M-923

Peto MacCallum Ltd.
CONSULTING ENGINEERS

WITNESS STATEMENT OF BRIAN R. GRAY, M.Eng., P.Eng.
(Reply Evidence)

RE: HALTON LANDFILL UNDERTAKING

Index of Tables, Figures and Appendices

165 Cartwright Avenue, Toronto, Ontario M6A 1V5
Tel: (416) 785-5110 Fax: (416) 785-5120
OFFICES IN TORONTO, HAMILTON, KITCHENER, BARRIE, OSHAWA

1. I am Vice-President, Geotechnical Engineering at Peto MacCallum Ltd., Consulting Engineers. I specialize in geotechnical and materials engineering relating to soil mechanics, foundation and earthworks construction. My curriculum vitae is attached as Appendix "A".

2. At the request of David Estrin, Barristers & Solicitors, I have reviewed evidence and reports presented by Conestoga Rovers and Associates (CRA) with respect to their position as to the need for a liner at Site F, and as to whether or not, assuming that a liner is deemed necessary, one could be constructed from on-site materials. I have also reviewed CRA's cost estimates for construction of a liner at Site F.

3. It is my professional opinion that, assuming a liner is deemed necessary at Site F, a 1-metre thick liner would be satisfactory.

4. I have reviewed a letter dated June 22, 1988 from the University of Western Ontario Geotechnical Research Centre to Geocon Inc. (Appendix "B"), reporting on laboratory testing of samples of weathered shale and Halton till taken at Site F. I have also reviewed a letter dated August 2, 1988 from Geocon Inc. to David Estrin, Barristers & Solicitors (Appendix "C"). I am satisfied

that at Site F on-site clay material is suitable for use in a liner, and on-site shale is suitable for use as intermediate and final cover.

5. With only some relatively straightforward changes to the Trow design of Site F, there would be sufficient materials available on-site to construct a 1-metre thick liner and to provide intermediate and final cover. Furthermore, the use of on-site materials as proposed would be operationally practicable.

6. Table IA shows the available site capacity and available volume of on-site materials at Site F if Trow's design is implemented.

7. Table IB shows the estimated additional volumes of material that would be generated if the Trow design base elevations were to be lowered by 1 metre. I estimate that an additional 373,000 m^3 of material would be generated, comprising approximately 50% clay (185,500 m^3) and 50% shale (187,500 m^3). The material required for infill of stream valleys would be reduced by approximately 117,000 m^3.

8. The estimates of additional volumes referred to in paragraph 7 above are derived from a review of the logs of

exploratory boreholes on Site F, as detailed in Table IV. With reference to Table IV, it is to be noted that there is substantial thickness of clay overburden below the proposed new landfill base: about 8 m at the north end of the property as represented by Borehole HC-1, and typically 4 to 8 m at the south end of the site, as represented by Boreholes HC-5, HC-6, HC-8. Elsewhere, in the central section of the site, shale outcrops occur.

9. Table IC shows the estimated total volumes of material that would be available if the base of the landfill is lowered by 1 metre.

10. Table II is a summary of the total materials required and total materials available after allowing for bulking of clay (10%) and shale (15%) if the base of the landfill is lowered by 1 metre and a 1.0 metre thick clay liner is constructed.

For Cell 1, the total volume required (clay for liner, clay and/or shale for cover) is 1,364,970 m^3. Total material available in Cell 1 is 1,612,549 m^3. Of this, about 388,619 m^3 is clay. This is more than sufficient to meet the requirement of 325,500 m^3 of clay for the Cell 1 liner. It would not be necessary to haul clay from the Cell 2 area, which could therefore be left unworked until it is actually needed for landfill at a much later date.

The total volume of material required for Cell 2 is 1,018,245 m^3 and a total of 842,520 m^3 is available. With the surplus material from Cell 1, there would be no shortage. Cell 2 would generate more than sufficient clay (618,904 m^3) to meet the requirements for liner construction (234,500 m^3) in that cell.

11. In my opinion, based on estimates I received from four earthworks contractors, CRA's estimate of $6.25 per m^3 for the cost of using on-site clay to construct a liner is a conservative (high) estimate but it is not unreasonable. This price would include excavation, hauling, spreading and recompaction of the clay on-site. Contractors from whom estimates were received are: Dufferin Construction, Toronto; Rumble Construction, Mississauga; Costa Earth Movers Inc., Concord; and Graham Brothers, Brampton.

12. Based on estimates from the above noted earthworks contractors, I estimate the cost of using on-site shale for intermediate and final cover, including excavation, hauling, processing, spreading and recompaction, to be approximately $8.00 per m^3. This is a conservative (high) estimate.

13. Table III summarizes the costs of the various liner-construction scenarios at Site F. The Table shows that by reducing the liner thickness from 1.2 m to 1.0 m, and by using only on-site clay, as I propose (Case #4), there would be a cost saving of $4,075,000 over CRA's proposal (Case #1). If, in addition to these modifications, the sand underdrain pad and associated monitoring/contingency pumping equipment were to be eliminated, a further $3,702,300 would be saved for a total cost savings of $7,777,300 over CRA's Case #1 scenario.

14. Figure 1 with attached Commentary illustrates how development of Site F could proceed if the base elevations are lowered by 1m and only on-site material is used for liner and cover.

15. A sand blanket under the liner is not required. It would be another level of redundancy.

16. I do not agree with Mr. Sheppard's position that the Hazen formula for determining the permeability of sand is not applicable to sand. In my opinion, the formula is primarily applicable to sand.

17. Figure 2 presents the results of a grain size analysis of a sample of "winter sand" that was obtained from Sherman

Sand & Gravel Ltd. and delivered to our laboratory. As shown on Figure 2, the sample meets the MTC gradation requirements for winter sand (Ontario Provincial Standard Specification #1004).

18. Figure 3 presents the results of a standard Proctor compaction test conducted on the same sample of winter sand. The results of a falling head permeability test on a sample prepared in our lab at 95% standard Proctor compaction density yielded a coefficient of permeability of 2×10^{-2} cm/sec. This correlates well with the estimate of the coefficient of permeability using Hazen's formula which would give about 8×10^{-1} cm/sec to 1×10^{-2} cm/sec.

underdrain

19. Assuming that a sand blanket is deemed necessary at Site F, Sherman Sand & Gravel's "winter sand" would be suitable material for the purpose.

TABLE I A

AVAILABLE VOLUMES (TROW DESIGN) *

CELL No.	PARCEL No.	TOTAL ** GROSS LANDFILL CAPACITY (m^3)	TOTAL MATERIAL TO BE EXCAVATED (m^3)	ESTIMATED CLAY (m^3)	ESTIMATED SHALE (m^3)	ESTIMATED TOPSOIL (m^3)	ESTIMATED INFILL REQUIRED (m^3)
1.	1-4	4,703,740	1,232,090	306,790	920,600	4,700	173,120
2.	5-7	3,379,860	570,270	423,640	146,010	620	150,370
TOTAL		8,083,600	1,802,360	730,430	1,066,610	5,320	323,490

* Data from Ex. H42-B, Addendum to the Site F Design and Operations Report (Trow, May 1987), Table 3-r

TABLE I B

ESTIMATED ADDITIONAL VOLUMES BY LOWERING LANDFILL BASE 1 M

1.	1-4	260,500	186,100	46,500	139,600	0	-74,400
2.	5-7	229,500	186,900	139,000	47,900	0	-42,600
TOTAL		490,000	373,000	185,500	187,500	0	-117,000

which takes this up

TABLE I C

TOTAL ESTIMATED VOLUME FOR LANDFILL BASE LOWERED 1 M

CELL No.	PARCEL No.	TOTAL ** GROSS LANDFILL CAPACITY (m^3)	TOTAL MATERIAL TO BE EXCAVATED (m^3)	ESTIMATED CLAY (m^3)	ESTIMATED SHALE (m^3)	ESTIMATED TOPSOIL (m^3)	ESTIMATED INFILL REQUIRED (m^3)
1.	1-4	4,703,740	1,418,190	353,290	1,060,200	4,700	98,720
2.	5-7	3,379,860	757,170	562,640	193,910	620	107,770
TOTAL		8,083,600	2,175,360	915,930	1,254,110	5,320	206,490

** **Total Gross Landfill Capacity** includes total available volume for wastes and for intermediate and final cover, at 4:1 waste to cover ratio.

NOTES:
(1) All volumes are in-situ. (assume no bulking)
(2) Quantities listed assume a vertical separation between ajacent cells.
(3) Stream valleys to be infilled with clean fill.
(4) Cells 1 and 2, and Parcels 1 to 7 are as shown on Ex. H-42B, Addendum to the Site F Design and Operations Report (Trow, May 1987)

TABLE II

SUMMARY FILL REQUIREMENTS

Cell No.	Total Materials Required (m^3)	Total Materials Available (m^3)	Total Clay Required (m^3)	Total Clay Available (m^3)
1.	1,364,970	1,612,549	325,500	388,619
2.	1,018,245	842,520	234,500	618,904
TOTAL	2,383,215	2,455,069	560,000	1,007,523

NOTES:

1) **Total Materials** and **Clay** volumes required and available assumes landfill base lowered 1.0m.

2) **Total Materials Required** includes volumes for daily and final cover, clay liner, dykes and berms, and infill areas.

3) **Total Clay Required** includes volumes for 1.0m thick liner and 70,000m^3 for dykes and berms.

4) Assume bulking - clay 10%, shale 15%.

TABLE III

COST COMPARISON

Case	Liner Details	Clay Liner Cost	Clay Liner Cost Savings from CRA Design	Total Potential Cost Savings from CRA Design *
1.	CRA Estimate •$600,000 m^3$ clay required (half on-site/half imported) •liner thickness 1.2m	$(300,000 m^3 \times \$6.25/m^3) +$ $(300,000 m^3 \times \$17.75/m^3)$ = $7,200,000	$0	$3,702,300
2.	Trow Estimate •$550,000 m^3$ clay required ($480,000 m^3$ clay liner, $70,000 m^3$ dykes/berms) •liner thickness 1.0m •use site clay entirely	$480,000 m^3 \times \$8.50/m^3$ = $4,080,000	$3,120,000	$6,822,300
3.	CRA Estimate Modified (a) reduce liner thickness to 1.0m •$500,000 m^3$ clay required ($300,000 m^3$ onsite - balance imported)	$(300,000 m^3 \times \$6.25/m^3) +$ $(200,000 m^3 \times \$17.75/m^3)$ = $5,425,000	$1,775,000	$5,477,300
	(b) Use on-site clay entirely •$600,000 m^3$ clay required •liner thickness 1.2m	$600,000 m^3 \times \$6.25/m^3$ = $3,750,000	$3,450,000	$7,152,300

TABLE III CONTINUED

COST COMPARISON

Case	Liner Details	Clay Liner Cost	Clay Liner Cost Savings from CRA Design	Total Potential Cost Savings from CRA Design *
4.	Peto MacCallum Estimate •Use on-site clay entirely and reduce liner to 1.0m •$500,000m^3$ clay required •liner thickness 1.0m	$500,000m^3 \times \$6.25/m^3$ = \$3,125,000	\$4,075,000	\$7,777,300

* **Total Potential Cost Savings** assumes sand underdrain pad and associated monitoring/contingency pumping equipment is eliminated, for a saving \$3,702,300 (CRA's estimated cost).

This potential saving is reflected in all totals in this column.

TABLE IV

SITE F
CLAY-SHALE BOUNDARY ELEVATION
FROM LOGS OF EXPLORATORY BOREHOLES
AND WATER SUPPLY WELLS *

Borehole Designation	Ground Surface Elevation	Elevation Clay Shale Boundary	Proposed Base Elevation**
HC-1	187.1	167.8	176
HC-2	175.5	170.9	170
MB-4	168.0	165.0	153
HC-4	137.4	137.4	141
HC-3	140.3	140.3	139
MB-5	127.0	123.4	127
HC-5	125.9	117.8	127
HC-6	127.8	122.9	127
HC-8	118.0	112.4	119

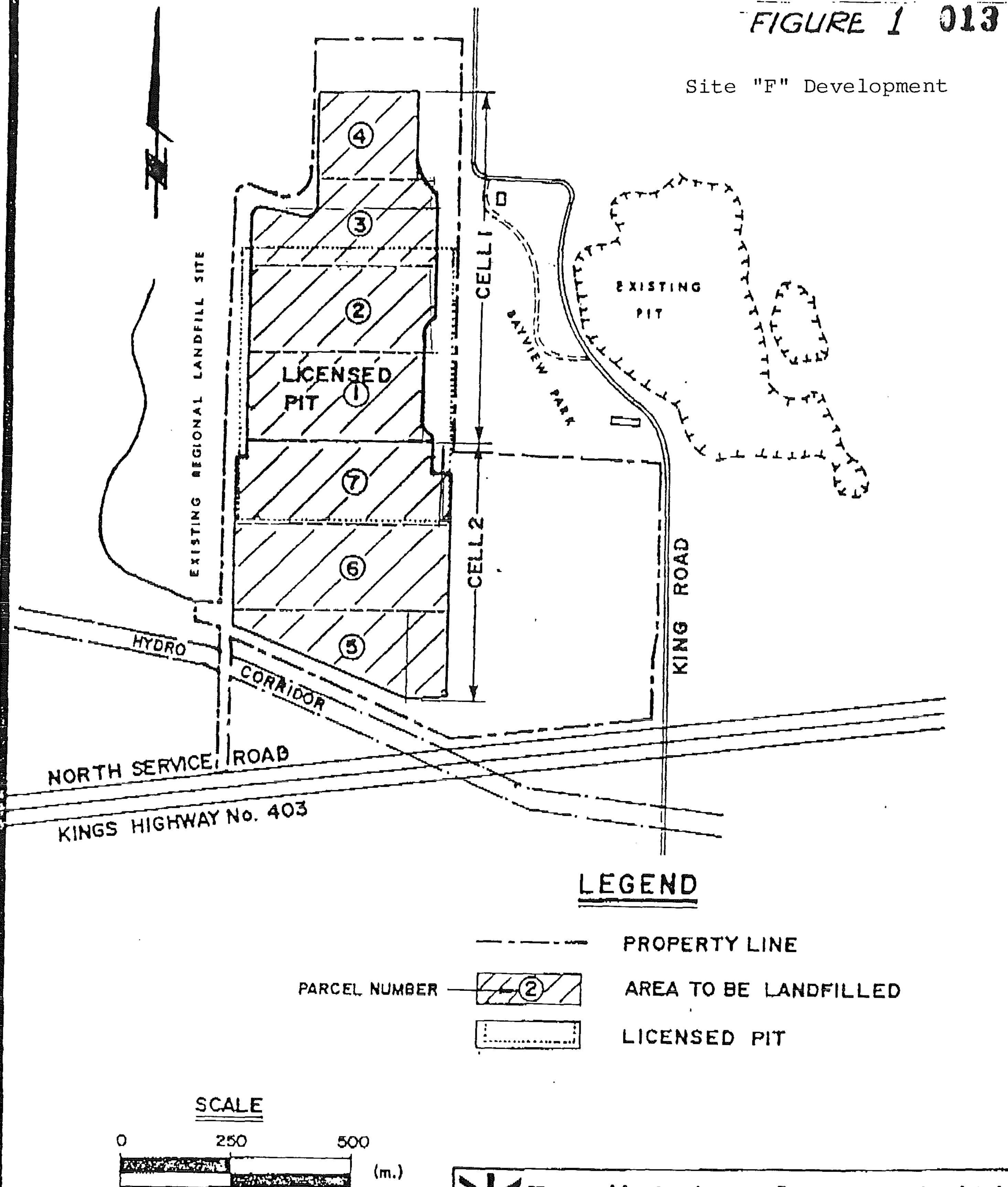
HC - Trow Hydrology Consults Ltd.
MB - Morrison Beatty

* Logs of Exploratory Boreholes and Water Supply Wells contained in Ex. H-40A, Site F: Appendices to Hydrogeological Report (Trow, Sept., 1986)

** Proposed base elevation is 1 m lower than Trow's design base elevation

FIGURE 1 **013**

Site "F" Development



Trow Hydrology Consultants Ltd.

SITE F

OPTION No. 3

Proj. No.	T 6600-HY
Scale	AS SHOWN
Drawn by	D.K
Appr. by	J.J.O
Revised	
Date	APRIL, 1987

FIGURE 3-r

Figure 1 Commentary
"Site 'F' Development"

(a) Commence cell 1 construction by stripping topsoil and excavating shale proceeding in south to north direction (parcels 1 to 4).

(b) Haul clay fill borrow from parcels 3 and 4, spread and compact to form berms and dykes.

(c) Haul, spread and compact cut fill material (shale and/or clay) at all infill areas (parcels 1, 2, and 3).

(d) Haul clay fill borrow from parcels 3 and 4, spread and compact to form liner, initially parcels 1 and 2.

(e) Stockpile of surplus cut material only in later stage of cell 1 construction. Deficit of available materials in parcels 1 and 2 to meet requirements.

(f) Proceed with clay liner construction in parcels 3 and 4 and placement of daily and final cover as landfill development progress.

(g) Commence cell 2 construction by stripping topsoil and excavating shale proceeding in south to north direction (parcels 5 to 7).

(h) Haul clay fill borrow from parcel 5, spread and compact to form berms and dykes.

(i) Haul, spread and compact cut fill material (shale and/or clay) at all infill areas (parcels 5, 6, and 7).

(j) Haul, spread and compact clay fill borrow in parcels 5 and 6 to form liner.

(k) Stockpile of surplus cut material only in later stage of cell 2 construction.

(l) Proceed with clay liner construction in parcel 7 and placement of daily and final cover as landfill development progress.

PARTICLE SIZE DISTRIBUTION CHART

OUR PROJECT NO. 87 F 374

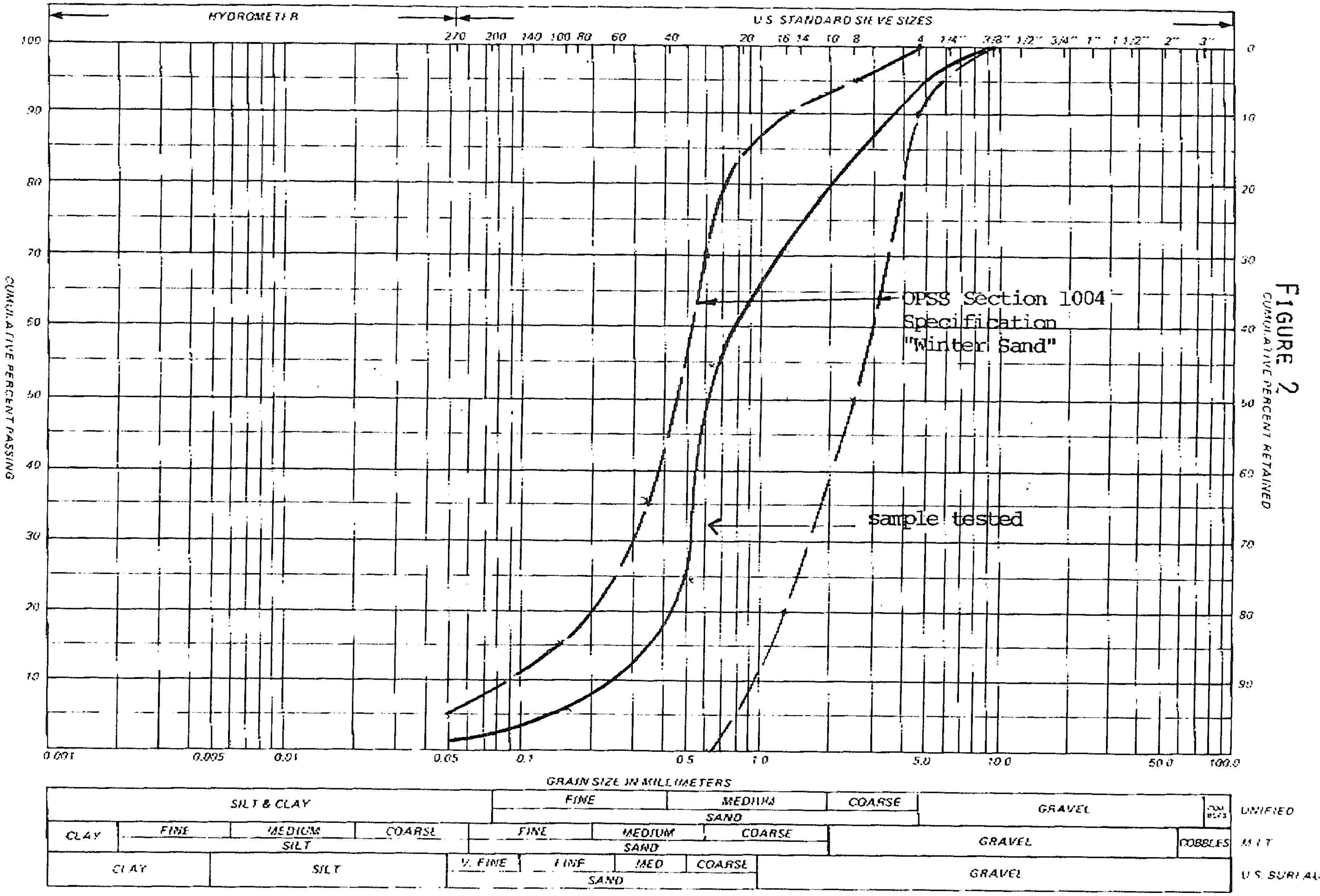


FIGURE 2

REMARKS Sample L 702 - SHERMAN SAND & GRAVEL

WINTER SAND

MOISTURE DENSITY RELATIONSHIP TEST REPORT

CLIENT TOWN OF MILTON

PROJECT SITE 'F' BURLINGTON

OUR PROJECT NO. 88 F 374

REPORT NO.

ENCLOSURE

FIGURE 3

SAMPLE TYPE WINTER SAND

SAMPLED FROM –

SAMPLE NO. L 702

DATE SAMPLED Aug 4/88

SAMPLED BY PB OF

DATE RECEIVED Aug 3/88

MOISTURE CONTENT AS RECEIVED 4.0%

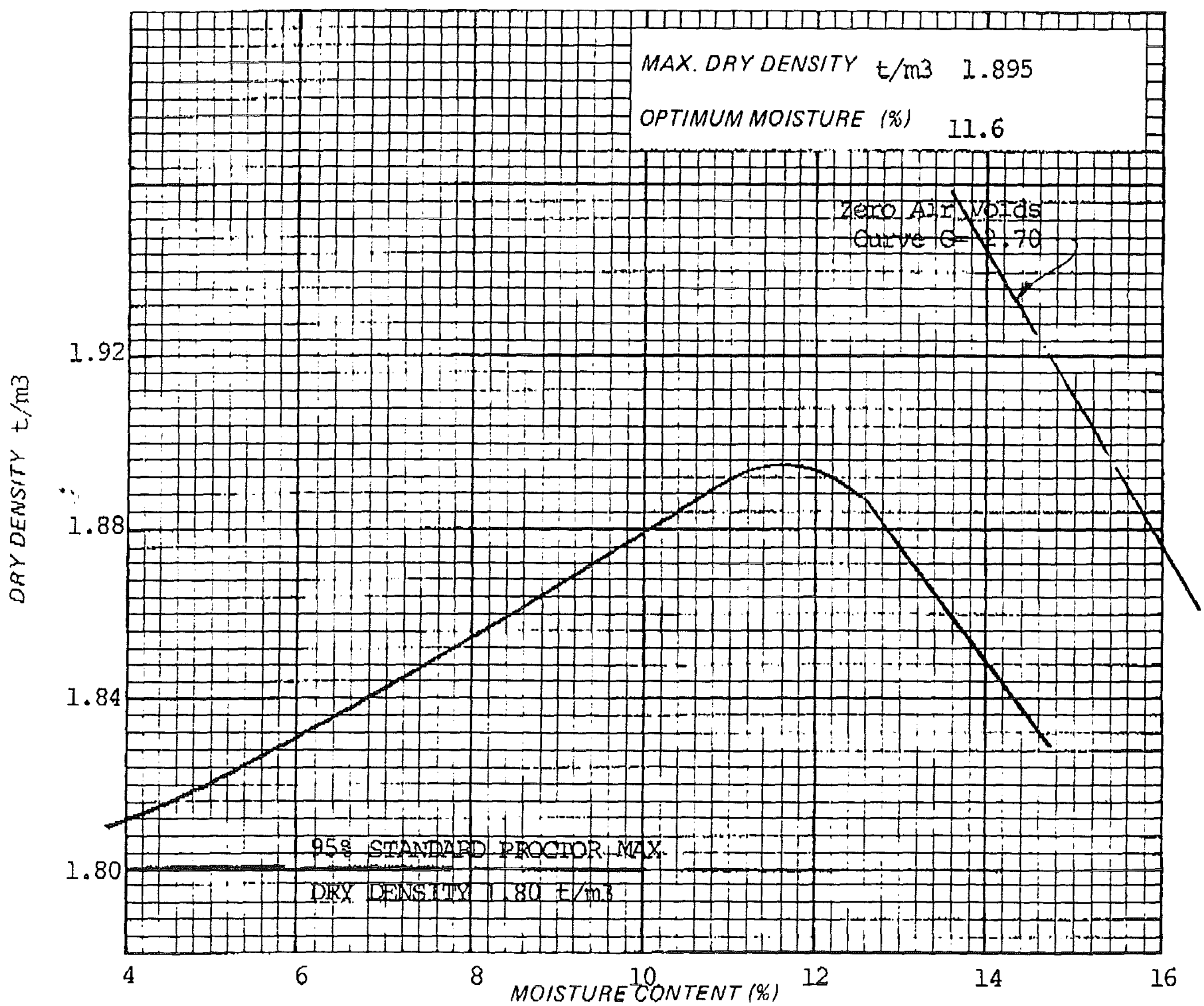
REMARKS

STANDARD PROCTOR – ASTM D698-70

METHOD – A ☐ B ☐ C ☒ D ☐

MODIFIED PROCTOR – ASTM D1557-70

METHOD – A ☐ B ☐ C ☐ D ☐



PML/122

APPENDIX "A"

BRIAN R. GRAY, P.Eng.
B.A.Sc., M.Eng.

VICE PRESIDENT, GEOTECHNICAL ENGINEERING

BACKGROUND

1968	B.A. Sc. Civil Engineering University of Toronto
1968	Soils Laboratory Engineer Ontario Hydro
1971	M.Eng. Geotechnical Engineering University of Toronto
1970-75	Project Engineer/Assistant Branch Manager-Windsor/Senior Geotechnical Engineer, Golder Associates, Toronto
1975 to Present	Regional Geotechnical Engineer/Managing Director, Geotechnical Engineer/Vice President, Geotechnical Engineering, Peto MacCallum Ltd., Toronto

EXPERIENCE

Mr. Gray has specialized in geotechnical and materials engineering relating to soil mechanics, foundation and earthwork construction. He has executed numerous assignments involving landslide and slope stability studies; subgrade and pavement evaluations and design; Benkelman beam studies; and soil stabilization programs.

Mr. Gray is Vice President, Geotechnical Engineering. He is responsible for the overall direction and supervision of the company's geotechnical engineering activities, including professional and technical staff, administration, business development and client liaison. The following is a resume of his career.

In the summer of 1968, Mr. Gray was employed by Ontario Hydro as a Soils Laboratory Engineer. He carried out routine soil classification; performed laboratory tests; and prepared foundation reports for several transmission towers, as well as generating, transformer, and switching stations, including Mississagi TS and Well GS. He prepared an instrumentation report for the research program on the uplift capacity of augered caissons at the Nanticoke Generating Station site, Lake Erie.

Mr. Gray joined Golder Associates in 1970 as a Project Engineer. He was involved with several mining projects, including:

- Fraser River dyke study
- Tailings Dam, Elliot Lake, Ontario
- Copper/lead/zinc mine, Ignace, Ontario
- Coal mine, Elk River, British Columbia.

He was also engaged as Resident Geotechnical Engineer for foundation and construction inspection at the Mattabi Mine site.

BRIAN R. GRAY, P.Eng.
B.A.Sc., M.Eng.

VICE PRESIDENT, GEOTECHNICAL ENGINEERING

EXPERIENCE (Continued)

From 1972 to 1974, Mr. Gray served as Project Engineer and Assistant Branch Manager of the Windsor office of Golder Associates. He was responsible for site investigations and the preparation of geotechnical reports for commercial, industrial, residential, and highrise building projects. He also devised shore protection measures and undertook geotechnical studies for the following projects:

- Sewage Works, Windsor
- Pumping Stations, Collector Sewers, and Outfall Sewers in Lake St. Clair
- Two Mile Long, 60 inch diameter trunk sanitary sewer requiring soft ground tunnelling with compressed air.

As Senior Geotechnical Engineer for Golder Associates from 1974 to 1975, Mr. Gray supervised and prepared geotechnical reports for a variety of major projects, such as:

- 1,200 m bulk materials handling wharf at Stelco, Nanticoke, Ontario
- Thirty bridge sites for a 102 km all-weather highway in the Sultanate of Oman and Emirate of Abu Dhabi (U.A.E.)
- Evaluation of local rock for rip rap sources in the Sultanate of Oman
- 550 kV transmission line from Nanticoke Generating Station to proposed Stelco Steel Mill on Lake Erie
- Conceptual layout and design of Tailings Dam Retention Scheme for a northern Ontario mining company

In 1975, Mr. Gray joined Peto MacCallum Ltd. as Regional Geotechnical Engineer. He was responsible for all geotechnical engineering operations and business development throughout southwestern Ontario for the Hamilton and Kitchener branch offices.

His areas of responsibility expanded to include Toronto and eastern Ontario when he was made Manager, Geotechnical Engineering in 1978. During this period Mr. Gray directed the successful completion of many major assignments, including:

- Study of earth dyke improvements extending 17 miles, Holland Marsh, Bradford, Ontario
- Lloyd D. Jackson Square, Phase II, Hamilton
- Slope stability investigations Weir Crescent, Oshawa and Hyland Creek, Scarborough
- Plant Additions for DeHavilland Aircraft, Downsview

Mr. Gray was appointed a Managing Director of Peto MacCallum Ltd. in 1979 and Vice President, Geotechnical Engineering in 1986. He has provided the overall direction and supervision of the geotechnical engineering aspects for the following representative assignments:

BRIAN R. GRAY, P.Eng.
B.A.Sc., M.Eng.

VICE PRESIDENT, GEOTECHNICAL ENGINEERING

EXPERIENCE (Continued)

Airports

- New Terminal 3 Development, Lester B. Pearson International Airport, Toronto
- B720 Hangar St. Hubert Airport, Quebec
- Pavement Evaluation Toronto Buttonville Airport, Markham, Ontario
- New 767 Flight Simulator DeHavilland Aircraft, Downsview
- New Municipal Airports/Upgrades Townships of Stanhope, Manitouwadge, and Marathon, Ontario

Automotive

- Numerous General Motors of Canada Ltd. Plant additions, Oshawa
- New GM-Suzuki Plant, Ingersoll
- New Hyundai Plant, Markham
- Ford Motor Company of Canada Ltd. Plant Expansions, Oakville and St. Thomas
- New F&P Mfg. Inc. Plant, Tottenham

Commercial

Additions to Shopping Malls:

- Georgian Mall, Barrie
- Place Vertu, Montreal
- Quinte Mall, Belleville
- Penn Centre, St. Catharines
- Yorkdale, Toronto

Government

- North York Civic Centre and Library
- Mississauga Civic Centre
- Town of Markham Civic Centre
- Town of Halton Hills Municipal Office

Highrises

- Renaissance Place, King's Landing and Harbour Terrace, Toronto
- Somerset Place, Eagle Ridge
- Granite Gates, Mississauga
- Concorde Place, 5000 Yonge St., Toronto

Industrial

- Terrain evaluation for a 117 km natural gas pipeline route, Niagara Loop (Milton to Niagara Falls)
- A $440M Ammonia II Plant for the C-I-L Inc. Cryogenic Plant, Courtright

Industrial/Residential Subdivisions

- Elgin West Community, Town of Richmond Hill
- Olde English Lane, Credit Pointe Village, Creditview Heights, Erin Mills Neighborhood, Mississauga
- Meadowvale Business Park
- Tudor Valley Business Park, Vaughan
- Glen Abbey, Oakville

Institutional

- New Credit Valley Hospital and Parking Garage, Mississauga
- New St. Joseph's Hospital, Guelph
- New YMCA Building, Mississauga

BRIAN R. GRAY, P.Eng.
B.A.Sc., M.Eng.

VICE PRESIDENT, GEOTECHNICAL ENGINEERING

EXPERIENCE (Continued)

Landfill Projects

- Keele Valley, Maple
- City of Niagara
- Britannia Road landfill, Mississauga
- Harold and Grace Baker Centre, Northwestern Hospital
- North York Hydro Office, Toryork Drive, North York

Municipal

- York-Durham Sewer, Oak Ridges Section, involving 3,500 m of 1.7 m diameter sewer pipe in tunnel construction as deep as 30 m below grade
- Trunk sewer involving 32 m vertical and 3 m diameter shaft with extensive in-situ instrumentation to monitor rock stress for tunnel under Sixteen Mile Creek, Oakville, Ontario

Sewer Tunnels
- Molson Park Drive, Barrie
- Scarlett Road, Toronto

Slope Stability Studies

- Denison Avenue (Humber River), Weston
- Mavis Road (Wolfedale Creek), Mississauga
- Armstrong Avenue, Irwin and Dawson Crescents (Credit River and Silver Creek)
- Town of Halton Hills
- Alder Road (Massey Creek) Borough of East York

Transportation

- Burlington Street Reconstruction, elevated expressway extending one mile through the industrial centre of Hamilton
- Park Road Grade Separation, Oshawa

Waterfront Development

- City of Orillia
- Toronto Island
- Hamilton Harbour
- Kenora

AFFILIATIONS

- Association of Professional Engineers of Ontario, designated Consulting Engineer
- Canadian Geotechnical Society

APPENDIX "B"

The University of Western Ontario
GEOTECHNICAL RESEARCH CENTRE

Faculty of Engineering Science
London, Ontario, Canada N6A 5B9

Dr. K.Y. Lo (519) 661-2125
Dr. M. Novak 661-2003
Dr. R.K. Rowe 661-2126
Dr. R.M. Quigley 661-3344

Telex No. 064-7134

1988 06 22

Geocon Inc.
3210 American Drive
Mississauga, Ontario
L4V 1B3

Attention: Ms. Janet Haynes

Dear Sirs:

Re: Hydraulic conductivity testing on compacted Halton Till and compacted weathered Queenston Shale

Hydraulic conductivity tests have been carried out on two compacted soil samples. It is understood that the test data are to be used to assist in forming an opinion of their suitability for use as barrier materials for retardation of domestic waste leachate migration. The two samples were identified as follows:

1. A bag sample of brown clayey silt till (identified as Halton Till and received on March 28, 1988).

2. A bag of air dry, brown, broken and weathered shale (identified as Queenston Shale and received on March 31, 1988).

Two types of tests were required on each sample, as follows:

1. A fixed ring test with a static vertical effective stress, σ'_v, of about 100 kPa on the samples; and

2. A flexible membrane (triaxial) test with an all round effective stress, σ'_3, of about 100 kPa.

For both sets of tests the hydraulic gradient, i, was to be as low as practical and preferably about 20.

Each sample was also subjected to a preliminary mineralogical assessment and to a Harvard Miniature compaction test for comparison to previous Proctor tests by others. The compaction tests were required to establish suitable compaction procedures for the hydraulic conductivity test specimens.

1.0 Description of Samples

The Halton till arrived in bags labelled T.P. 1 and consisted of brown clayey silt with a weak to modest dry strength. The sample was identified as a combined till sample from Site F.

The Queenston shale arrived as a well graded air dry mixture of weathered, broken, medium reddish brown shale fragments varying in size from 5 cm to fine silt sized soil.

Both samples were highly reactive to dilute HCl indicating abundant carbonate.

Detailed descriptions are presented in Table 1.

2.0 Compaction Tests

The Harvard Miniature compaction test results for both the Halton till and the Queenston shale are presented on Figure 1. Since the mold for these tests is only 62.35 cm^3 in size (33.36 mm diam.), stones larger than 2 mm were removed from the Halton Till sample and the Queenston Shale was crushed by hand with a rubber tipped pestle to sizes less than 2 mm. Distilled water was added to both samples to bring them to the required molding water content. The testing protocol is presented in Appendix I of this report.

The two curves presented on Figure 1 present Harvard Miniature compaction data similar to Proctor results (by others) which are summarized in

Table 2. It was therefore concluded that the hydraulic conductivity test specimens would be compacted up to 2% wet of "Harvard" optimum using the Harvard Miniature compaction rod to simulate kneading compaction in the field. According to the saturation lines on these plots, the saturation state should be about 95% for these compaction conditions using the value of specific gravity measured on each sample.

The data points for the compacted specimens used for the hydraulic conductivity testing are also plotted on Figure 1 as Points #1 to #3. Point #3 for the two Halton Till specimens plots at its expected location whereas the two Queenston Shale specimens (Points #1 and #2) are slightly more dense than anticipated. Slight variations in composition, gradation, specific gravity, etc. associated with crushing, mixing and sampling make such scatter inevitable. All four samples, however, were quite satisfactory for k testing.

3.0 Hydraulic Conductivity Tests

3.1 Fixed Ring Method

The fixed ring hydraulic conductivity tests were run on 2 cm thick specimens compacted into 54 mm diameter permeameter cells using the same rod and the same energy per mass of soil used for the Harvard Miniature compaction tests. A static effective stress of 100 kPa was applied to the porous stone on top of the sample by means of a spring-loaded cell cap. The permeant, which consisted of ordinary tap water, was forced through the specimens using computer-controlled, constant flow rate equipment developed at Western. Two flow rates were employed, one producing a gradient of 30 to 50 and the other producing a gradient of 400 to 600 depending on the properties of the sample. Flow was maintained until the continuously measured k was deemed constant. Further details are presented in Appendix I.

3.2 Flexible Membrane (Triaxial) Method

The specimens for triaxial hydraulic conductivity testing were compacted into a 50 mm diameter mold using the same methods described for the fixed ring tests. The samples were removed from the mold, fitted with a greased double membrane and installed in a triaxial cell for saturation at a back pressure of 50 kPa, followed by isotropic consolidation to $\sigma_3' = 100$ kPa. Once the rate of consolidation had reduced to a negligible value, the inlet pressure at the top end of the triaxial specimen was increased to generate flow from the top to the bottom of the sample. Both the influent and effluent flow volumes were measured and the test continued until the two flow rates were as similar as practical. A low gradient of ~ 20 was employed for both tests. Further details of the test procedure are presented in Appendix I.

3.3 Halton Till Results

A plot of hydraulic conductivity, **k**, versus pore volumes of fluid passed through the sample is presented on Figure 2 for the Halton Till compacted at $\omega_o = 19.4\%$ (data point #3 on Figure 1). At a very slow flow rate of 6.0×10^{-6} mL/s, a stable average **k** of 5.2×10^{-9} cm/s was obtained after 8 days of flow corresponding to a gradient of 50 and a max seepage drag at the bottom of the sample (J_{max}) of 10 kPa. The long slow drop in **k** is a feature of this equipment and represents a combination of system compliance and loss of influent into air space in the compacted sample (i.e. increased saturation from its original calculated S_o value of ~ 96% shown in Table 3).

Increasing the flow rate to 89×10^{-6} mL/s produced a final **k** of 6.9×10^{-9} cm/s with a corresponding gradient of ~ 560 and J_{max} of ~ 110 kPa. The $\mathbf{k}_f$ is actually higher at the larger gradient than it was at i = 50. This probably is a result of gas compression under the influence of the higher fluid pressure which compensated for the 0.125 mm of sample consolidation caused by the seepage drag (Figure 2b). This phenomenon is somewhat similar to the increased saturation produced by the back pressures applied in the triaxial tests.

Plots of cumulative influent and effluent flow versus time for the triaxial **k** test on the Halton Till are presented on Figure 3. The **k** values calculated for each final flow rate are $\mathbf{k}_{influent} = 9.5 \times 10^{-9}$ cm/s and $k_{effluent}$ = 7.08 x 10^{-9} cm/s. The two flow curves are not quite parallel and therefore the test had not quite reached equilibrium even though it had run for 11 days when terminated. The fact that both curves seem to be changing slope towards parallelism suggests a final equilibrium k of ~ 8 x 10^{-9} cm/s.

The high gradient fixed ring k value of 7 x 10^{-9} cm/s is quite similar to the averaged triaxial value of 8 x 10^{-9} cm/s.

3.0 Queenston Shale Results

A plot of hydraulic conductivity, **k**, versus pore volumes of fluid passed through the sample is presented on Figure 4 for the Queenston Shale compacted at ω_o = 14.9% (data point #1 on Figure 1). At the slow flow rate of 6 x 10^{-6} mL/s, a k of 7 to 8.5 x 10^{-9} cm/s was obtained after 8 days of flow at a final gradient of ~ 30 and a J_{max} of 6 kPa.

Increasing the flow rate to 89 x 10^{-6} mL/s produced a slight increase in the k to a final value of 9.6 x 10^{-9} cm/s at a corresponding gradient of 405. As in the fixed ring test on the Halton till, this increase in k is attributed to greater saturation of the samples under the influence of the higher influent pressure. This was in spite of the small decrease in void ratio indicated by the 0.087 mm of settlement caused by drag forces during seepage which amounted to 80 kPa at the base of the sample.

Plots of cumulative influent and effluent flows versus time for the triaxial test performed on compacted Queenston shale are presented on Figure 5. The k values calculated for each final flow rate are $k_{influent}$ = 1.2 x 10^{-8} cm/s and $k_{effluent}$ = 9.6 x 10^{-9} cm/s. If the two k values are averaged as was done for the Halton Till a suggested equilibrium value of k = 1.06 x 10^{-8} cm/s is calculated. This is very similar to the high gradient value of 0.96 x 10^{-8} cm/s obtained by fixed ring testing.

4.0 Mineralogical Assessment

The preliminary mineralogical assessment consisted of the following tests:

i) x-ray powder diffraction on pulverized, air dry whole soil or whole rock samples;

ii) dispersion in very dilue NaOH solutions and x-ray diffraction of the oriented fines, water-wet, air dried, glycerol treated and heated to 550°C for 30 min;

iii) same as item (ii) except K^+ saturated;

iv) gasometric analyses for total carbonate content using the Chittick apparatus.

4.1 Halton Till

The powder trace (Figure 6) used in combination with the carbonate (Chittick) analyses indicate the following very approximate composition:

Calcite and dolomite	~ 33%
Quartz	~ 14%
Feldspars	~ 23%
Clay minerals	~ 30%

The oriented traces (Figure 7a) indicate that the clay minerals consist of abundant illite (1.0 nm peaks) plus chlorite which has been extensively weathered to a swelling clay (1.4 - 1.5 nm) so that no distinct 1.4 nm peak is obtained by oven drying at 550°C. The ragged nature of the illite peak and its presence at 1.01 nm rather than 1.00 nm suggests some interlayering with a swelling phase.

Potassium saturation of the fines (Figure 7b) produces some reduction in the size of the 1.52 nm smectite peak and a shift to 1.42 nm but does not collapse it in the water-wet state. Air drying, however, causes significant collapse indicating that the swelling clay is borderline in charge between vermiculite and smectite. This soil is of similar mineralogy to other soils in Ontario presently believed to be performing satisfactorily as clay barriers against domestic waste leachate migration.

4.2 Weathered Queenston Shale

The powder trace (Figure 6) used in combination with the carbonate (Chittick) analyses suggest the following very approximate composition.

Calcite and dolomite	~ 22%
Quartz	~ 29%
Feldspars	Nil
Clay minerals	45-55%

The oriented traces (Figure 8a) indicate that the clay minerals consist of abundant illite, moderate chlorite and a trace amount of swelling clay. The chlorite in the shale produces a distinct 1.40 nm peak on heating to 550°C unlike the weathered Halton till.

Potassium saturation of the shale fines (Figure 8b) has little effect on the traces producing no major collapse of any peaks although the slight amount of smectite shown by the traces on Figure 8a seems less evident. The mineralogy of the shale suggests that it is a potentially suitable liner material provided it is adequately comminuted in the field either by the compaction process or by crushing and wetting before compaction.

5.0 Concluding Remarks

I trust that this preliminary assessment of these two potentially suitable liner materials is adequate for your present purposes.

Yours truly,
GEOTECHNICAL RESEARCH CENTRE

R.M. Quigley

R.M. Quigley, P.Eng.
Professor

RMQ:em

Attachments: Tables 1 to 4
Figures 1 to 8
Appendix 1

Table 1. Description of Test Samples

Sample	Description
Halton Till	Medium brown, air dry clayey silt till having a modest dry strength. Yellowish to reddish colour on surface of some chunks, freshly broken surfaces appear mottled. Some fracture surfaces coated with a whitish material and others coated with a brown stain. All particles highly reactive to dilute HCl. Carbonate ≈ 33% on two specimens. As received water content ≈ 13.4%. Specific gravity, D_R = 2.75 (2.764, 2.742).
Queenston Shale	Well graded air dry mixture of broken, weathered, medium reddish brown shale containing scattered lumps of dry soil up to 2 cm in diameter. Shale fragments vary from silt sizes to tabular chunks up to 5 cm in diameter and 1 cm thick. All particles coated with a more yellowish looking silt. Freshly broken fragments a darker, medium brown shale. Shale fragments moderately to highly reactive to dilute HCl. Carbonate ≈ 19% and 25.5% on two specimens. As received water content 6 to 8.5%. Specific gravity, D_R = 2.74 (two measurements).

Table 2. Compaction Data

Sample	γ_d(max) (pcf)	ω_{opt} (%)	Type Compaction	Source
<u>Halton Till</u>				
Till (Site F)	111.1	16.7	Std. Proctor	J. Haynes (phone - Mar. 31, 1988)
Combined till (Site F)	114.1	17.25	Harvard* Miniature	U.W.O. (this report)
<u>Queenston Shale</u>				
Weathered shale (Site F)	118.6	13.4	Std. Proctor	Geocon (Mar. 1988)
Weathered shale	124.0	12.8	Std. Proctor	Golder report to Morrison Beatty (June 1980)
Weathered shale	119.3	14.25	Harvard Miniature	U.W.O. (this report)

*Mold height = 71.38 mm, diam = 33.36 mm, volume = 62.35 cm³; soil compacted in 5 layers, 20 tamps/layer, using an 0.142 kN (32 lb) spring; contact pressure of 12.5 mm diam. rod = 1160 kPa.

Table 3. Geotechnical Characteristics of Compacted Soil/Shale Samples before Hydraulic Conductivity Testing

Soil Characteristic	Halton Till		Queenston Shale	
	Fixed Wall	Triaxial	Fixed Wall	Triaxial
Moisture content ω_o (%)	19.4	19.4	14.9	15.5
Void Ratio e_o	0.557	0.556	0.418	0.428
Dry density ρ_{do} (Mg/m^3)	1.767	1.767	1.932	1.919
Saturation S_o (%)	~ 96	~ 96	~ 98	~ 99
Sample volume V_o (mL)	45.8	196.4	45.8	117.2
Pore Volume PV_o (mL)	16.4	70.2	13.5	35.1

Table 4. Geotechnical Characteristics of Compacted Soil/Shale Samples during and after Hydraulic Conductivity Testing

Soil Characteristic	Halton Till		Queenston Shale	
	Fixed Wall	Triaxial	Fixed Wall	Triaxial
Applied static effective stress (kPa)	$\sigma'_v = 100$	$\sigma'_{3C} = 100$	$\sigma'_v = 100$	$\sigma'_{3C} = 100$
Volume change due to(1) static stresses (mL)	- 0.786	- 4.29	- 0.426	- 3.25
Volume change due to seepage (mL)	- 0.286	-	- 0.199	-
Final average moisture content ω_f (%)	18.2	18.4	14.3	15.4
Final void ratio e_f	0.520	0.529	0.399	0.425
Final dry density $\rho_{d(f)}$ (Mg/m³)	1.809	1.800	1.959	1.922
Final Saturation S_f (%)	~ 96	~ 96	~ 98	~ 99

(1) Directly measured in fixed wall tests; inferred from water of consolidation in triaxial tests.

032

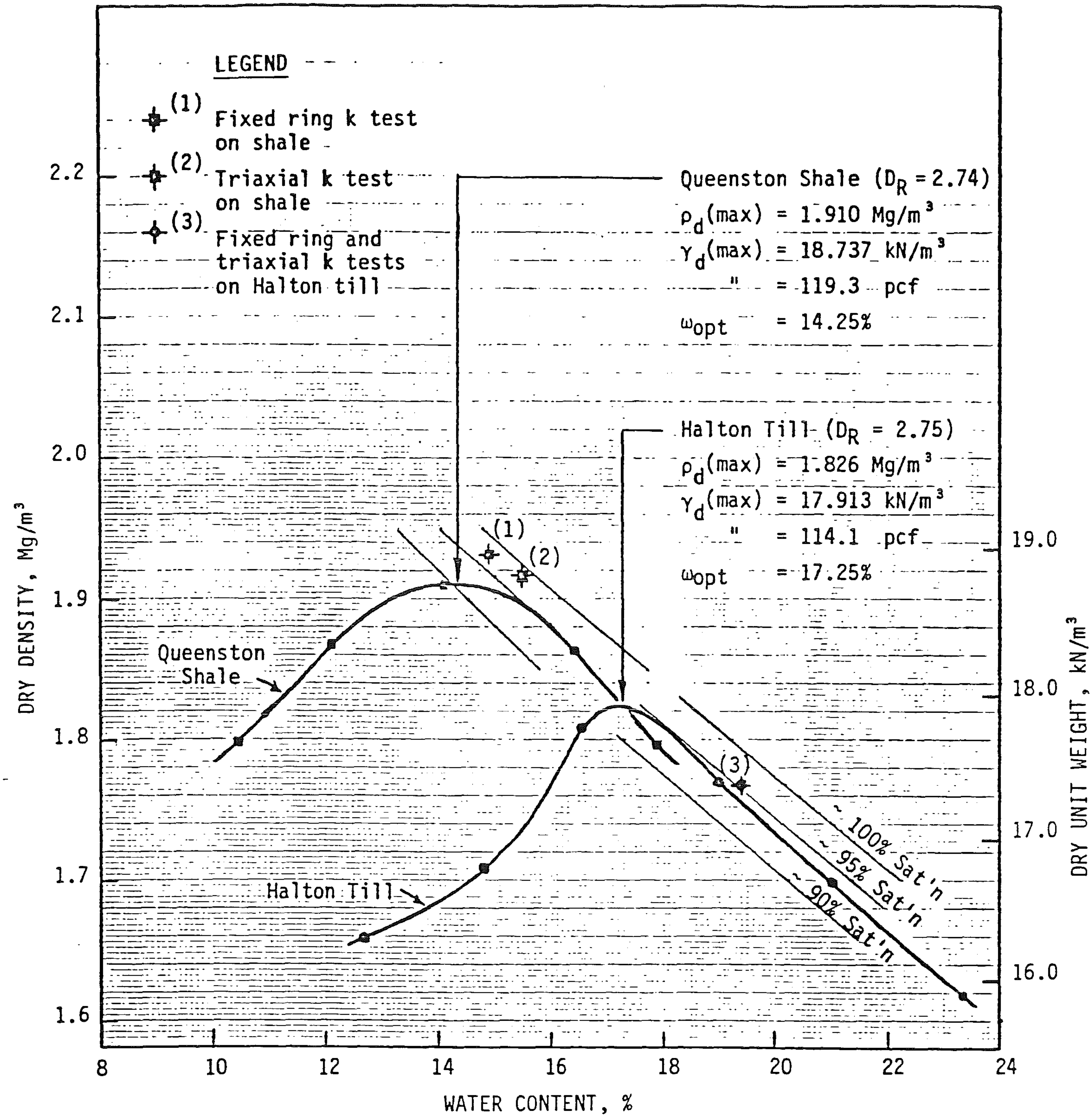


FIGURE 1. HARVARD MINIATURE COMPACTION CURVES FOR HALTON TILL AND QUEENSTON SHALE TEST SAMPLES

033

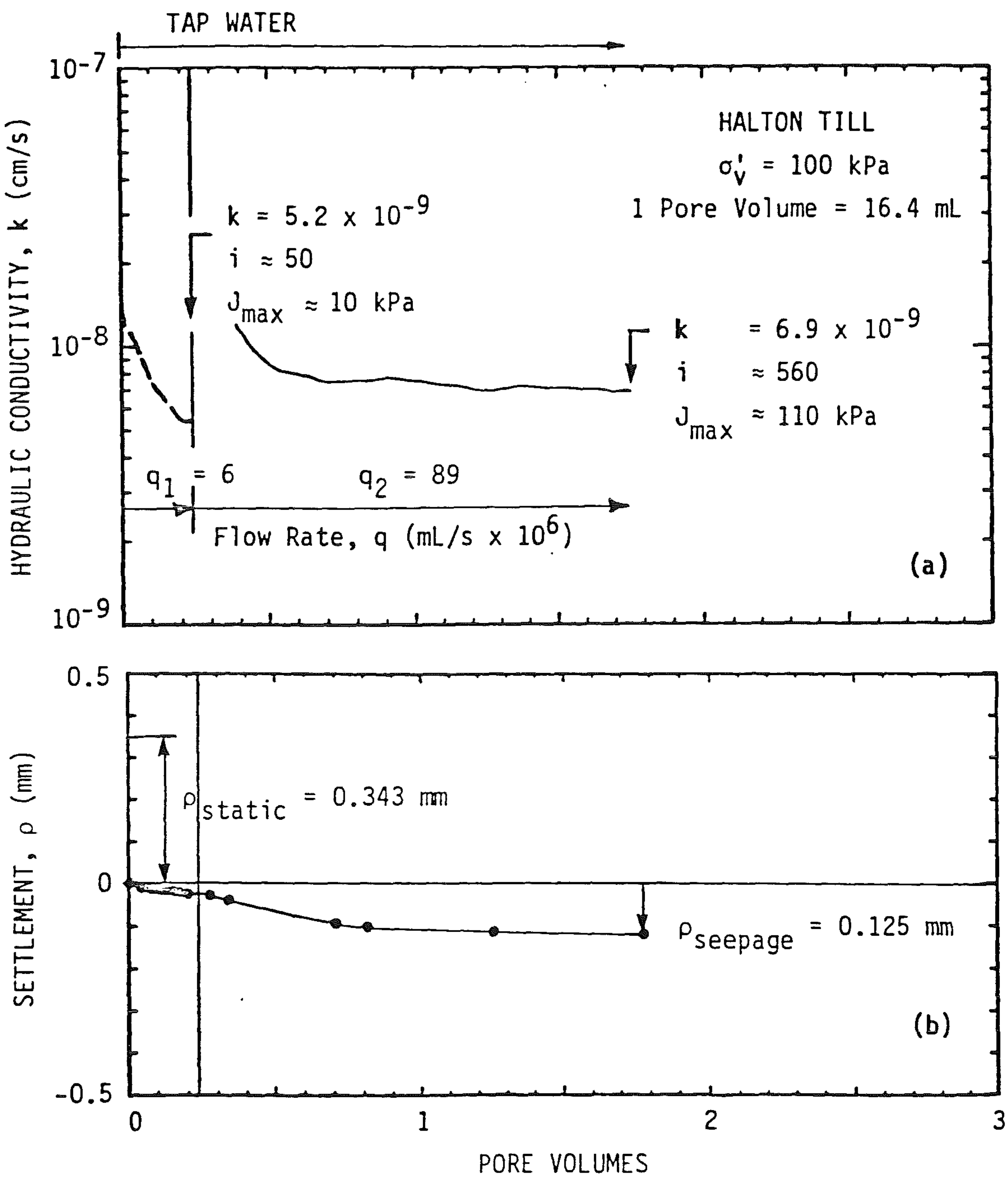


FIGURE 2. FIXED-RING, CONSTANT FLOW RATE, HYDRAULIC CONDUCTIVITY TEST RESULTS ON HALTON TILL:
(a) k VS PORE VOLUMES PASSED THROUGH;
(b) SETTLEMENT VS PORE VOLUMES DURING TESTING

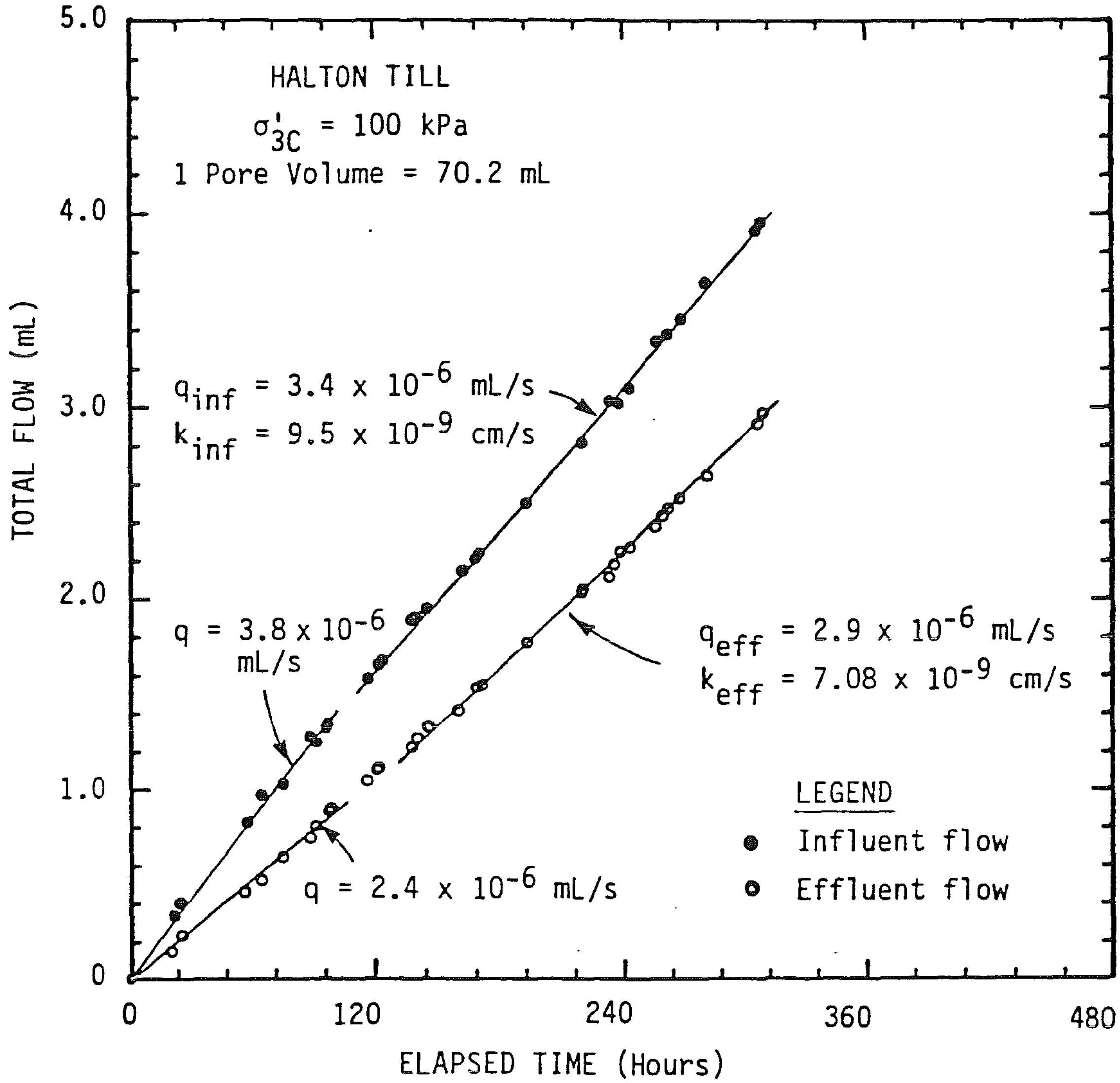


FIGURE 3. INFLUENT AND EFFLUENT FLOW VS TIME FOR COMPACTED HALTON TILL PERMEATED WITH DISTILLED WATER IN A TRIAXIAL CELL (FLEXIBLE WALL TEST, i ≈ 20)

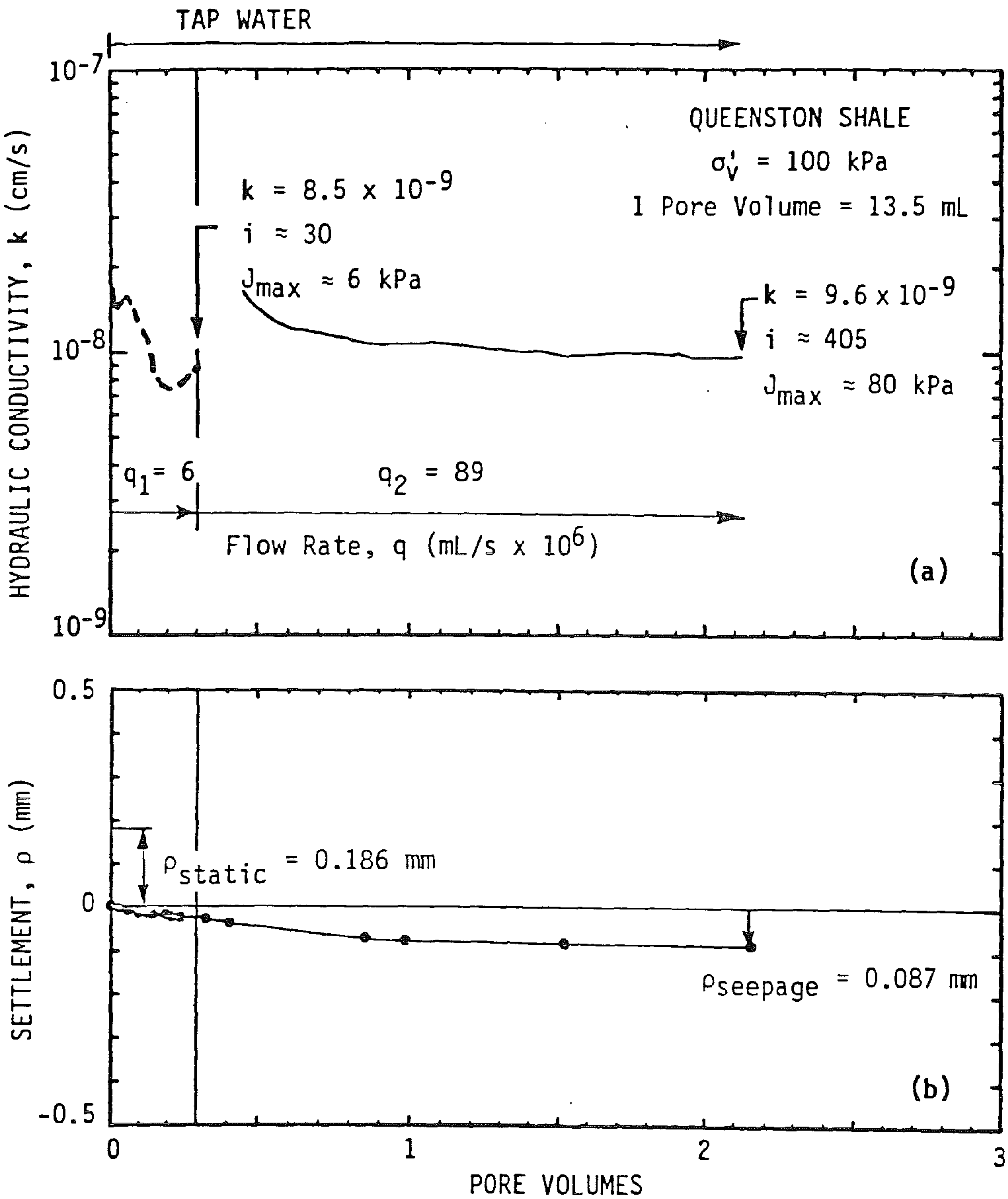


FIGURE 4. FIXED RING, CONSTANT FLOW RATE, HYDRAULIC CONDUCTIVITY TEST RESULTS ON QUEENSTON SHALE
(a) k VS PORE VOLUMES PASSED THROUGH
(b) SETTLEMENT VS PORE VOLUMES DURING TESTING

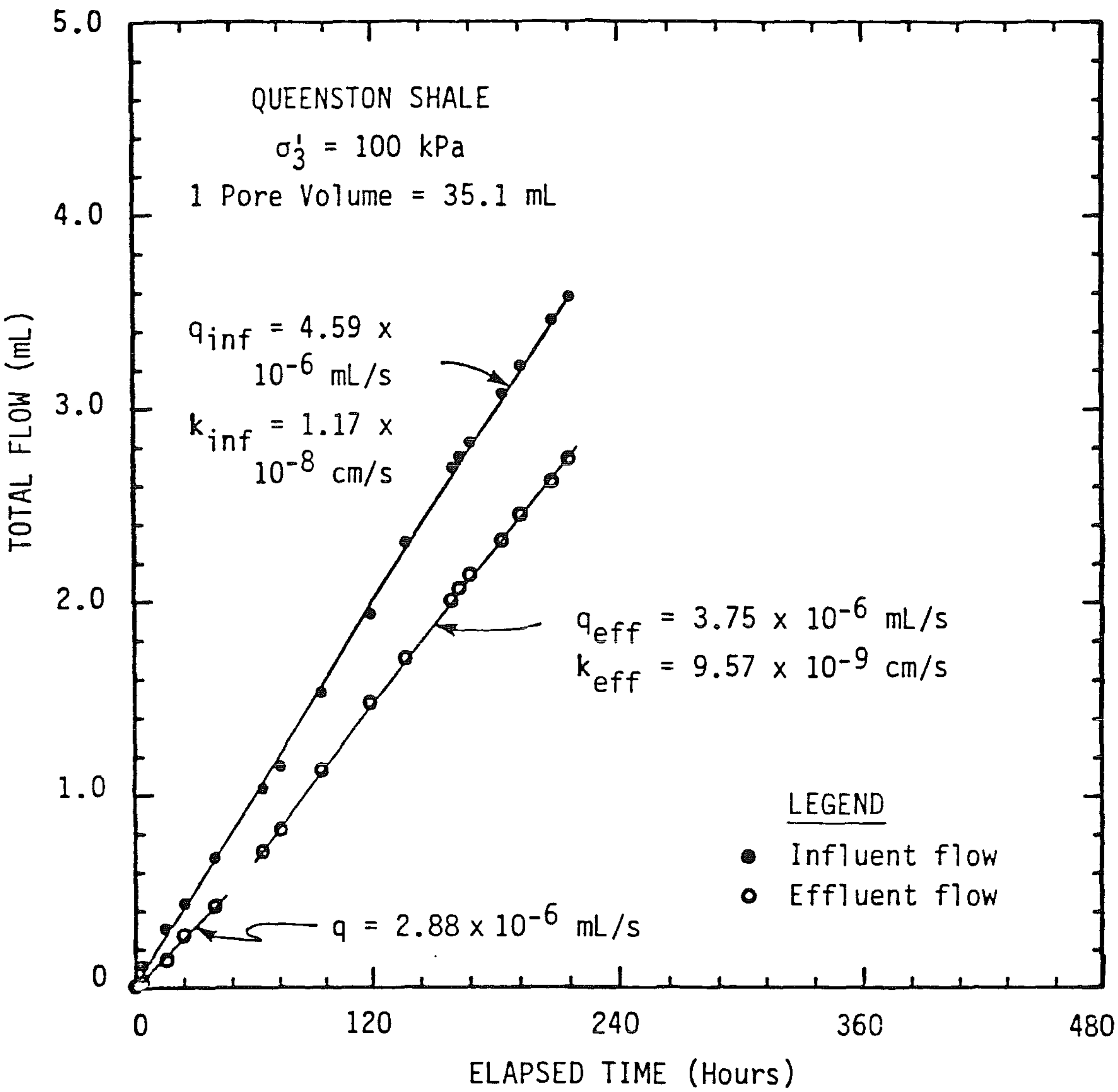


FIGURE 5. INFLUENT AND EFFLUENT FLOW VS TIME FOR COMPACTED QUEENSTON SHALE PERMEATED WITH DISTILLED WATER IN A TRIAXIAL CELL (FLEXIBLE WALL TEST, i ≈ 20)

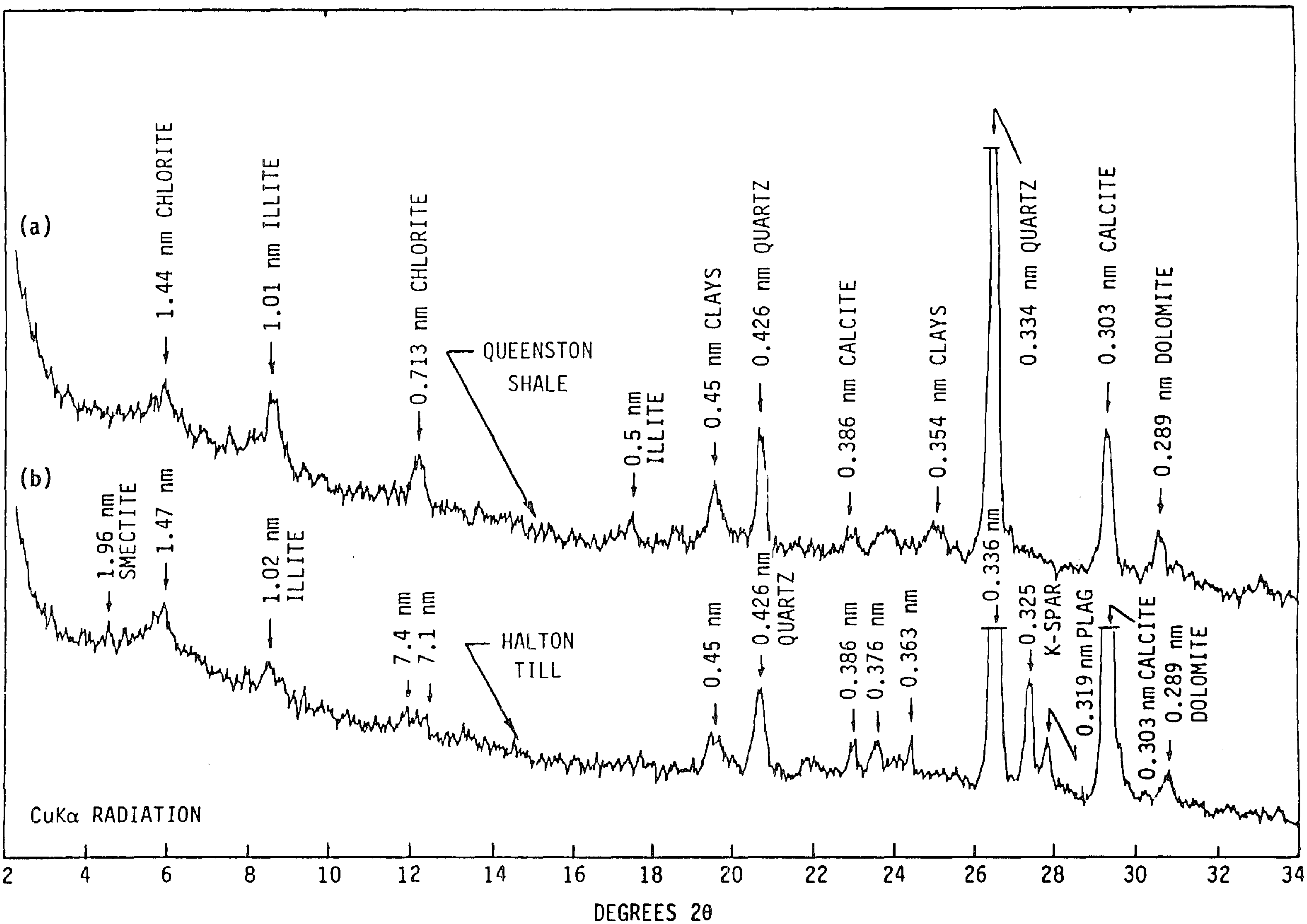


FIGURE 6. X-RAY POWDER PATTERNS OF PULVERIZED WHOLE SOIL SAMPLES OF HALTON TILL AND QUEENSTON SHALE TEST SAMPLES

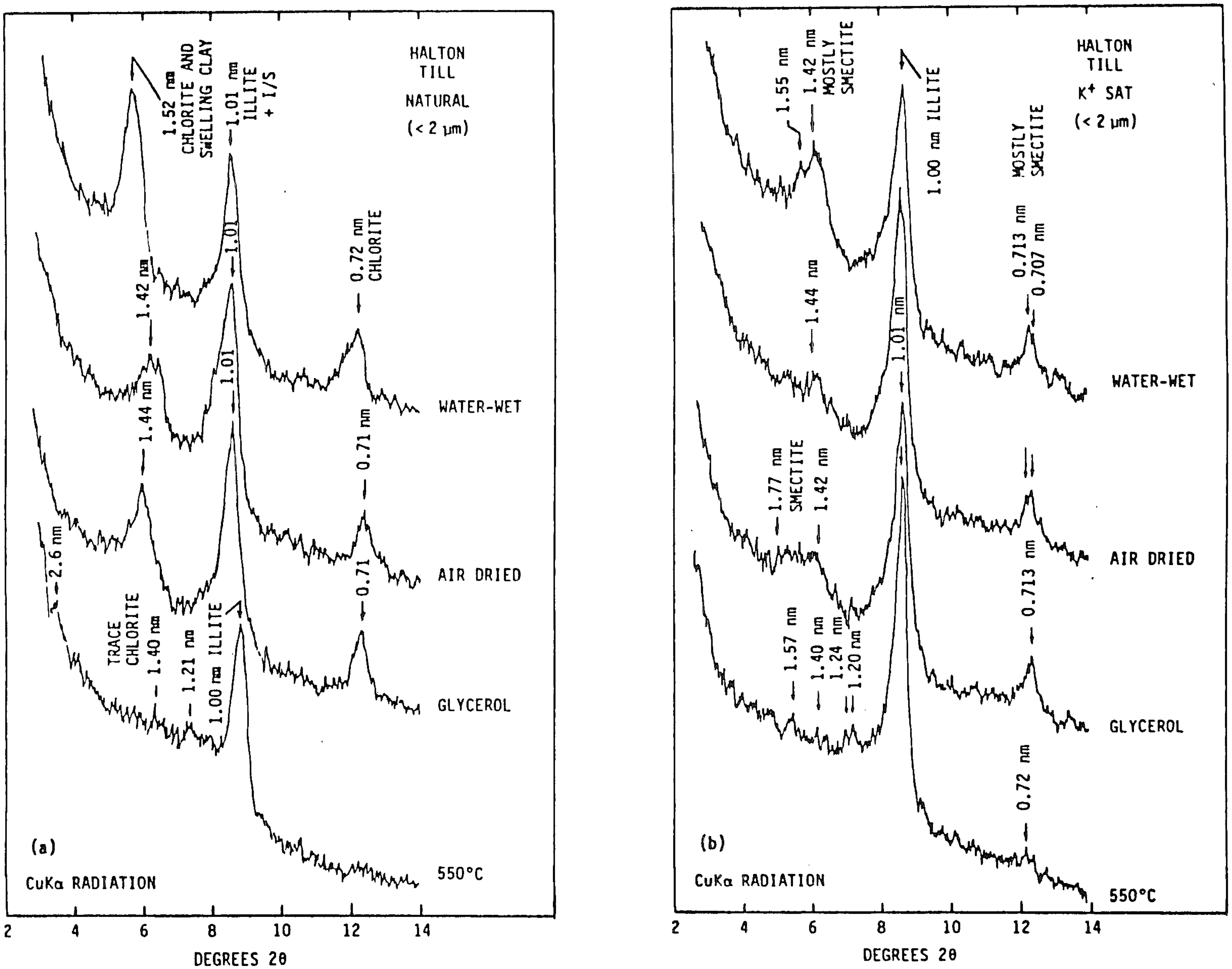


FIGURE 7. X-RAY TRACES OF ORIENTED, < 2 μm FINES OF HALTON TILL: (a) UNTREATED EXCEPT FOR VERY DILUTE NaOH IN DISTILLED WATER FRACTIONATION; (b) K^+ SATURATED

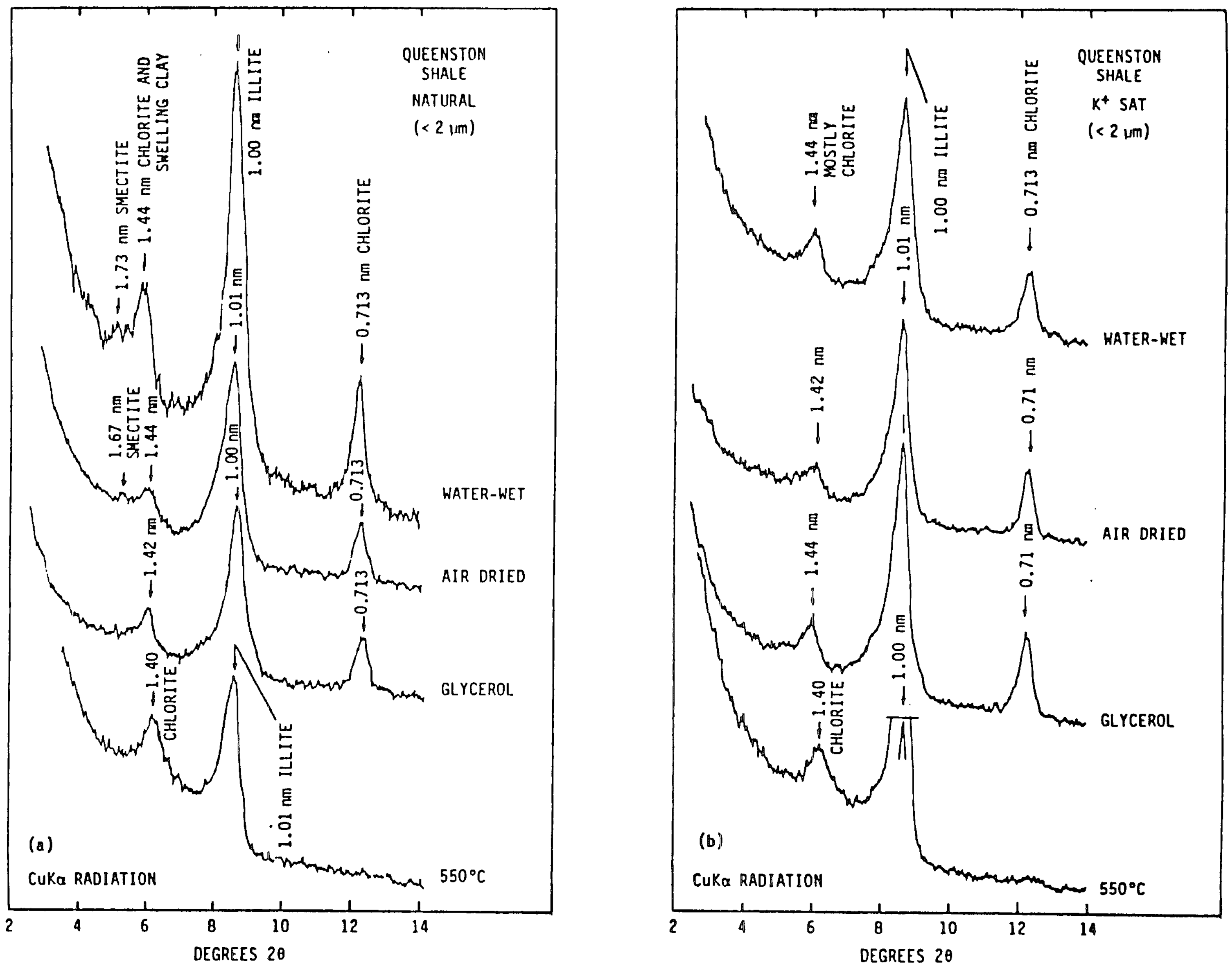


FIGURE 8. X-RAY TRACES OF ORIENTED, < 2 μm FINES OF QUEENSTON SHALE: (a) UNTREATED EXCEPT FOR VERY DILUTE NaOH IN DISTILLED WATER FRACTIONATION; (b) K^+ SATURATED

APPENDIX 1

TESTING PROTOCOLS

1. Harvard Miniature Compaction
2. Fixed Ring Hydraulic Conductivity
3. Triaxial Hydraulic Conductivity

1. Harvard Miniature Compaction Test

This test involves compaction of samples into a small, 62.35 cm^3 mold using a spring-loaded 12.5 mm diameter rod pushed many times into the soil to simulate kneading compaction. The procedures used are those outlined in Procedures for Testing Soils, ASTM Committee D-18, April 1958; "Suggested method of test for moisture-density relations of soils using Harvard compaction apparatus", S.D. Wilson, pp. 133-135.

Briefly, each test specimen is allowed to equilibrate with water at least overnight, then compacted in five layers in a 71.38 mm high mold. For the tests employed in this series, 20 prods per layer were applied using a 0.142 kN spring.

Experience in our laboratory has shown that for the clayey soils of southern Ontario, the above procedures produce compaction densities and optimum water contents very close to those obtained by Proctor compaction (Quigley, R.M., 1978. Compaction-strength-stabilization properties of weathered surface clays of southwestern Ontario, Ontario Ministry of Transportation and Communication Research Report #RR 215, 36 p.).

2. Fixed Ring Hydraulic Conductivity

2.1 Soil Compaction

The soil was compacted in stainless steel, fixed ring permeameters having a 5.4 cm inside diameter. A 12.5 mm diameter Harvard Miniature rod mounted with a 0.142 kN (32 lb) spring was used to compact the soils in three layers using 30 prods/layer. The upper surface was then trimmed producing a sample 2 cm in thickness.

Although the compaction energy/mass of soil is similar to that employed in the Harvard miniature compaction test, the presence of only three layers means that more energy is transferred to the lower layers during compaction and occasionally greater densities may be obtained as more air is apparently expulsed from the samples during compaction. This may have been the case for hydraulic conductivity test specimen #1 for the

Queenston shale (Figure 1) which appears to be ~ 97% saturated when plotted with the compaction data and had a calculated value of ~ 98% (Table 3). Alternatively, a sample of slightly different composition may have been taken from the bulk sample. The more "plastic" Halton till was not influenced by the number of layers (Point #3, Figure 1).

In comparing results from one test to another, especially for the shale, it should be remembered that the shale arrived in variably weathered broken chunks so small variations in specific gravity and compaction behaviour would be expected due to difficulties with sample mixing and sampling.

2.2 Soil Permeation

Once trimmed, a vertical consolidation pressure, σ'_v, of 100 kPa was applied to the top of the sample by means of a spring-loaded cell cap. Volume changes were measured by a dial gauge mounted on the permeability cell.

Following overnight consolidation, the permeameter cells were filled with tap water and connected to the piston system forcing the permeant through the samples at the selected constant flow rate, q. The induced head drop across the sample, Δh_t, was monitored by a pressure transducer and values of hydraulic conductivity, **k**, were calculated according to Darcy's law:

$$k = \frac{q \times Th}{\Delta h_t \times A} = \frac{q}{iA}$$

where Th and A are the sample thickness and cross-sectional area, respectively. The test specimens are weighed after testing to determine an approximate value of percent saturation.

3. Triaxial Hydraulic Conductivity

3.1 Soil Compaction

The test soil was compacted in a 5 cm inside diameter, brass cylinder in layers of ~ 1 cm thickness using 30 prods per layer, which provides a compaction energy per mass of soil similar to the Harvard miniature compaction tests presented on Figure 1. The soil sample was then extruded, trimmed to the desired thickness and placed in a standard triaxial cell. The lateral surface of the specimen was sealed by two latex membranes separated by a layer of high vacuum grease. The triaxial cell was then assembled and filled with distilled water. A pressure of 50 kPa was applied to both the cell and back pressure lines and the pore pressure response checked until the pore pressure parameter B was greater than 0.96. The cell pressure, σ_3, was then raised to its final value of 150 kPa producing an isotropic consolidation pressure, σ_3', of 100 kPa. Both ends of the sample were allowed to drain. All pressure lines were connected to a mercury/water constant pressure system.

The compacted Halton till was tested using a specimen height of ~ 10 cm. The compacted Queenston shale was tested using a specimen ~ 6 cm high to expedite consolidation which took 4 days for the till and 2 days for the shale. The Halton till had the same initial density as for the fixed wall test with a data point close to the Harvard compaction curve (Point #3, Figure 1). The compacted shale also had a density close to that for the fixed wall test (Point #2 compared to Point #1), but again plotted above the compaction curve suggesting close to 98% saturation which calculations showed to be the case (~ 99%, Table 3).

3.2 Soil Permeation

Following consolidation, the pore pressure acting on the top of the sample was raised to produce a constant downward hydraulic gradient, i, of 20. Influent and effluent flows were monitored using volume change devices until reasonably constant flow rates, q, were established. The hydraulic conductivity was then calculated using

$$k = \frac{q}{iA}$$

where A is the cross-sectional area of the sample.

In this type of test the sample is not rigidly confined so that creep and stress release effects may cause changes in the length and cross-sectional area of some test specimens. It may thus take a very long time for the influent and effluent flows to equalize. Practical considerations such as the necessary level of accuracy balanced against cost determine when the test is terminated.

APPENDIX "C"

Geocon

GEOTECHNICAL CONSULTANTS

GEOCON INC.
3210 AMERICAN DRIVE, MISSISSAUGA
ONTARIO, CANADA L4V 1B3
TEL.: (416) 673-1664
FAX.: (416) 673-0282

August 2, 1988

David Estrin
Barristers and Solicitors
22 Scollard Street
Toronto, Ontario
M5R 1E9

RE: SUITABILITY OF ON-SITE BARRIER MATERIALS
PROPOSED LANDFILL SITE F
BURLINGTON, ONTARIO

Dear Mr. Estrin:

In March 1988 samples of weathered Queenston Shale and Halton Till were collected from the site to conduct laboratory testing to assess the suitability of the two materials for use as compacted liner or cover barriers. After classification tests, grain size and Atterberg Limits, and preliminary compaction testing were undertaken in Geocon Mississauga laboratory, samples of each of the two materials were shipped to the University of Western Ontario for testing at the Geotechnical Research Centre. In addition to the specialized hydraulic conductivity testing on compacted shale and till, specimens were also subjected to preliminary mineralogical assessment in the laboratory. The results of the specialized testing are presented in the University of Western Ontario report accompanying this letter.

Source of Materials

The Halton Till and weathered Queenston Shale samples, delivered to the University of Western Ontario, were collected by Geocon from Site F in Burlington in March 1988. The shale sample was hand dug from a weathered shale bedrock exposure on the west side of the site. The till sample was the combination of a soil sample taken at a depth between 2 and 2.5 m from two backhoe test pits, which were approximately 100 m apart and were located in the open field on the north portion of the site. The test pits were located within the proposed fill area as shown on attached Drawing No. 1.

The till and shale samples were bagged as collected in the field. Representative specimens from each bulk sample were subjected to classification tests and preliminary compaction testing. Following this testing in Geocon Mississauga laboratory, the two similar till samples were combined into a composite sample and shipped to the University of Western Ontario with the weathered shale sample. These samples were shipped at their natural moisture content without any pre-treatment.

Characteristics of Materials

Laboratory testing of the Halton Till reveals the clay fraction varies between 24 and 32 percent and the material has a plasticity index of about 16 percent. These characteristics are within the range reported by Gartner Lee Limited and Trow from testing of other till samples in and adjacent to the north portion of the site. The laboratory testing at the University of Western Ontario on compacted Halton Till gave hydraulic conductivity values of 7 to 8 x 10^{-9} cm/sec. In addition, the mineralogical testing revealed that the mineralogy of the Halton clay sample was similar to other soils in Ontario that have performed satisfactorily as clay barriers against domestic waste leachate migration.

Geocon

Further testing of both the Halton Till and Queenston Shale materials should be anticipated to not only investigate leachate compatability with these materials but also in the development of material and construction specifications and field quality assurance protocols for construction of the liner and final cover barriers. It may be necessary to construct test pads with the two materials in order to demonstrate field performance.

We trust that you will find the opinions of this letter and the accompanying University of Western Ontario report satisfactory for your current needs on the project. If you should have any questions or wish any additional information then please call.

Very truly yours
GEOCON INC.

R.A. Sullivan, P.Eng.
General Manager

RAS:bg
T11131/43805