, Tuscon, Arizona, 1985.

APPENDIX IV

AREAL VARIATION AND CHEMICAL MODIFICATION OF
WEATHERED SHALE INFILTRATION CHARACTERISTICS

Areal Variation and Chemical Modification of Weathered Shale Infiltration Characteristics¹

R. J. LUXMOORE, B. P. SPALDING, AND I. M. MUNRO²

ABSTRACT

Spatial variability of infiltration into a weathered shale subsoil were evaluated at a site proximal to one used for shallow land burial of low-level radioactive waste at Oak Ridge National Laboratory. Double-ring infiltrometers were installed at 48 locations on a 2- by 2-m grid after the removal of 1 to 2 m of soil (Litz-Sequoia association, Typic Hapludults). Infiltration rates were measured before and during the 0- to 20- and 239- to 259-day periods following treatment with solutions of NaOH, KOH, NaF, NaAlO₂, and Na₂SiO₃ at rates of 151 equivalents/m². None of these chemical treatments significantly altered infiltration rate, indicating that chemical modification of soil exchange properties may be achieved without inducing hydrologic disturbance in these subsoils. A semivariogram analysis of infiltration data showed that areal variability was random; any spatial patterning must therefore occur at a smaller scale than 2 m. The frequency distribution of infiltration rate fitted a lognormal model with a geometric mean of 2 cm per day and a coefficient of variation of 130%.

Additional Index Words: drainage, hydraulic conductivity, lognormal distribution, semivariogram, radioactive waste.

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SHALLOW LAND BURIAL is the routine method for the disposal of low-level radioactive waste at a number of government and commercial facilities (Carter et al., 1979). At Oak Ridge National Laboratory, leaching of buried waste by groundwater for periods of up to 30 years has led to the movement of some ⁹⁰Sr from the waste to soils and sediments surrounding such disposal areas. To understand and mathematically describe the transport of ⁹⁰Sr, information is needed on the hydraulic properties of the subsoil used for this waste disposal. Soil profiles exposed by the routine excavation of burial trenches show considerable fracturing and folding and have a strong structural C horizon. In addition, both textural and color differences among the C-horizon strata are obvious by visual inspection. Knowing how this stratigraphic heterogeneity is related to groundwater movement, both into

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and out of burial trenches, would be extremely useful in assessing the potential transport of radionuclides. Although average groundwater velocities and fluxes can be determined for drainage basins by monitoring water inputs and outputs, such approaches do not lend themselves to establishing the variability of the rates at which groundwater can flow through such a fractured, stratified subsoil medium. Therefore, a major objective of the present study was to characterize the areal variability in infiltration rates through this weathered shale subsoil on a scale applicable to the basal area of typical burial trenches ($\sim 50 \text{ m}^2$).

In addition, recent work has indicated that treatment of soil with solutions of selected chemicals can lead to increased fixation and retarded leaching of radiostrontium (Spalding, 1980, 1981). However, the influence of such chemicals as alkali metal hydroxides, fluorides, carbonates, silicates, and aluminates on *in situ* hydraulic properties of this weathered shale subsoil are unknown. Many of these alkaline chemicals are excellent soil dispersing agents (Day, 1965) and might be expected to effect a considerable decrease in hydraulic conductivity of soil. Any potential effects of such chemicals on subsoil hydraulic conductivity must be coupled with their effects on soil adsorptive properties to assess the integrated effect of such chemical treatment on radiostrontium transport.

MATERIALS AND METHODS

Site Preparation and Characterization

An approximately 800-m² area located near Solid Waste Disposal Areas 4 and 5 (Webster, 1979) at Oak Ridge National Laboratory was cleared of second growth pine. Before clearing, this area was comparatively level but well drained, due to 15 to 20% slopes on the adjacent east, west, and south sides. Soils in this location have been mapped as members of the Litz-Sequoia association (Soil Conservation Service, 1961) and would be classified as Typic Hapludults. Tree stumps and the A- and B-soil horizons were bulldozed downslope, removing a total of 1 to 2 m of soil. This removal exposed that portion of the

profile which was largely free of tree roots; the area then appeared typical in stratification to the horizons used for waste disposal. A slope of 1 to 2% to the south in the final surface was established to facilitate surface runoff of precipitation during the 12-month study period. A final 3- to 7-cm depth of the exposed surface was removed with a road grader, yielding a well-defined exposure of the site stratification. All of the exposed C-horizon material was weathered Conasauga shale which exhibited a strong, coarse, angular-block structure. The mineralogy of this Conasauga shale was dominated by illite, vermiculite, and quartz (Tamura and Jacobs, 1960).

Eight samples of soil were collected from the study site: six equispaced along the perimeter of, and two from within, the rectangular array of infiltrimeters described below and depicted in Fig. 1. Bulk density was determined by the paraffin clod method (Blake, 1965), using 5- to 10-cm long by 4.7-cm diam cores. Samples for particle size analysis and chemical characterization were removed from the 0- to 10-cm depth of the area surrounding the core holes. These loose samples were dried at 70°C for 48 hours prior to sieving. Manual sieving (< 2 mm) was performed by rubbing aggregates, in several batches, over a wire screen for about 2 min. Oven-dry weights of both the coarse and < 2-mm fractions were determined on 20- to 40-g aliquots (105°C, 16 hours) and are the basis for expressing all analyses. Particle size analysis of the < 2-mm fraction was performed by the hydrometer method (Day, 1965) after shaking the soil overnight (100 oscillations/min) in 0.5% sodium hexametaphosphate. Organic matter was determined by the Walkley-Black method (Allison, 1965). Exchangeable acidity was determined by the BaCl₂-triethanolamine method (Peech, 1965). Exchangeable Ca, Mg, K, and Na were determined by atomic absorption of 1N ammonium acetate, pH 7.0, soil extracts (Chapman, 1965). Cation exchange capacity (CEC) was calculated as the sum of exchangeable bases and exchangeable acidity. Soil pH was measured with a glass electrode in 0.01M CaCl₂ (2:1 solution-to-soil ratio). The average characteristics for these eight samples (Table 1) show a large proportion of coarse fragments, a low degree of base saturation, and a low pH.

Infiltrimeter Design and Operation

Double-ring infiltrimeters were inserted into the soil to a depth of 8 to 10 cm with the hydraulic press of a 5-ton drilling rig. Care was observed to minimize any rocking motion of the rings during insertion by employing a 2.5-cm thick steel cover to hold each ring during insertion. Inner rings were inserted first, followed by the concentrically placed outer ring. Rings were constructed of 0.3- by 30.5-cm steel plate with inner rings fabricated to a 40.6-cm diam and outer rings to a 61.0-cm diam. The lower edges were beveled to facilitate insertion. A total of 48 infiltrimeters were installed in a 6 by 8 rectangular array on a 2- by 2-m grid. Each of the eight rows of six infiltrimeters was set on a line parallel to the strike of the remnant bedding planes of the shale. The surface 2 cm of the soil within each inner ring was removed with a trowel to achieve a reasonably undisturbed surface structure; loose fragments were removed by vacuuming out the inner ring. Sealing along the inside wall of the inner ring was assured by directing a slow

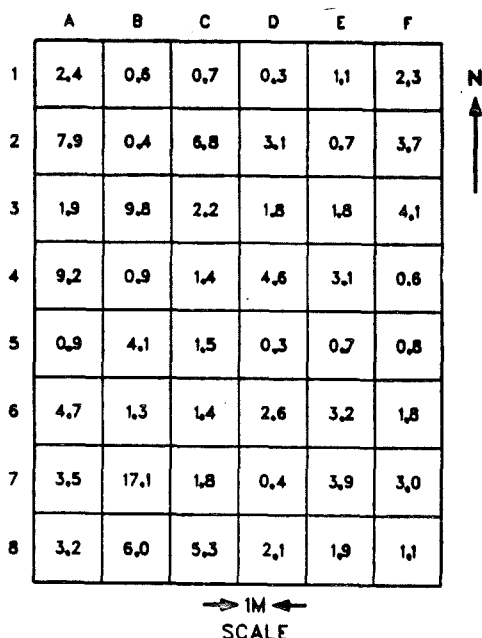


Fig. 1—Infiltration rates (cm/day) for 48 locations on 2- by 2-m grid obtained before chemical treatment.

Table 1—Average characteristics of the Litz-Sequoia subsoil from the study site.

Character	Units	Mean†	Standard deviation
Bulk density	g/cm ³	1.63	0.17
Coarse fragments	% > 2 mm	68	6
Fine material	% < 2 mm	32	—
Clay	%	42	10
Silt	%	22	6
Sand	%	35	11
Organic matter	%	0.3	0.3
Exchangeable cations (< 2 mm fraction)			
H ⁺	meq/100 g	16.8	2.3
Ca ²⁺	meq/100 g	0.5	0.5
Mg ²⁺	meq/100 g	1.5	0.9
K ⁺	meq/100 g	0.3	0.1
Na ⁺	meq/100 g	0.1	0.1
Cation exchange capacity	meq/100 g	19.1	3.1
pH	—	3.2	0.1

† Mean of eight samples (0- to 10-cm depth) collected from within and on the perimeter of the study site.

flow of 90% (clay weight/slurry volume) bentonite slurry along the inside circumference. This slurry penetrated no more than 0.5 cm from the inner wall and minimized water flow along the ring-soil interface. A 2- to 3-cm layer of coarse sand was placed over the soil surface of the inner and outer rings to prevent disturbance during addition of water or chemical.

Infiltration measurements were begun by adding water to within 0.5 cm of the top of both the inner and outer rings and, at intervals, recording the elevation of water in the inner ring. Actual measurements by this falling head method were not begun until after the soil had remained under standing water for up to 3 weeks by daily water addition to the infiltrometers. The rather slow infiltration rates obviated the initial design to use Mariotte-bottle type constant-head devices; diurnal and periodic temperature fluctuations caused very significant volume changes in the airspace of supply reservoirs. Infiltration was followed for 3 days with several observations of water elevation per day. Water levels were restored to the initial levels daily, or more often if necessary, in both the inner and outer rings during the measurement period. Evaporation losses as well as precipitation gains in water levels were avoided by covering each infiltrometer with a plastic-covered wooden lid when measurements were not being taken. Following these infiltration measurements, water was allowed to drain out of both rings or, for slower infiltration rates, was siphoned or bailed out before treating with chemical solutions. The ring water-level measurements were estimated to within 0.05 cm, and calculated infiltration rates were estimated to within 0.1 cm/day.

Chemical Treatment

Five chemicals were selected for subsoil treatment: NaOH, KOH, NaAlO₂, Na₂SiO₃, and NaF. Because strongly acid soils, similar to those at this site, can react with up to 0.4-meq NaOH/g, a treatment level of one-half of this maximum was selected. Particle size has little effect on this potential reactivity of this shaley soil material since the larger particles (> 2 mm) are only aggregates composed of the same minerals as the smaller particles. Our unpublished observations have shown that the amounts of exchangeable cations and acidity are substantially the same in the > 5-mm, 5- to 2-mm, and < 2-mm size fractions. A calculated equivalent weight of each chemical to react with a 1-m deep cylindrical volume of soil below the inner infiltrometer ring was dissolved in 10 liters of water. At an assumed bulk density of 1.5 g/cm³, this soil volume would have a mass of 97.5 kg and would therefore require 19.5 equivalents of a given chemical. Since solubility limited the desired concentration, 20 liters of water were employed in the case of NaF. The technical-grade NaAlO₂ failed to dissolve completely with 2 days of continuous stirring; the residual undissolved material (approximately 20%) was discarded by decanting the dissolved phase. The chemical solutions were poured into the inner ring and allowed to infiltrate until the solution levels in all infiltrometers were below the sand layer. Many of the infiltrometers remained dry for up to 10 days before solutions at the slower sites drained into the sand surface.

Based on the infiltration rates before chemical treatment, rings were classified into eight groups of six with similar infiltration rates. Within each of these rate groups, rings were randomly selected for one of the five chemical treatments or for continued functioning as an untreated control. This experimental design yielded eight replicate observations per chemical treatment, stratified over eight infiltration rate ranges. Inner and outer rings were again flooded, and the infiltration measurements were repeated as described above. Measurements were continued on each infiltrometer until a cumulative infiltration of 30 to 35 cm had been achieved, or, in the case of slower rates, 20 days of observed values had been recorded. To observe any longer-term effects of these chemical treatments on infiltration rates, the infiltration measurements were repeated during 239 to 259 days after treatment. During this 8-month period the infiltrometer rings were covered to prevent rainfall entry. This allowed the chemicals time to react with the shale subsoil and perhaps change infiltration characteristics.

Calculations

a) *Infiltration Rate Estimation*—A linear regression was employed to estimate the infiltration rate (cm/day) from a plot of cumulative infiltration vs. time for both the before- and after-chemical treatment observations. The before-treatment observations were generally begun under saturated conditions because

standing water had been maintained in most of the infiltrometers for more than 2 weeks prior to measurements. Under such conditions, the early, nonlinear portion of the water intake curve into a dry soil (Philip, 1957) was rarely observed. Because of the differential rates of chemical solution infiltration, many infiltrometers had been free of standing water for up to 10 days when the after-treatment infiltration measurements were initiated. Such initial, unsaturated conditions led to typical nonlinear water uptake vs. time plots (Philip, 1957). In all cases, the infiltration rates were estimated from the linear portion of the infiltration curves. This was achieved by the regression of cumulative infiltration on time after deleting any observation(s) at times less than one day post-flooding. Similar deletions of the nonlinear portion of the infiltration curves were also performed on both the eight-month post- and pre-treatment observations.

b) *Semivariogram*—The 48 infiltration rate values estimated for locations on a 2- by 2-m grid were analyzed statistically by the semivariogram procedure as an aid to the identification of any spatial patterning of infiltration rate over the 250-m² study area. The semivariogram is a continuous function which relates the semivariance between points as a function of the distance between them (Burgess and Webster, 1980). Semivariance is calculated as one-half of the variance of the infiltration rate differences between points, for each point-to-point combination in the data set. For example, the semivariance for locations separated by 2 m (2-m lag) is given by

$$n = 82$$

$$\sum_{i=1}^n \frac{(R_{i+1} - R_i)^2}{2}$$

$$n-1$$

where R is the infiltration rate. It is noted that the 48 grid values have 82 difference combinations at 2-m spacings. The value of n and the infiltration rate combinations change with increase in lag distance. The semivariances for 11 average lag distances between points were calculated with a computer program written by Dr. Jack Metthey, Dep. of Land, Air and Water Resources, Univ. of Calif., Davis.

RESULTS

Infiltration rates determined before chemical treatment show a range of values from 0.3 to 17.1 cm/day, and visual examination of the 48 values suggest some spatial patterning with high values occurring in the A6, A7, A8, B7, B8, and C8 locations (Fig. 1). The semivariogram (Fig. 2) analysis of the infiltration data, however, shows that there is no basis for establishing any areal pattern at the study site since the smaller separation distances do not have lower semivariances.

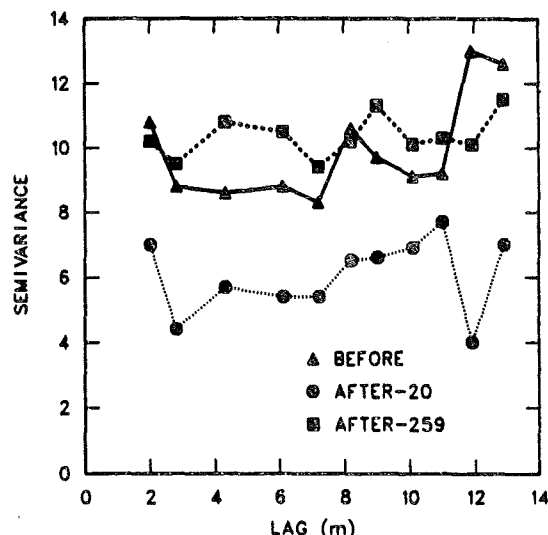


Fig. 2—Semivariograms of infiltration rate data obtained before and at 20 and 259 days after chemical treatment.

Table 2—Mean infiltration rates of weathered shale obtained before and at 20 and 259 days after five chemical treatments.

Chemical	Infiltration rate (cm/day)		
	Before	After 20 days	After 259 days
Control	2.6	2.6	2.9
NaOH	2.1	2.7	2.6
KOH	2.6	2.1	2.0
Na ₂ SiO ₃	3.0	1.9	4.3
NaAlO ₂	3.6	3.3	3.0
NaF	4.1	2.9	2.3
Mean	3.0	2.6	2.9

In other words, neighboring locations do not have similar infiltration rates. The arithmetic mean of the 48 infiltration values is 3.0 cm/day with a variance of 9.3. This variance lies within the range of semi-variances in Fig. 2. The conclusion may be made that the data have no spatial structure and can be described by frequency distribution properties. Spatial structure may occur at the site but at lag distances < 2 m.

An analysis of variance was conducted on the two sets of infiltration data obtained after chemical treatment (6 treatments times 8 replicates) to evaluate the effect of the chemicals on infiltration rate. There were no significant effects or trends that could be identified in the data analyses (Table 2). Similar statistical analyses conducted on the difference in infiltration rate before and after chemical treatments showed no significant effects. The arithmetic mean of 2.6 cm/day for the 48 values obtained 20 days after chemical treatment was lower than the before-chemical data but not statistically lower. The variance for these after-chemical data of 5.9 is within the range of semi-variance values for the 20-day after-chemical data (Fig. 2). The final infiltration data obtained 259 days after chemical treatment had an arithmetic mean of 2.9 cm/day with a variance of 10.1.

Frequency distributions determined for the before- and after-chemical infiltration rates based on 0.5-cm/day class intervals show patterns that were strongly skewed to low values. These distributions suggest a lognormal model. Two tests for lognormality were conducted with the data. The first test used log-transformed data and the second (k_3), third (k_3), and fourth (k_4) moments of the corresponding frequency distribution. Four new parameters were calculated with the following (Fisher, 1973):

$$g_1 = k_3/k_2^{1.5}(\text{skewness}) \quad g_2 = k_4/k_2^2(\text{kurtosis}),$$

$$\sqrt{b_1} = \frac{(n-2)}{n(n-1)} g_1, \text{ and}$$

$$b_2 = \frac{(n-2)(n-3)}{(n-1)(n+1)} g_2 + \frac{3(n-1)}{(n+1)},$$

where n is the number of data values, 48 in this case. The $\sqrt{b_1}$ and b_2 parameters were used to estimate the probability that the log-transformed data satisfied a normal frequency distribution characteristic using isopleth probability figures given by Bowman and Shenton (1975). In all three cases, the probability that the experimental data were from lognormal frequency distributions was greater than 99%.

A second, more conservative test was made using the Shapiro and Wilk (1965) method, available as a computer subroutine in the SAS software system (Helwig

and Council, 1979). The values for the W-statistic calculated for the log-transformed data were 0.981, 0.982, and 0.974 for the before- and the 20- and 259-day after-chemical treatments, respectively. The accompanying probabilities that the data were from lognormal frequency distributions were 75, 80, and 50%, respectively. Although the 50% probability is low, the lognormal frequency distribution can be viewed as an acceptable model for representing infiltration rate data from the study site. This being the case, the geometric mean infiltration rates are better than arithmetic means for representing a characteristic rate of the site. The geometric means for the before- and the 20- and 259-day after-chemical treatment infiltration rates were 2.0, 1.8, and 1.5 cm/day, respectively, with corresponding coefficients of variation of 129, 128, and 222%.

DISCUSSION

A high degree of variability in infiltration rate was determined for the 250-m² weathered, shale study area. High variability in soil properties is well known for physical (Nielsen et al., 1973; Carvallo et al., 1976; Warrick et al., 1977) and chemical characteristics (Beckett and Webster, 1971); the coefficient of variation of 130% and higher exceeds many other published data, however. Sharma et al. (1980) reported a 60% coefficient of variation in infiltration characteristics obtained at 26 grid locations over a 8.6-ha watershed. At smaller space scales, Beckett and Webster (1971) concluded that one-half of the variance within a field may be present within any one square meter of it.

The lognormal frequency distribution of infiltration rate agrees with the pattern noted by Rao et al. (1979) that flux-related soil variables (e.g., hydraulic conductivity, air permeability, solute dispersion coefficient) tend to be characterized by lognormal frequency distributions, whereas state variables (e.g., bulk density, clay content, water content at a given pressure) tend to be normally distributed. Sisson and Wierenga (1981) and Sharma et al. (1980) report lognormal frequency distributions for infiltration characteristics of a small plot (40 m²) and watershed, respectively.

The test for spatial patterning using the semivariogram approach proved negative. This leads to the conclusion that if patterning exists at all, it occurs on a spatial scale less than the 2 m used in this study. Peck and Watson (1979) showed that the spatial variation in saturated hydraulic conductivity measured in 214 wells (100- to 200-m grid spacing) over five separate watersheds was essentially random since statistical analysis showed a poor correlation between neighboring conductivity values. The hydraulic conductivity data were also shown to exhibit a lognormal frequency distribution. Some spatial patterning was identified in the detailed infiltration rate data obtained by Sisson and Wierenga (1981). Their infiltration rates measured in 125 consecutive 5-cm-diam rings on each of five line transects (625 total measurements) were shown to follow a first-order autoregressive process. Neighboring infiltration rates tended to be similar; with increasing distances away from a given location, covariances between locations diminished.

The fact that high rates of chemical application did not alter the infiltration characteristics of the weathered shale attests to the structural and chemical stabil-

ity of the matrix. The present subsoil material has maintained its structure through many centuries of weathering, and it will probably continue to maintain its structure. The alkaline chemical treatments have been shown to increase the retention of ^{85}Sr (Spalding, 1980) by greatly increasing the CEC of the shale. Although much useful field-scale knowledge could have been gained by determining the distribution of salts and exchangeable cations in the soil below the infiltrometers after the experiment, the required destructive sampling precluded this because of the need to preserve the site for future investigations. The percolation of such chemical solutions at equivalent application rates through laboratory-scale soil columns has been shown to achieve the chemical modifications necessary for increased radiostrontium retention in similar soils (Spalding, 1981). These results suggest that chemical pretreatment of trenches before addition of contaminated waste would provide a desirable influence without modification of the trench hydrologic regime.

CONCLUSIONS

- 1) Weathered shale exhibited very large variability in infiltration rate with a geometric mean value of 2 cm/day.
- 2) Infiltration rate exhibited a lognormal frequency distribution.
- 3) Infiltration rate was a randomly distributed property with no spatial patterning at a 2-m grid spacing.
- 4) Alkaline, Na, and K treatments did not change infiltration into the strongly structured C-horizon.

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LITERATURE CITED

1. Allison, L. E. 1965. Organic carbon. In C. A. Black et al. (ed.) *Methods of soil analysis*, part 2. Agronomy 9:1372-1376. Am. Soc. of Agron., Madison, Wis.
2. Beckett, P. H. T., and R. Webster. 1971. Soil variability: a review. *Soils Fert.* 34:1-15.
3. Blake, G. R. 1965. Bulk density. In C. A. Black (ed.) *Methods of soil analysis*, part 1. Agronomy 9:375-377. Am. Soc. of Agron., Madison, Wis.
4. Bowman, K. O., and L. R. Shenton. 1975. Omnibus test contours for departures from normality based on $\sqrt{b_1}$ and b_2 . *Biometrika* 62:243-250.
5. Burgess, T. M., and R. Webster. 1980. Optimal interpolation and isarithmic mapping of soil properties. 1. The semivariogram and punctual kriging. *J. Soil Sci.* 31:315-331.
6. Carter, M. W., A. A. Moghissi, and B. Kahn. 1979. Management of low-level radioactive waste. Vol. 1 and 2. Pergamon Press, New York.
7. Carvallo, H. O., D. K. Cassel, J. Hammond, and A. Bauer. 1976. Spatial variability of *in situ* unsaturated conductivity of Maddock sandy loam. *Soil Sci.* 121:1-8.
8. Chapman, H. D. 1965. Total exchangeable bases. In C. A. Black (ed.) *Methods of soil analysis*, part 2. Agronomy 9:902-904. Am. Soc. of Agron., Madison, Wis.
9. Day, P. R. 1965. Particle fractionation and particle-size analysis. In C. A. Black (ed.) *Methods of soil analysis*, part 1. Agronomy 9:545-577. Am. Soc. of Agron., Madison, Wis.
10. Fisher, R. A. 1973. Statistical methods for research workers. Hafner Publ. Co., New York.
11. Helwig, J. T., and K. A. Council (ed.) SAS User's Guide, 1979 Edition. SAS Institute Inc., Raleigh, N.C.
12. Nielsen, D. R., J. W. Biggar, and K. T. Erh. 1973. Spatial variability of field measured soil-water properties. *Hilgardia* 42:215-259.
13. Peck, A. J., and J. D. Watson. 1979. Hydraulic conductivity and flow in non-uniform soil. p. 31-39. In *Workshop on soil physics and soil heterogeneity*. Division of Environmental Mechanics, C.S.I.R.O., Canberra, Australia.
14. Peech, M. 1965. Exchange acidity. In C. A. Black (ed.) *Methods of soil analysis*, part 2. Agronomy 9:910-911. Am. Soc. of Agron., Madison, Wis.
15. Philip, J. R. 1957. The theory of infiltration. 4. Sorptivity and algebraic infiltration equations. *Soil Sci.* 84:257-264.
16. Rao, P. V., P. S. C. Rao, J. M. Davidson, and L. C. Hammond. 1979. Use of goodness-of-fit tests for spatial variability of soil properties. *Soil Sci. Soc. Am. J.* 43:274-278.
17. Shapiro, S. S., and M. B. Wilk. 1965. An analysis of variance test for normality (complete samples). *Biometrika* 52:591-611.
18. Sharma, M. L., G. A. Grander, and C. G. Hunt. 1980. Spatial variability of infiltration in a watershed. *J. Hydrol.* 45:101-122.
19. Sisson, J. B., and P. J. Wierenga. 1981. Spatial variability of steady-state infiltration rates as a stochastic process. *Soil Sci. Soc. Am. J.* 45:699-704.
20. Soil Conservation Service. 1961. Soil survey of Loudon County, Tennessee. p. 75-89. In *Soil Survey Series 1958*, no. 2, USDA, U.S. Government Printing Office, Washington, DC.
21. Spalding, B. P. 1980. Adsorption of radiostrontium by soil treated with alkali metal hydroxides. *Soil Sci. Soc. Am. J.* 44:703-709.
22. Spalding, B. P. 1981. Chemical treatments of soil to decrease radiostrontium leachability. *J. Environ. Qual.* 10:42-46.
23. Tamura, T., and D. G. Jacobs. 1960. Structural implications in cesium sorption. *Health Phys.* 2:391-398.
24. Warrick, A. W., G. J. Mullen, and D. R. Nielsen. 1977. Scaling field-measured soil hydraulic properties using similar-media concept. *Water Resour. Res.* 13:355-362.
25. Webster, D. A. 1979. Land burial of solid radioactive waste at Oak Ridge National Laboratory, Tennessee. A case history. p. 731-746. In M. W. Carter, A. A. Moghissi, and B. Kahn (ed.) *Management of low-level radioactive waste*. Pergamon Press, New York.