Appendix A

Acres International Limited Dam Inspection Report

Form B2
November 12 and 19, 2002

Form B2

Dam Inspection Report

Date: November 12 and 19, 2002

Structure: Harrington Dam

Municipality: Zorra

Location: Zorra Township, Oxford County, in the town of Harrington

GPS Coordinates: UTM, NAD83: 17 500 642 E, 4 787 565 N
Lat/Long: 43° 14' 27" N, 80° 59' 32" W

Inspected By: B. Craig, T. Hartung, P. Last, M. Ragwen and B. Sinclair of Acres International Limited

Weather: Cloudy overcast, air temperature approximately 6°C

1. Earth Embankment

For details, see the photographs in Appendix A and the attached drawings.

- Upstream slope of left embankment sparsely protected by cobbles and boulders. Signs of erosion are visible along shore.
- Downstream left embankment shows signs of bulging. Seepage visible below bulging. Evidence of internal erosion along seepage path exists.
- Indication of washout between left embankment and concrete spillway.
- No signs of distress on right embankment. Steep slope on downstream side layered with geotextile netting to promote vegetation growth.
- Invert of millrace seems higher than crest of dam.

2. Concrete Structures (wingwalls, piers, deck, spillways, apron, etc)

For details, see the photographs in Appendix A and the attached drawings.

- Spalls and cracking over surface of left and right abutment and wingwalls. Potential alkali-aggregate reaction.
- Significant map cracking and spalls at upstream end of left abutment. Steel reinforcement exposed.

- Hydraulic erosion near waterline on left abutment and around low-level outlet.
- Large stress crack in downstream left wingwall at location of low water outlet.

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3. **Wooden and Metal Structures** (decks, gains, railings, conduits, etc)

For details, see the photographs in Appendix A and the attached drawings.

- Railings along the top of the bridge deck appear in good condition but require painting. Height meets code requirements; however, openings must be checked since dam has full public access.
- Steel gains are in good condition with light rust.
- The 51-mm deep galvanized steel deck grating is in good condition with minor rust. Significant deflection obtained when standing midspan.
- Steel deck supports appear in good condition with minor rusting.

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4. **Gates and/or Stop Logs** (identified looking downstream left to right)

For details, see the photographs in Appendix A and the attached drawings.

Three stop log bays directly below steel decking. Steel decking must be removed for access. Operator survey indicates that logs are left all year and are difficult to remove during flood conditions. Stop logs appear in good condition.

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5. **Water Level Gauge** (reading and condition)

For details, see the photographs in Appendix A.

No water level gauge is located on-site. Operator survey indicates that measurements are taken with reference to dam deck.

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6. **Winches** (type and number)

For details, see the photographs in Appendix A.

No winches are located on-site. Logs removed manually.

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7. **Valves** (type and number)

For details, see the photographs in Appendix A.

One low-level outlet valve in the left abutment. Controls for valve housed in locked wooden box on upstream side of left abutment. Access hatch is located within a locked fenced area. Not operated during inspection. Minor leaking visible from outlet. Operator survey indicates valve may not be operable.

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8. **Boom** (driftwood, chains, anchors)

For details, see the photographs in Appendix A.

No boom present at this site, and none is recommended.

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9. **Erosion** (upstream and downstream)

For details, see the photographs in Appendix A and the attached drawings.

Erosion has on the upstream shore of the left abutment.

Washout and erosion between embankments and abutments due to overtopping of dam. Repair has been attempted with gabions and sandbags.

Internal erosion of dam through seepage planes is visible by muddy water emanating from downstream face of left embankment.

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10. **Seepage or Leaks**

For details, see the photographs in Appendix A.

Seepage through the left embankment seen as wet areas on the downstream face.

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11. **Access Route** (location of gate keys, winch handles and keys)

Vehicular access is possible to left dam bank via access road from Harrington CA parking lot. UTRCA maintenance has keys to access gate and low flow valve access hatch. Dam within walking distance from Harrington CA parking lot.

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12. **Safety Issues** (public and operator)

- No warning signs for boaters or public using the dam.
- Invert of millrace seems higher than dam crest promoting overtopping during flood conditions.

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13. Signage

For details, see the photographs in Appendix A.

- "Danger, Fast Current, No Boating, No Swimming" sign missing from upstream side of dam.
- No "Use at Own Risk" signs for public using the deck as a pedestrian crossing.

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14. Divestment and/or Decommissioning Opportunities

Annual agreement for area management.

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15. General Remarks

The dam is generally in adequate condition but requires further repair and maintenance. Major dam safety deficiencies exist.

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16. Recommendations

- Install "Use at Own Risk" signs at both ends of the dam.
- Ensure openings in guardrails on deck to conform to code requirements in areas where public access is permitted.
- Check serviceability of bridge deck and modify as required.
- Investigate operational status of low flow valve.
- Regrade millrace to ensure proper diversion during flood conditions.
- Repair cracks and deterioration in dam as required.
- Determine extent of seepage through dam and repair as required.

Appendix B

Acres International Limited Borehole Log, Laboratory Test Results and Civil/Structural and Geotechnical Assessments

Boreholes HT BH1, HT BH2 and HT BH3
Plasticity Chart
Consolidated Undrained Triaxial Compression Tests
Grain Size Distributions
Civil/Structural Assessment
Geotechnical Assessment

ACRES

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority

PROJECT: Dam Safety Assessment

HOLE: HT BH1

PAGE: 1 **OF:** 4

SITE: Harrington Dam

COORDINATES: 4.90m from left pier on dam centerline

DIP DIRECTION: 0

DIP: 90

ELEVATIONS

DATUM: Geodetic

PLATFORM:

GROUND: 331.04

END OF HOLE: 322.81

CONTRACTOR: Atcost Soil Drilling Inc.

DRILL TYPE: CME 75

METHOD SOIL: Hollow stem auger

ROCK:

CASING: Auger 200mm OD

CORE:

STARTED: 21 Nov 2003

FINISHED: 21 Nov 2003

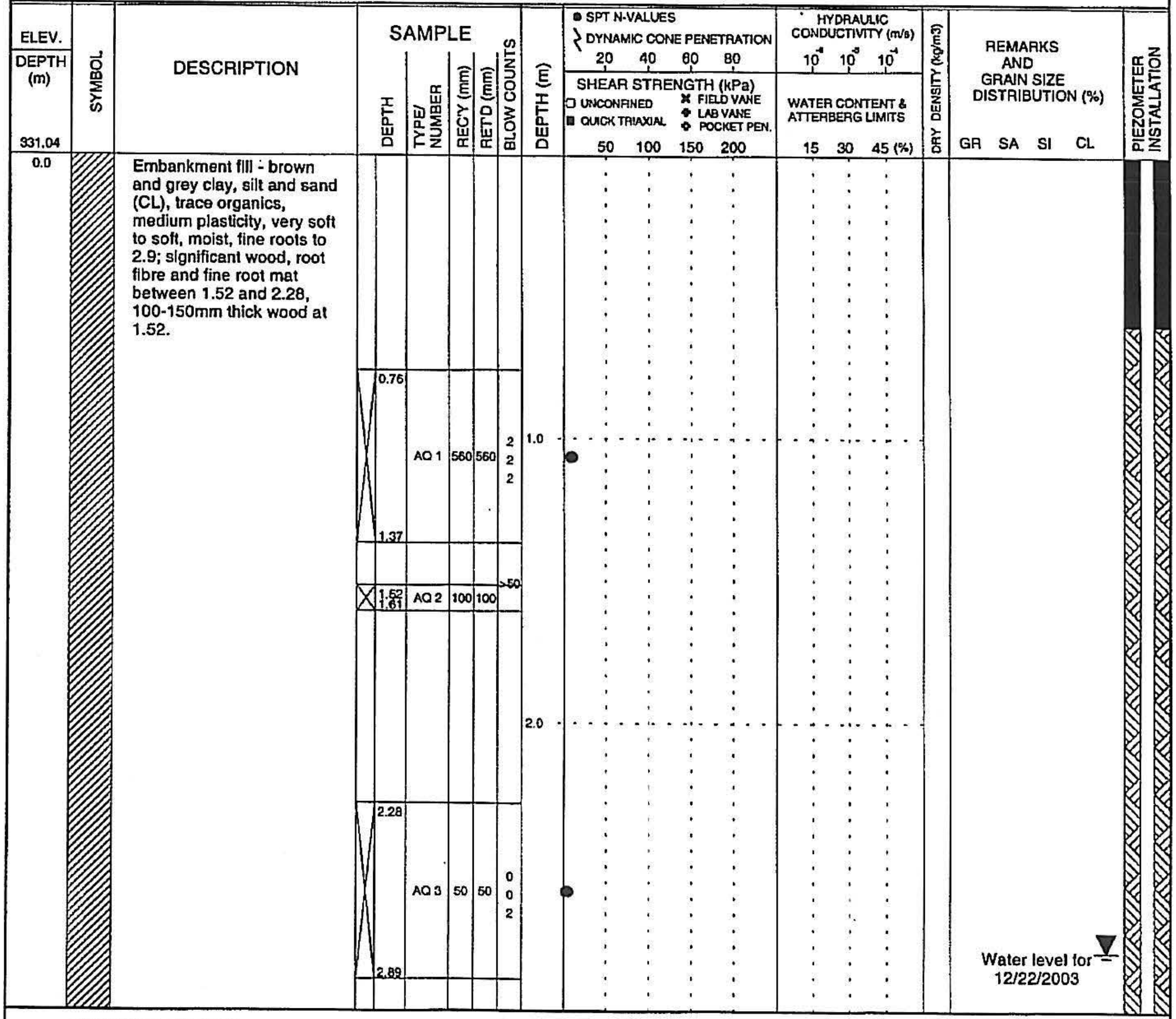
INSPECTOR: D. Besaw

LOGGED BY: D. Besaw

REVIEWED: B Sinclair

DATE: 31/3/04

See end of log for detailed groundwater measurements



ELEV. DEPTH (m)	DESCRIPTION	SAMPLE DEPTH	TYPE/ NUMBER	RECY (mm)	RET'D (mm)	BLOW COUNTS
331.04 0.0	Embankment fill - brown and grey clay, silt and sand (CL), trace organics, medium plasticity, very soft to soft, moist, fine roots to 2.9; significant wood, root fibre and fine root mat between 1.52 and 2.28, 100-150mm thick wood at 1.52.					
		0.76 1.37	AQ 1	560	560	2 2 2
		1.52 1.61	AQ 2	100	100	>50
		2.28 2.89	AQ 3	50	50	0 0 2

Water level for 12/22/2003

SAMPLING METHOD

A - Split Tube
B - Thin Wall Tube
C - Piston Sample
D - Core Barrel
E - Auger
F - Wash
G - Shovel Grab
K - Slotted

SHIPPING CONTAINER

N - Insert
O - Tube
P - Water Content Tin
Q - Jar
R - Cloth Bag
S - Plastic Bag
U - Wooden Box
Y - Core Box
Z - Discarded

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W_N — LIQUID LIMIT W_L

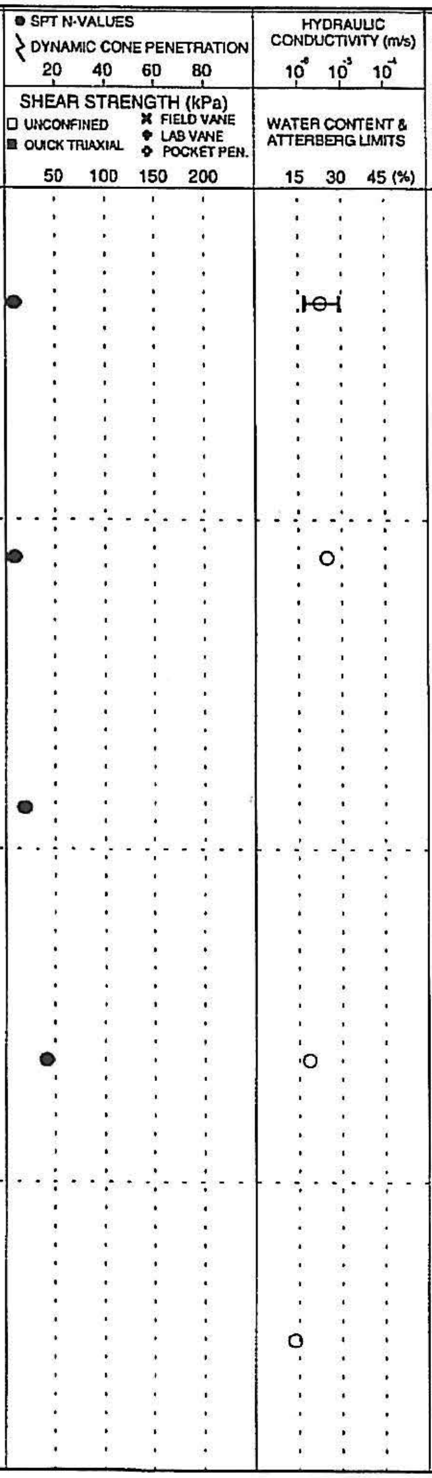
☐ Constant Head Test
▨ Falling Head Test
▩ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: HT BH1
PAGE: 2 **OF:** 4

ELEV. DEPTH (m)	DESCRIPTION	SAMPLE DEPTH	TYPE/ NUMBER	REC'Y (mm)	RET'D (mm)	BLOW COUNTS	DEPTH (m)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		3.04 3.65	AQ 4	610	610	0 2 2		
		3.81 4.42	AQ 5	585	585	1 1 3	4.0	
326.37 4.67	Foundation - tan colored silt, clay and sand (ML), low plasticity, stiff, moist, no dilatancy, contains roots and wood fibre, capped with 100 mm of organic sand and gravel with max size of 40 mm. Original ground is defined by about 300mm of moist black organics and wood at base of embankment.	4.57 5.18	AQ 6	215	215	6 5 3	5.0	
		5.33 5.94	AQ 7	510	510	5 6 11		CME continuous sample also taken from 5.33-6.09 See gradation for AQ7.
324.95 6.09	Sand (SW)- fine to coarse silty sand with 10% gravel, max size gravel 75mm, fine roots, wet. At times dirty, large gravel piece in AQ 8 sample is encased in 1 cm of silt/clay.	6.09 6.85	AS 8	200	200		6.0	See gradation for AS8. AS 8 is CME continuous sampler



DRY DENSITY (kg/m3)

PIEZOMETER INSTALLATION

SAMPLING METHOD

A - Split Tube
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PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W_N LIQUID LIMIT W_L

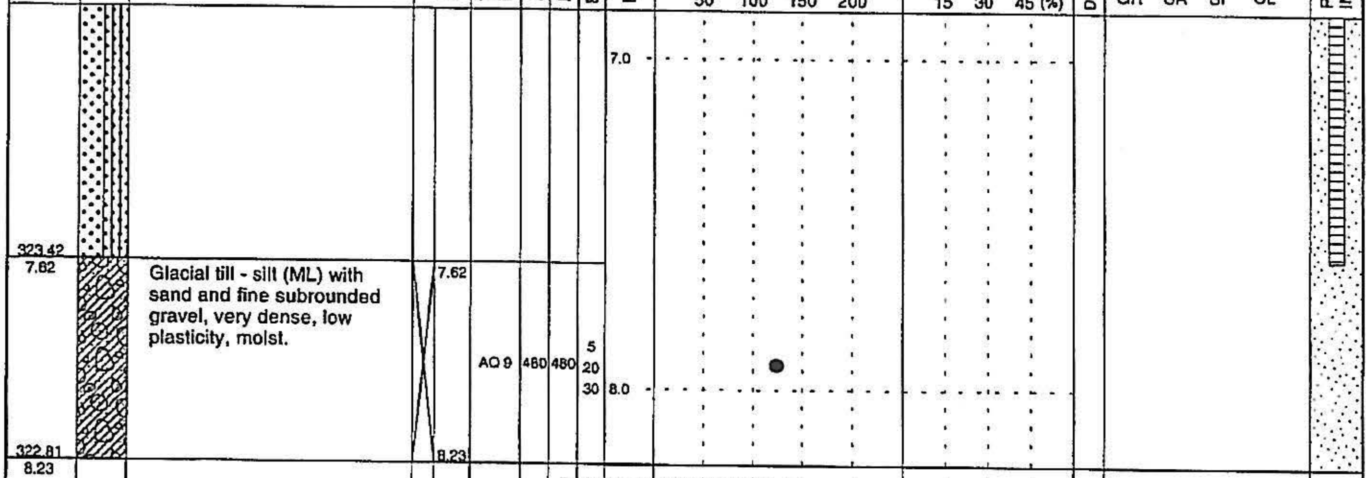
☐ Constant Head Test
▨ Falling Head Test
▩ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: HT BH1
PAGE: 3 **OF:** 4

ELEV. DEPTH (m)	SYMBOL	DESCRIPTION	SAMPLE DEPTH	SAMPLE TYPE/ NUMBER	SAMPLE REC'Y (mm)	SAMPLE RET'D (mm)	SAMPLE BLOW COUNTS	DEPTH (m)	● SPT N-VALUES / DYNAMIC CONE PENETRATION 20 40 60 80 / SHEAR STRENGTH (kPa) ☐ UNCONFINED ■ QUICK TRIAXIAL ✖ FIELD VANE ✚ LAB VANE ✥ POCKET PEN. 50 100 150 200	HYDRAULIC CONDUCTIVITY (m/s) 10^{-6} 10^{-5} 10^{-4} / WATER CONTENT & ATTERBERG LIMITS 15 30 45 (%)	DRY DENSITY (kg/m3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	PIEZOMETER INSTALLATION
								7.0					
323.42 7.62		Glacial till - silt (ML) with sand and fine subrounded gravel, very dense, low plasticity, moist.	7.62 8.23	AQ 9	480	480	5 20 30	8.0	●				
322.81 8.23		END OF BOREHOLE											



SAMPLING METHOD

A - Split Tube
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PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W_N — LIQUID LIMIT W_L

☐ Constant Head Test
░ Falling Head Test
▒ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority

PROJECT: Dam Safety Assessment

HOLE: HT BH1

PAGE: 4 **OF:** 4

WATERLEVEL READINGS

11/21/2003 2:30:00 PM	2.88
12/22/2003 2:35:00 PM	2.81

NOTES/COMMENTS

1 **Water level Measurements**

Water level measurements are recorded from ground level.

The reservoir level was 0.93 m below borehole crest elevation for the 2.81 reading. Reservoir elevation was 330.00 m.

2 **Piezometer Installation**

Surface - Flush mount cap embedded in Sakcrete

0.0-0.6 Bentonite chips
0.6-5.79 Bentonite slurry
5.79-6.09 Coarse sand pack
6.09-7.6 Slotted screen in coarse sand pack
7.6-8.23 Coarse sand pack

Note: - riser and slotted screen consist of 50mm ID rigid, flush-coupled PVC

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment
HOLE: HT BH2
PAGE: 1 **OF:** 4

SITE: Harrington Dam

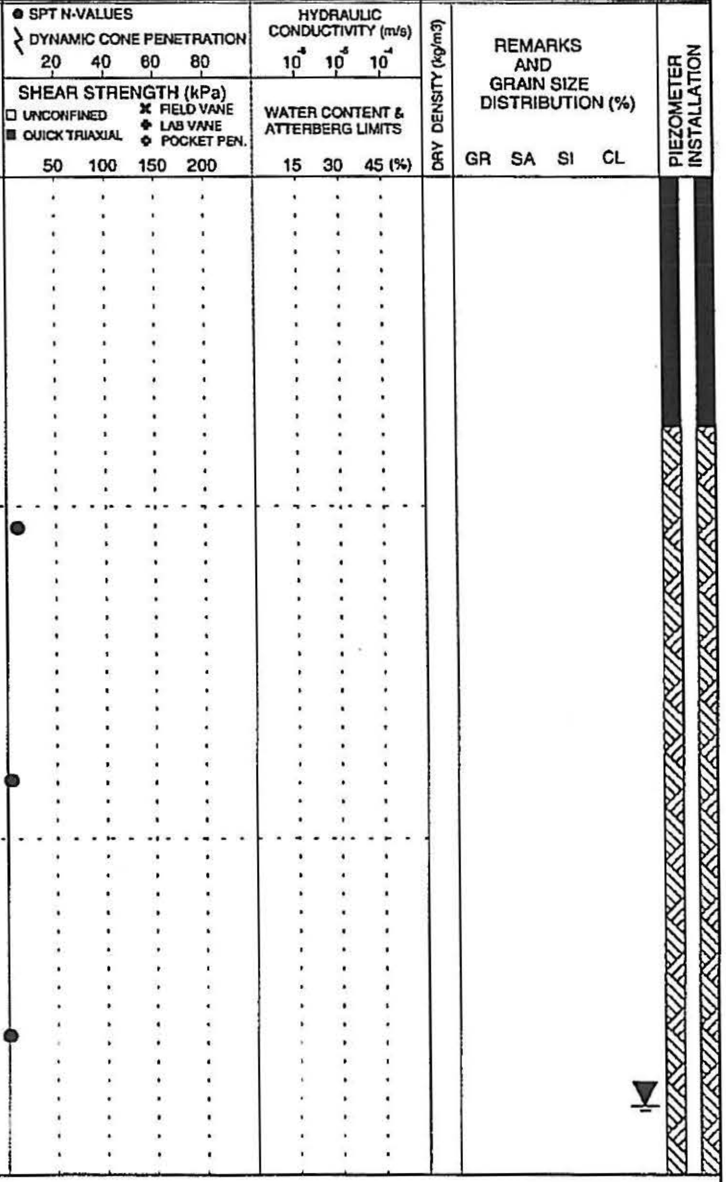
COORDINATES: 25.7 m from left pier on dam centerline
DIP DIRECTION: 0
DIP: 90
ELEVATIONS
DATUM: Geodetic
PLATFORM:
GROUND: 331.02
END OF HOLE: 323.56

CONTRACTOR: Atcost Soil Drilling Inc.
DRILL TYPE: CME 75
METHOD SOIL: Hollow stem auger
ROCK:
CASING: Auger 200mm OD
CORE:

STARTED: 21 Nov 2003
FINISHED: 21 Nov 2003
INSPECTOR: D. Besaw
LOGGED BY: D. Besaw
REVIEWED: B Sinclair
DATE: 31/3/04

See end of log for detailed groundwater measurements

ELEV. DEPTH (m)	SYMBOL	DESCRIPTION	SAMPLE DEPTH	TYPE/ NUMBER	REC'Y (mm)	RET'D (mm)	BLOW COUNTS	DEPTH (m)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
331.02 0.0	▨	Embankment fill - dark brown and dark gray clay, silt and sand (CL), very soft, low plasticity, lenses of sand with gravel at 1 m and 2 m, trace organics, moist, fine roots to 1.3 m.							
			0.76 1.37	AQ 1	560	560	0 2 3	1.0	
			1.52 2.13	AQ 2	610	610	1 1 1	2.0	
			2.28 2.89	AQ 3	510	510	0 0 1		



SAMPLING METHOD

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- D - Core Barrel
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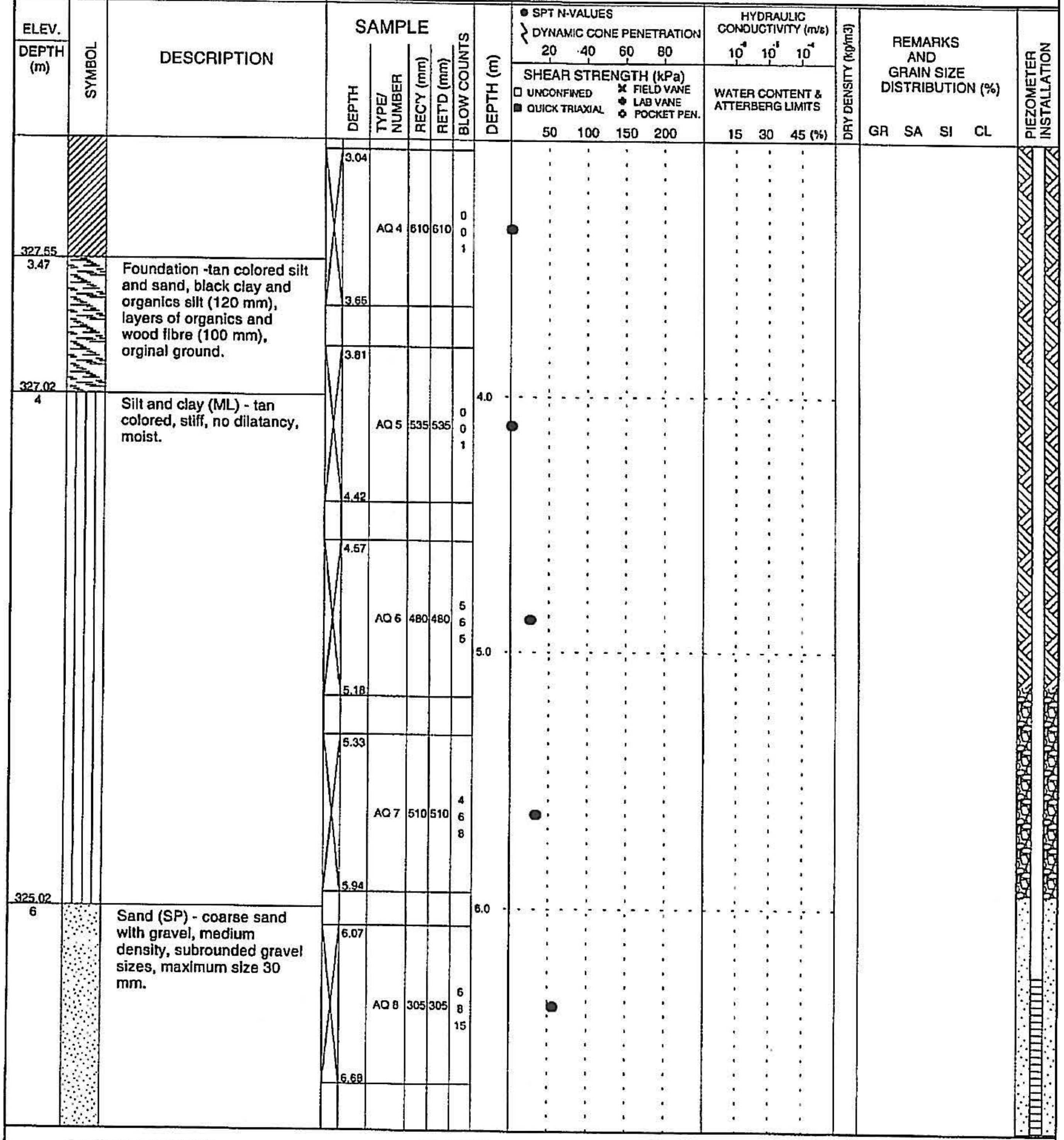
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W_N — LIQUID LIMIT W_L

☐ Constant Head Test
▦ Falling Head Test
▩ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment
HOLE: HT BH2
PAGE: 2 **OF:** 4



ELEV. DEPTH (m)	DESCRIPTION
327.55 3.47	Foundation -tan colored silt and sand, black clay and organics silt (120 mm), layers of organics and wood fibre (100 mm), orginal ground.
327.02 4	Silt and clay (ML) - tan colored, stiff, no dilatancy, moist.
325.02 6	Sand (SP) - coarse sand with gravel, medium density, subrounded gravel sizes, maximum size 30 mm.

DEPTH	TYPE/ NUMBER	RECY (mm)	RETD (mm)	BLOW COUNTS
3.04 3.65	AQ 4	610	610	0 0 1
3.81 4.42	AQ 5	535	535	0 0 1
4.67 5.18	AQ 6	480	480	5 6 5
5.33 5.94	AQ 7	510	510	4 6 8
6.07 6.68	AQ 8	305	305	6 8 15

SAMPLING METHOD

- A - Split Tube
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- G - Shovel Grab
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SHIPPING CONTAINER

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PLASTIC LIMIT — NATURAL MOISTURE CONTENT — LIQUID LIMIT

W_P — W_N — W_L

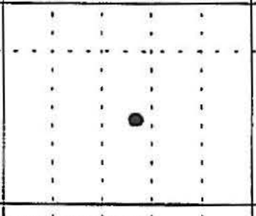
☐ Constant Head Test
⬚ Falling Head Test
▨ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: HT BH2
PAGE: 3 **OF:** 4

ELEV. DEPTH (m)	SYMBOL	DESCRIPTION	SAMPLE DEPTH	SAMPLE TYPE/ NUMBER	SAMPLE RECY (mm)	SAMPLE RET'D (mm)	SAMPLE BLOW COUNTS	DEPTH (m)	● SPT N-VALUES / DYNAMIC CONE PENETRATION 20 40 60 80 / SHEAR STRENGTH (kPa) ☐ UNCONFINED ■ QUICK TRIAXIAL ✖ FIELD VANE ✚ LAB VANE ✧ POCKET PEN. 50 100 150 200	HYDRAULIC CONDUCTIVITY (m/s) 10^{-4} 10^{-5} 10^{-4} / WATER CONTENT & ATTERBERG LIMITS 15 30 45 (%)	DRY DENSITY (kg/m3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	PIEZOMETER INSTALLATION
324.07 6.95		Glacial till - silt with sand (ML) and fine subrounded gravel, very dense, moist.	6.95 7.46	AQ 9	480	480	10 18 36	7.0	●				
323.56 7.48													



END OF BOREHOLE

SAMPLING METHOD

A - Split Tube
B - Thin Wall Tube
C - Piston Sample
D - Core Barrel
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SHIPPING CONTAINER

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PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W_N — LIQUID LIMIT W_L

☐ Constant Head Test
▨ Falling Head Test
▩ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
HOLE: HT BH2
PROJECT: Dam Safety Assessment
PAGE: 4 **OF:** 4

WATERLEVEL READINGS

11/21/2003 5:27:00 AM	2.79
12/22/2003 2:35:00 PM	2.81

NOTES/COMMENTS

1 Water Level Measurements

All water level measurements are referenced to ground level.

The reservoir level was 1.02 m below borehole crest elevation for the 2.81 reading. Reservoir elevation is 330.00 m.

2 Piezometer Installation

Surface - Flush mount protection cap

0-0.3 Flush mount embedded in Sakcrete
0.3-0.76 Bentonite chips capped with sand
0.76-5.12 Bentonite slurry
5.12-5.94 Flowing material
5.94-6.25 Coarse sand pack
6.25-6.85 Slotted screen in coarse sand pack

Note: - riser pipe and slotted screen consist of 50 mm ID flush coupled PVC.

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: HT BH3
PAGE: 1 **OF:** 3

SITE: Harrington Dam

COORDINATES: on toe of slope
14 m d/s of HT BH2
DIP DIRECTION: 0
DIP: 90
ELEVATIONS
DATUM: Geodetic
PLATFORM:
GROUND: 328.33
END OF HOLE: 324.68

CONTRACTOR: Atcost Soil Drilling Inc.
DRILL TYPE: CME 55
METHOD SOIL: Hollow stem auger
ROCK:
CASING: Auger 200mm OD
CORE:

STARTED: 24 Nov 2003
FINISHED: 24 Nov 2003
INSPECTOR: D. Besaw
LOGGED BY: D. Besaw
REVIEWED: B Sinclair
DATE: 31/3/04

See end of log for detailed groundwater measurements

ELEV. / DEPTH (m) 328.33	DESCRIPTION	SAMPLE DEPTH	TYPE/ NUMBER	REC'Y (mm)	RET'D (mm)	BLOW COUNTS	DEPTH (m)	● SPT N-VALUES / DYNAMIC CONE PENETRATION 20 40 60 80 / SHEAR STRENGTH (kPa) ☐ UNCONFINED ■ QUICK TRIAXIAL ✖ FIELD VANE ✚ LAB VANE ✜ POCKET PEN. 50 100 150 200	HYDRAULIC CONDUCTIVITY (m/s) 10^{-8} 10^{-5} 10^{-4} / WATER CONTENT & ATTERBERG LIMITS 15 30 45 (%)	DRY DENSITY (kg/m3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL ▼
0.0	Embankment fill - brown clay, silt and sand (CL), organic, soft, moist.	0 – 0.61	AQ1	520	520	1 1 2					
327.83 0.5	Foundation- black, firm, organic silt and organics, moist, topsoil, original ground.	0.76 – 1.37	AQ2	360	360	1 2 3	1.0				
326.91 1.42	Sand (SW) - sand with gravel, compact, fine to coarse, clean, subrounded gravel fraction, dense, wet, max size 25 mm.	1.52 – 2.13	AQ3	390	390	4 15 12	2.0				
		2.28 – 2.89	AQ4	520	520	13 18 15					

PIEZOMETER INSTALLATION

SAMPLING METHOD

A - Split Tube	E - Auger
B - Thin Wall Tube	F - Wash
C - Piston Sample	G - Shovel Grab
D - Core Barrel	K - Slotted

SHIPPING CONTAINER

N - Insert	R - Cloth Bag
O - Tube	S - Plastic Bag
P - Water Content Tin	U - Wooden Box
Q - Jar	Y - Core Box
	Z - Discarded

PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT
W_P W_N W_L

☐ Constant Head Test
⬚ Falling Head Test
▩ Lab. Permeability

ACRES

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: HT BH3
PAGE: 2 **OF:** 3

ELEV. DEPTH (m)	SYMBOL	DESCRIPTION	SAMPLE DEPTH	SAMPLE TYPE/ NUMBER	SAMPLE RECY (mm)	SAMPLE RET'D (mm)	SAMPLE BLOW COUNTS	DEPTH (m)	● SPT N-VALUES; DYNAMIC CONE PENETRATION 20 40 60 80; SHEAR STRENGTH (kPa) ☐ UNCONFINED ■ QUICK TRIAXIAL ✖ FIELD VANE ✚ LAB VANE ✜ POCKET PEN. 50 100 150 200	HYDRAULIC CONDUCTIVITY (m/s) 10^{-6} 10^{-5} 10^{-4}; WATER CONTENT & ATTERBERG LIMITS 15 30 45 (%)	DRY DENSITY (kg/m3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	PIEZOMETER INSTALLATION
325.28 3.07		Glacial till - silt (ML) with fine subrounded gravel (5-10%), very stiff, maximum size gravel is 50 mm; 100 mm wide fine to coarse sand lense in till.	3.04 3.65	AQ5	560	560	8 14 22						
324.68 3.65		END OF BOREHOLE											

SAMPLING METHOD

A - Split Tube
B - Thin Wall Tube
C - Piston Sample
D - Core Barrel
E - Auger
F - Wash
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K - Slotted

SHIPPING CONTAINER

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PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W_N LIQUID LIMIT W_L

☐ Constant Head Test
▨ Falling Head Test
▨ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment
HOLE: HT BH3
PAGE: 3 **OF:** 3

WATERLEVEL READINGS

11/24/2003 1:00:00 PM	0.0
12/22/2003 2:40:00 PM	-.02

NOTES/COMMENTS

1 **Water Level Measurements**

All water level measurements are referenced to ground level.

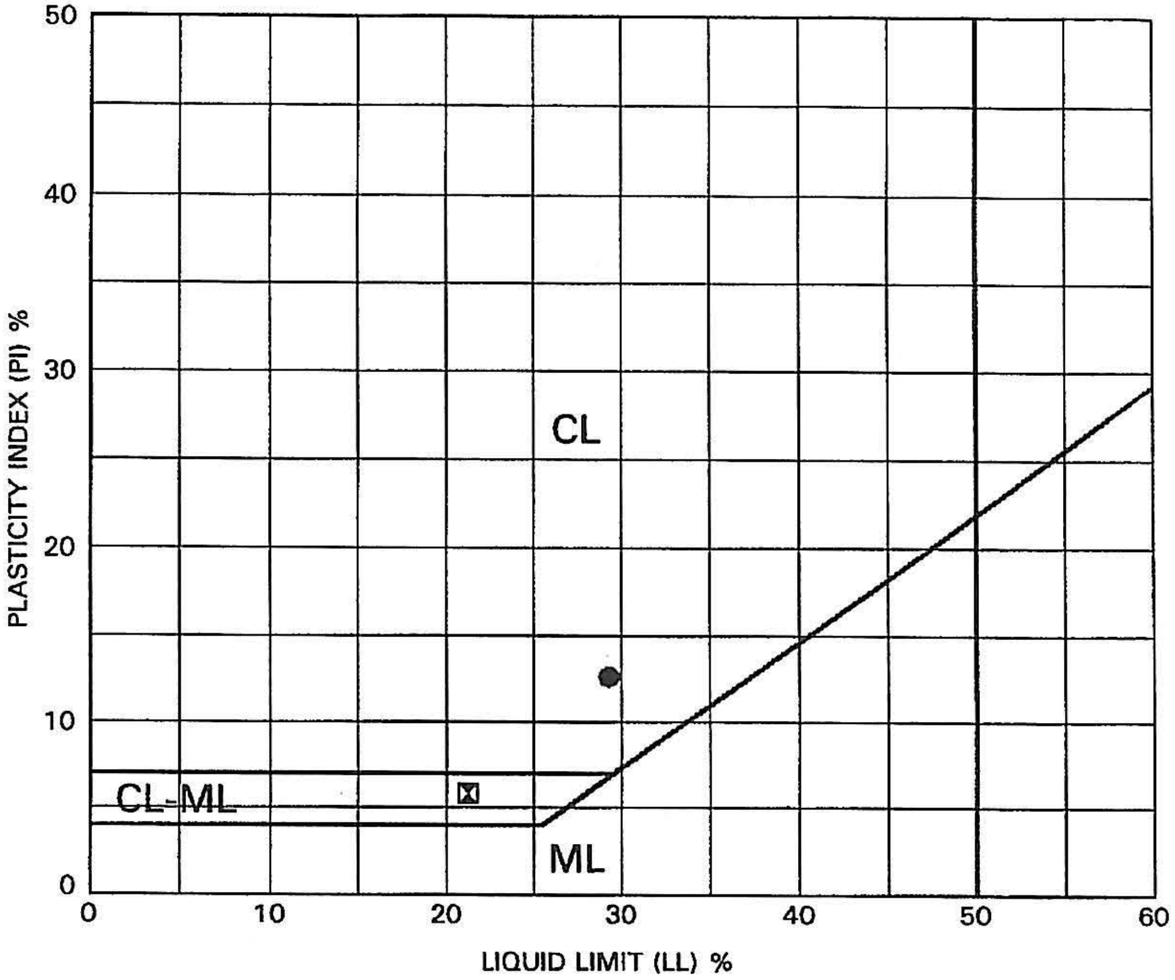
Reservoir elevation was 330.00 m.

2 **Piezometer Installation**

Surface - Flush mount protection cap

0-0.3 Flush mount embedded in Sakcrete
0.3-1.82 Bentonite chips capped with sand
1.82-2.13 Coarse sand pack
2.13-3.07 Slotted screen in coarse sand pack
3.07-3.65 Coarse sand backfill

Note: - riser pipe and slotted screen consist of 50 mm ID flush coupled PVC.



LEGEND	BH	DEPTH (m)	LL%	PL%	PI%
●	HTBH1	3.05	29	17	13
☒	HTBH2	0.00	21	16	6

Note: Sample from HTBH2 is a remolded composite sample taken from AQ2 and AQ3 and used for triaxial testing.

Geotechnical Investigations
Upper Thames Conservation Authority

PLASTICITY CHART
Harrington Dam

PROJECT NO. P1450404

ACRES

NP = Non-Plastic

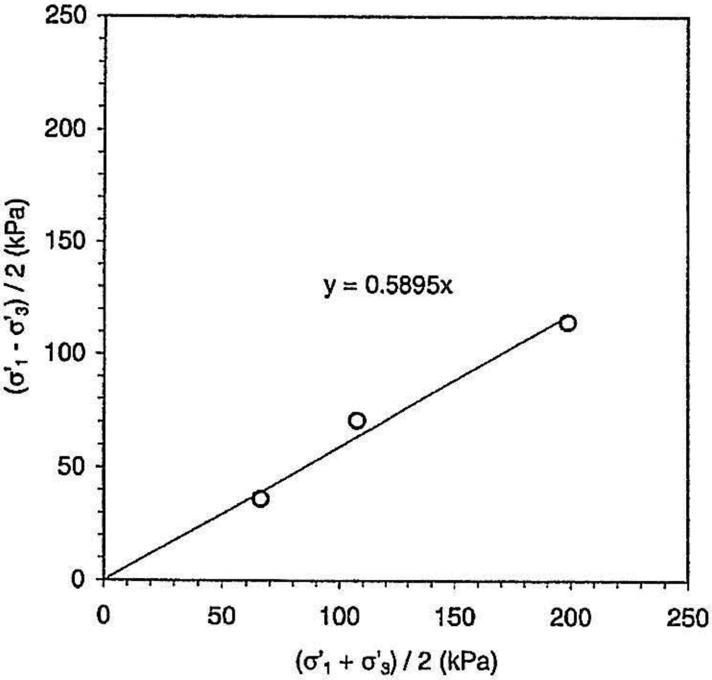
Acres International Ltd.

Project: P14504.04
Harrington Dam - UTRCA
Summary of Consolidated Undrained Triaxial Compression Tests

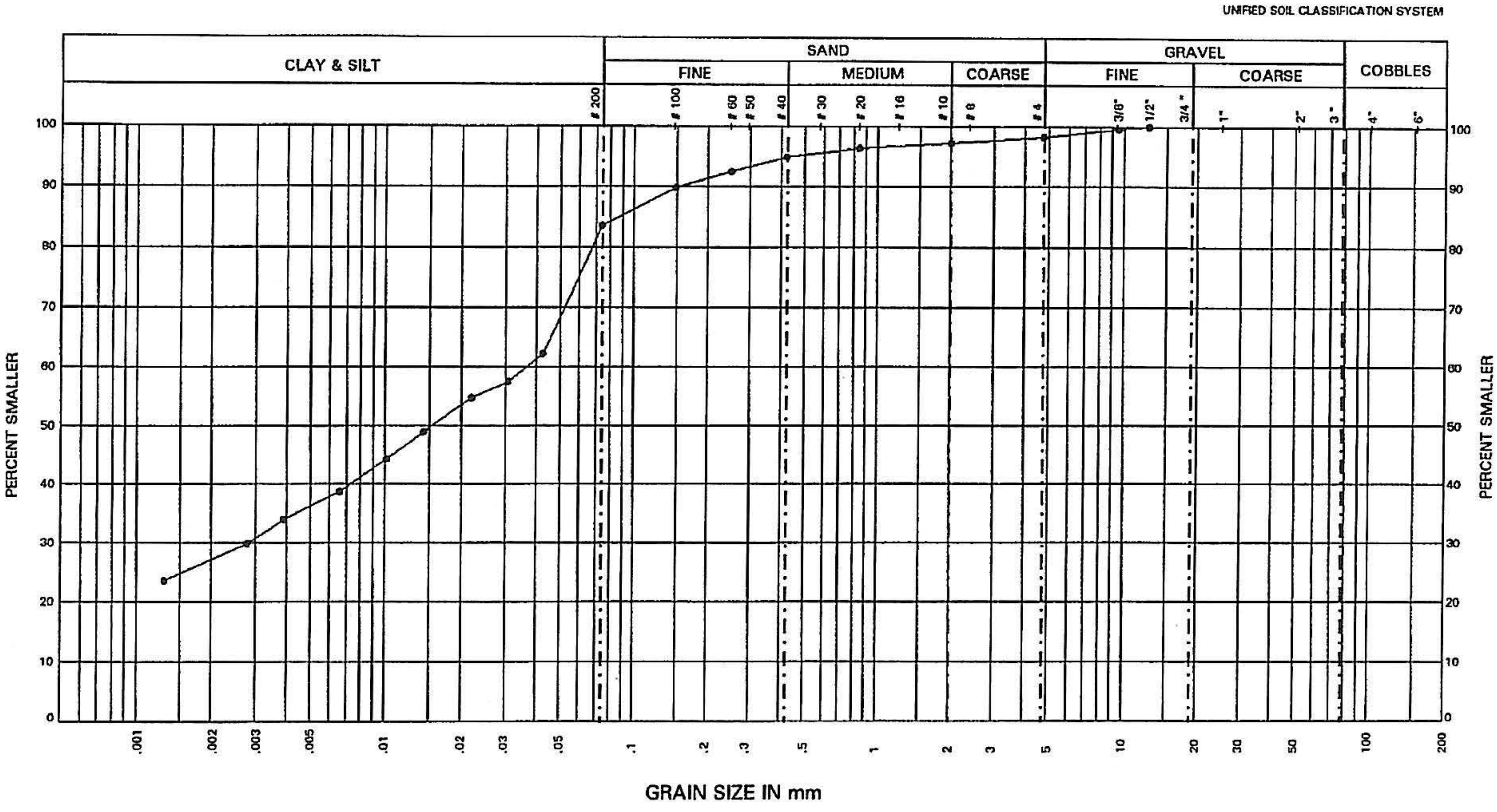
Date: March-04

Test	$(\sigma'_1 - \sigma'_3) / 2$ (kPa)	$(\sigma'_1 + \sigma'_3) / 2$ (kPa)
1	36.1	66.3
2	70.6	107.7
3	114.3	198.9

Harrington Dam
Consolidated Undrained Triaxial Compression Tests



Effective friction angle, $\phi' = 36^{\circ}$
Cohesion, c = 0

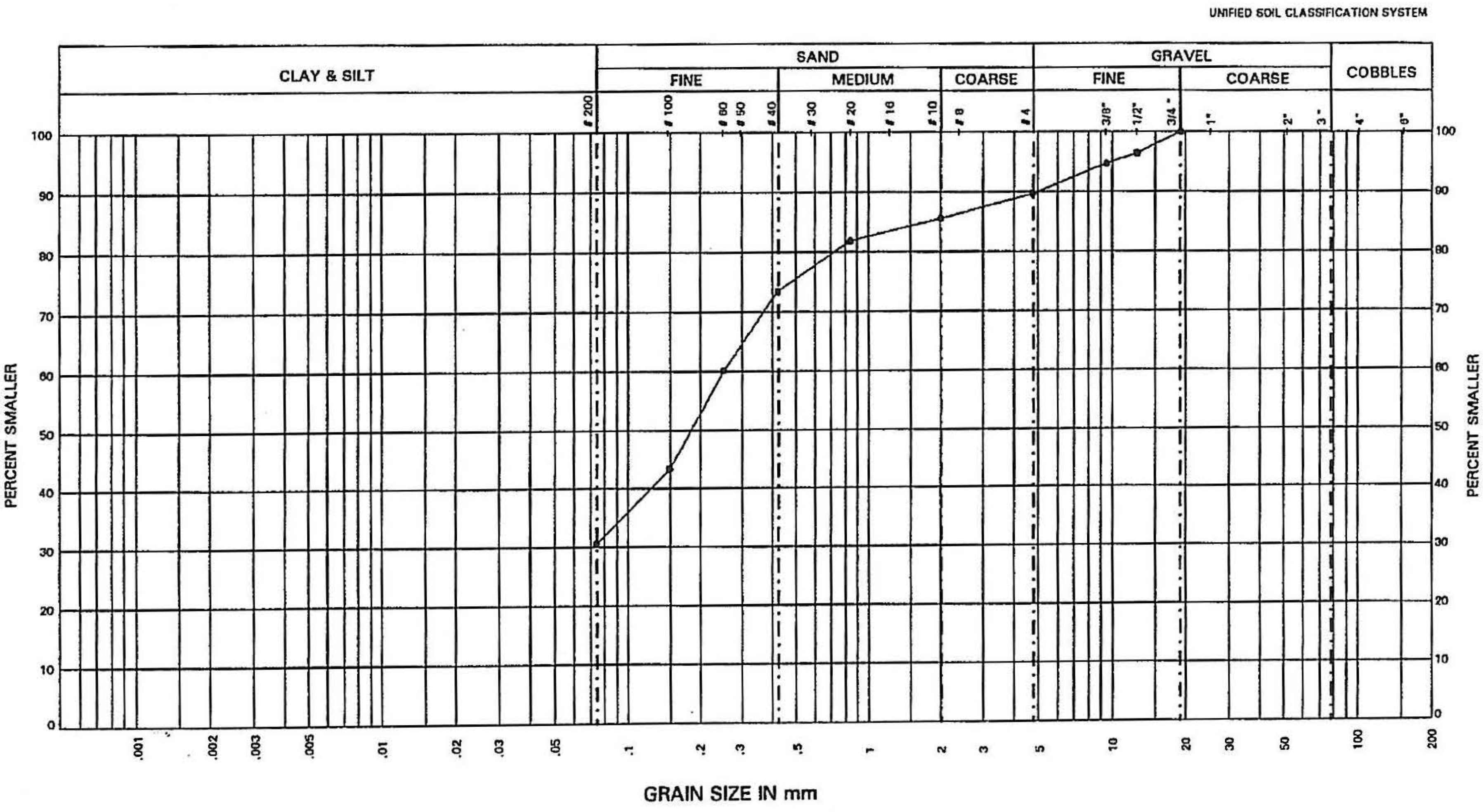


LAB SAMPLE NO.
LAB TEST NO.
DATE December 2003
TESTED BY RS
CHECKED BY AT
APPROVED BY

REMARKS: Harrington Dam
UTRCA

GRAIN SIZE DISTRIBUTION

DEPTH	SAMPLE NO.	HOLE NO.	JOB NO.
5.33m	AQ7	BH1	P14504.04



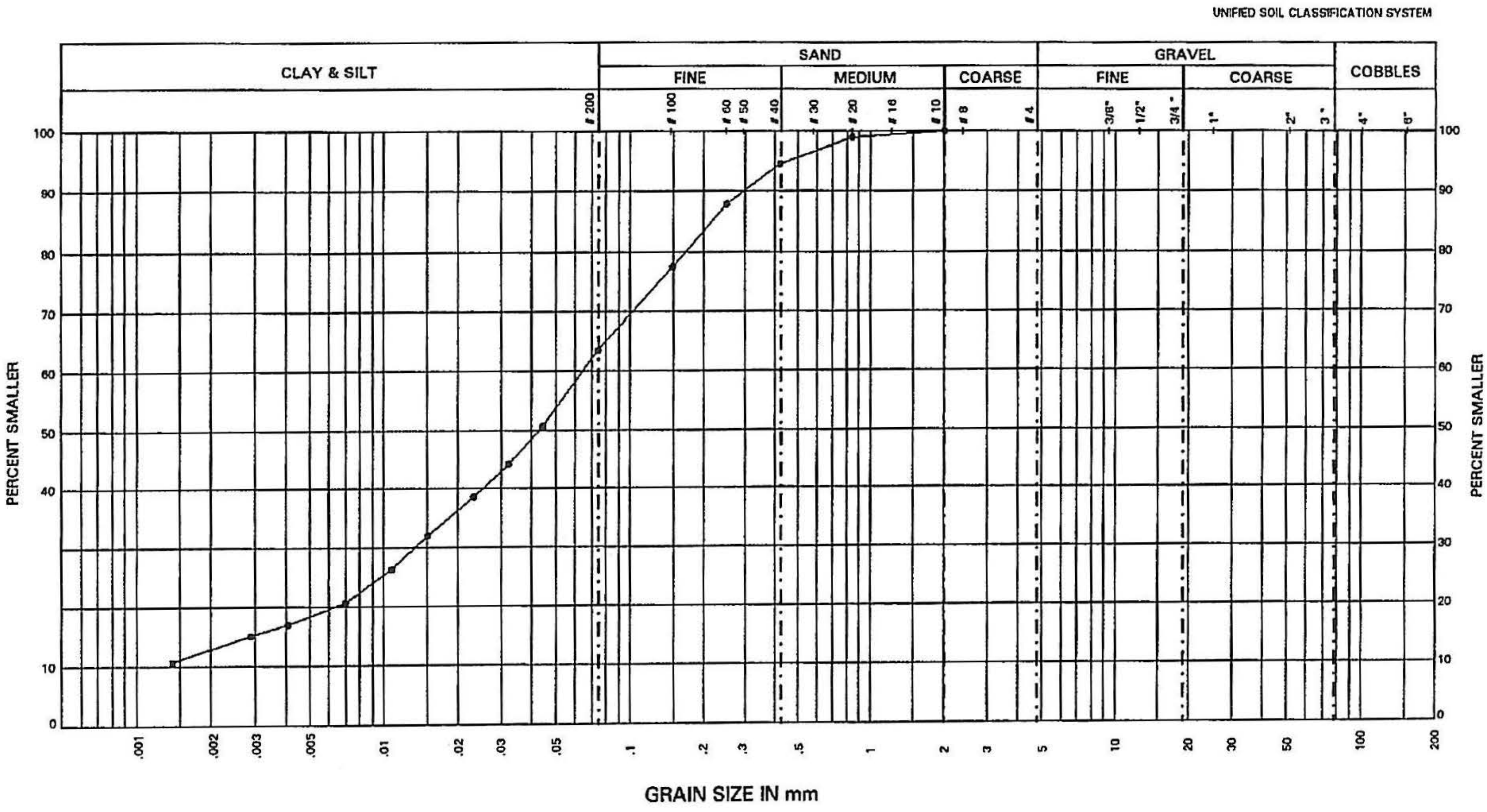
LAB SAMPLE NO.
LAB TEST NO.
DATE December 2003
TESTED BY RS
CHECKED BY AT
APPROVED BY

REMARKS: Harrington Dam
UTRCA

GRAIN SIZE DISTRIBUTION

DEPTH	SAMPLE NO.	HOLE NO.	JOB NO.
6.09m	AS8	BH1	P14504.04

ACRES



LAB SAMPLE NO.

LAB TEST NO.

DATE March 2004

TESTED BY RS

CHECKED BY AT

APPROVED BY

REMARKS: Harrington Dam (Triaxial Test Sample)
UTRCA

GRAIN SIZE DISTRIBUTION

DEPTH	SAMPLE NO.	HOLE NO.	JOB NO.
1.52m to 2.89m	AQ2 to AQ3	BH2	P14504.04

7 Civil/Structural Assessment

7.1 Introduction

Stability analyses were performed using the parameters and the general methods described herein. In performing these analyses, maps and photographs produced during the site inspection phase of the work, as well as site-specific geologic data, were used to assist in the assessment of the structure. These site-specific data obtained during the site visit are described in Section 4 of this report. The results of the stability analyses were used to determine if the Harrington Dam satisfies current draft Ontario Dam Safety Criteria, according to the criteria provided in Sections 6.0 and 7.0 of the draft ODSG. The results from these analyses, together with the results obtained from the various other assessments prepared as part of this study, form the basis of the recommendations for remedial work as detailed in Section 11 of this report.

7.2 Methods of Analysis

The dam safety analyses involved the assessment of the ability of the structure to resist

- sliding at the dam-foundation interface, within the dam and at any plane in the foundation under all loading conditions
- overturning
- overstressing of the concrete dam or foundation.

The analyses were based on 'rigid body' limit equilibrium method with the various load combinations treated as static because of the relatively sustained nature of loads involved.

For critical representative sections of the structures, sliding stability in the upstream-downstream direction, the compressive and bearing stresses in the concrete and the location of the resultant were determined. Where the location, magnitude, direction and duration of computed tensile stresses were such that the stresses would be likely to produce cracking, the extent of cracking was evaluated.

Seismic analyses are typically performed at different levels of sophistication depending on the hazard potential rating of the dam and the probability of unacceptable performance. For the low earthquake potential in southwestern Ontario, pseudostatic methods of analysis are used.

7.3 Selection of Loads and Load Combinations

The following loads were considered in the assessment of the concrete structure:

- dead loads of permanent structures, rock or soil backfill, silt deposited against the structure and any significant equipment loads
- the maximum flood headwater level based on the IDF with corresponding tailwater levels
- internal water pressure and foundation uplift
- static thrust created by an ice sheet
- MDE.

7.3.1 Ice Loads

The thermally driven, static, ice loads used in the design review were assessed by taking into consideration site-specific characteristics and operating information.

For ice loadings, it was assumed that horizontal thrust created by thermal expansion of ice sheets would occur 0.3 m below the headpond level. Research by OPG, Manitoba Hydro, Fleet Technology and others has shown that the magnitude of this ice thrust depends on factors such as the thickness of the sheet of ice, the average ambient temperature, the rate of temperature change in the ice, fluctuations in the water surface, reservoir characteristics and wind drag.

The temperature data required as part of the ice load assessment was established by considering the January 1% temperature (see Table 7.1 for the definition of this term) from the OBC. For the Harrington Dam, the closest geographically available weather station reports were at Stratford and St. Marys. The average January 1% temperature was found to be -20°C.

Table 7.1

Thermal Ice Loads on Concrete Dams

Reservoir Shoreline Characteristics	**Winter Air Temperature (January 1% Temperature* from OBC)**		
	Mild 0° to -20°C	**Average -21° to -29°C**	**Severe -30°C and Lower**
Flat shore (<20° slope)	58.4 kN/m (4 kips/ft)	80.2 kN/m (5.5 kips/ft)	102.1 kN/m (7 kips/ft)
Steeper shore (20° to 45° slope)	73.0 kN/m (5 kips/ft)	87.5 kN/m (6 kips/ft)	116.7 kN/m (8 kips/ft)
Steep rocky shore (>45° slope)	87.5 kN/m (6 kips/ft**)	116.7 kN/m (8 kips/ft**)	145.9 kN/m (10 kips/ft**)

Notes:

1 * The January 1% temperature is defined as the lowest temperature at or below which only 1% of the hourly exterior air temperatures in January occur. The January 1% temperature for selected locations in Ontario is tabulated in the Ontario Building Code (OBC).

2 ** For steep rocky shoreline, careful study of the site-specific condition with regard to the shape of the headpond, snow cover data and temperature records is required to determine the design ice load magnitude, as the ice load can be larger than the values shown in the table.

3 Ice load for steel gates = 50% of the values shown in the table.

4 Ice load for timber logs = 29.2 kN/m (2.0 kips/ft).

5 Ice load reduction where timber crib remains exist at or above the waterline shall be based on the location, top elevation, and flexibility of the subject timber crib structure.

6 Minimum ice load where ice sheet existed against the structure = 29.2 kN/m (2.0 kips/ft).

7 Maximum water level in January from past records (from 30 to 80 years) shall be considered for the 'winter operating condition' in the design review. However, this water level may not be much different from the maximum headwater level given for the summer condition.

8 Site-specific conditions based on the design review inspection shall be used in selecting the appropriate design ice load.

Headpond shoreline characteristics, such as slope, were measured from the topographic details established during the site survey. On the basis of procedures for estimating ice loads presented by OPG at a workshop on ice held at the annual Canadian Dam Association conference in 2000 as detailed in Table 7.1, the resulting ice thrust values were estimated. The results of this assessment showed that the following ice loads should be considered at the Harrington Dam:

ice load on concrete	73.0 kN/m.
ice load on stop logs	29.2 kN/m.

7.3.2 Hydrostatic Uplift

Hydrostatic pressures within the dam and foundation are considered as follows.

- **Case 1:** For dams with no foundation drains or pressure relief systems, full uplift, varying linearly from 100% headwater pressure at the upstream face to 100% tailwater pressure at the downstream face, is assumed to act on the entire base area of the dam.

- **Case 2:** For dams equipped with an effective drainage and/or pressure relief system (based on field investigations and/or monitoring data), reduced uplift is used. The reduced uplift varies from 67% of upstream headwater pressure to 100% tailwater pressure, only if the actual recorded uplift is less.

At the Harrington Dam, Case 1 applies. Due to the presence of the steel sheetpiling, additional analysis using flownets were performed to provide a more accurate estimate of the uplift force and location of the resultant.

The uplift assumption corresponds to the design water levels and does not consider any 'locked in' pressures. If base tensions exceed allowable limits (typically assumed to be one half of the threshold shear strength), it is assumed that tension cracking of the base occurs at the level which allows full uplift pressures to be transmitted along the crack for cases not involving earthquake loadings. In the case of earthquakes, it is assumed that the motions are of such a short duration that uplift pressures will not be increased within any crack that may be theoretically induced from the earthquake loadings.

The uplift assumption corresponds to the design water levels and does not consider any 'locked in' pressures. If base tensions exceed allowable limits (typically assumed to be one half of the threshold shear strength), it is assumed that tension cracking of the base occurs at the level which allows full uplift pressures to be transmitted along the crack for cases not involving earthquake loadings. In the case of earthquakes, it is assumed that the motions are of such a short duration that uplift pressures will not be increased within any crack that may be theoretically induced from the earthquake loadings.

7.3.3 Seismic Loads

Probabilistic earthquake parameters for the damsite was established based on data obtained from the Geological Society of Canada, as summarized in Table 7.2.

Table 7.2

Probabilistic Earthquake Parameters

Peak Horizontal Ground Acceleration (*g*)				
Probability of Exceedance per Year	**0.010**	**0.005**	**0.0021**	**0.001**
Harrington Dam	0.021	0.029	0.039	0.051

These seismic loads were considered to act in a horizontal direction (increasing the driving force) and a vertical upwards direction (decreasing the horizontal resisting force). In the pseudostatic method of analysis, two thirds of the peak ground acceleration (PGA) is used to simulate the sustained ground motion in combination with two thirds of that value acting in a vertically upward direction.

The draft ODSG require that dams

> *". . . be designed and evaluated to withstand ground motions associated with a Maximum Design Earthquake (MDE), without release of the reservoir"*

with the selection of the MDE for a dam being based on the hazard potential classification and consequences of dam failure. As shown in Table 1.3, for any given site, the MDE increases with increasing hazard potential due to dam failure.

For the case of the Harrington Dam, an IHP classification of VERY LOW/ VERY LOW (flood/sunny day) was established. A 1:100-yr earthquake event was selected as the design load case for stability assessment.

7.3.4 Hydrostatic Loads

Water levels used in the assessment of the various load cases were derived for the various load cases based on the IHP classification of the dam and the IDF equivalent to the PMF event. These levels were determined to be as follows:

• normal summer	headwater level	=	330.01 m
	tailwater level	=	326.81 m
• normal fall/winter	headwater level	=	330.01 m
	tailwater level	=	326.81 m
• PMF Flood I	headwater level	=	331.13 m
and PMF Flood II	tailwater level	=	328.40 m.

7.3.5 Load Combinations

The various loading combinations are shown schematically in Figure 7.1 and are described as follows. Numbers in parenthesis refer to the numbers in Figure 7.1.

Usual Loading (1) and (2)

Permanent and operating loads were considered for both summer and winter conditions, including self-weight, ice, silt, earth pressure, and the normal maximum operating water level with appropriate uplift pressures and tailwater level.

Unusual Loading (3)
Where earthquake-induced cracking at the rock concrete interface or any weak section was identified, a stability analysis was carried out to determine the stability of the structure, in its post-earthquake condition, under the effects of the usual loading conditions that could include concurrent ice loadings in areas where appropriate. Full reservoir pressure within the earthquake-induced cracks is assumed for the post-earthquake case.

Flood Loading (4)
Permanent and operating loads of the usual loading case, except for ice loading, were considered in conjunction with reservoir and tailwater levels and uplift resulting during the passage of the IDF. The effect of ice loads was not considered simultaneously with design flood conditions in accordance with the requirements of the draft ODSG.

Flood Loading (5)
For the Harrington Dam, this case is identical to Flood Loading I as the stop logs are not manipulated.

Seismic Loading (6)
Permanent and operating loads from the usual loading were considered in conjunction with the seismic loads that would be generated during the MDE. During this extreme load case, ice loads are also considered. Uplift pressures were assumed to be those corresponding to the normal loadings, and were not modified during the seismic event.

7.4 Performance Indicators

The assessment of the suitability of the concrete structures was based on the following performance indicators:

- position of resultant force
- normal stresses at the heel and the toe

- calculated sliding factors and strength factors.
- overturning factors.

7.4.1 Position of Resultant Force

The draft ODSG indicate that the position of the resultant should be within the middle third of the base for normal loading conditions and within the base for other load cases. Therefore, the intent of the guidelines is that this is a desirable, but not mandatory, requirement for the evaluation of concrete dams. On this basis, dams that satisfy the following conditions:

- existing structure with a history of service and no signs of significant distress
- low incremental consequence category
- satisfy sliding stability criteria
- satisfy compressive strength criteria

were considered to meet the intent of the dam safety requirements even if the position of the resultant was outside the middle third of the base for the normal case.

7.4.2 Tensile Stresses

Within the dam, tensile stresses are acceptable so long as the stresses remain within 0.1 f_c' to 0.05 f_c' (where f_c' is the compressive strength of concrete) within the mass concrete and at lift joints, respectively.

7.4.3 Sliding Factor

The resistance of a gravity dam against sliding on any surface is designed or assessed by comparing the net driving force with its available shear strength. The ratio of these components is the factor of safety (FOS) against sliding or sliding factor (SF).

$$\text{SF} = \frac{\text{Available Shear Strength}}{\text{Net Driving Force}}$$

The draft ODSG recognizes two states of available shear strength: 'peak' and 'residual'.

(a) Peak shear strength is based on the following components:

$$\text{Available Peak Shear Strength } = \sum A_c\{(\sigma_n)\tan(\phi'') + \tau_o\}$$

where,

σ_n = normal stress
ϕ'' = peak angle of internal friction 'a'
A_c = area of compression
τ_o = the available peak shear strength at zero normal stress.

(b) The residual or post-peak strength is defined as

$$\text{Available Residual Shear Strength } = \sum \mathrm{A_c}\{(\sigma_n)\tan(\phi')+\tau_n\}$$

where,

ϕ' = residual angle of sliding friction
τ_n = nominal residual shear strength value at zero normal stress. According to the MNR guidelines, this value may range up to 100 kPa (15 lb/in.^{2}), if supported by tests.[*] Without tests, it is assumed to be zero. For this study, the residual value was assumed to be zero for all structures since no test data was available.

7.5 Acceptance Criteria

Acceptance criteria used in the analysis of concrete structures are as listed in Tables 7.3 to 7.5.

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[*] As discussed, it is not strictly correct to assume a cohesive strength when considering residual shear strength, as concrete-to-bedrock bonds are broken at very small strains. For the so-called 'residual' shear resistance, it is better to consider all bonding to be lost and any apparent cohesion to be a function of roughness.

Table 7.3

Acceptable Sliding Factors for Gravity Dams

Type of Analysis (a) (f)	Load Case			
	Usual	Unusual (Post-Earthquake)	Earthquake (MDE) (b)	Flood (IDF)
Peak sliding factor (PSF) - no tests	3.0	2.0	1.3	2.0
Peak sliding factor (PSF) - with tests (c)	2.0	1.5	1.1	1.5
Residual sliding factor (RSF) (d) (e)	1.5	1.1	1.0	1.3
Concrete strength factor (g)	3.0	1.5	1.1	2.0

Notes:

(a) PSF is based on the peak shear strength. RSF is based on the residual or post-peak strength. See Section 6.4.1 for details.

(b) The stated value under the MDE load case is based on pseudostatic analysis. Performance evaluation of the dam should also take into consideration the time-dependent nature of earthquake excitations and the dynamic response of dam.

(c) Adequate test data must be available through rigorous investigation carried out by qualified professionals.

(d) If PSF values do not meet those listed above, the dam stability is considered acceptable provided the RSF values exceed the minimum.

(e) The minimum values of RSF may be reduced for low hazard potential dams provided data is available to support such reduction.

(f) For low hazard potential dams, if they are judged to be performing satisfactorily, based on an inspection and review of available data, and if conditions are expected to be no less favorable in the future, stability analysis may not be necessary.

(g) These values are recommended where test data is not available.

Table 7.4

Load Cases

Load Case	Description	Minimum FOS	
		No 'Cohesion'	With 'Cohesion'*
1	Normal reservoir loading	1.5	3.0
2	Normal winter reservoir levels + ice	1.5	3.0
3	IDF Flood I	1.3	2.0
4	IDF Flood II	1.3	2.0
5	Earthquake	1.0	3.0
6	Post-earthquake loading	1.1	2.0

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* With no supporting tests.

Table 7.5

Acceptance Criteria

Dam Safety Indicator	Acceptance Criteria
Location of resultant	Within middle third for normal load cases and within the base for all other load cases.
Bearing stresses	Below allowable bearing capacity limits.
Sliding stability	Above minimum requirements for given load case (see above).
FOS against overturning	This parameter was calculated, but there are no specific criteria given in the draft ODSG (acceptance governed by position of resultant).

7.6 Results of Analyses Performed for the Harrington Dam

7.6.1 Assumptions

The Harrington Dam has an overall IHP rating of VERY LOW and is founded on the nonorganic silt and clay layer as discussed in Section 8.2 of this report. Loads were assumed to be as discussed in Section 7.3. Ice loads were taken as 29.2 kN/m on the stop logs, based on the discussion in Section 7.3.1. It should be noted that the location of the ice force (at el 329.71 m), based on a winter water level of 330.01 m, in the spillway bays, was on the stop logs, but only above the concrete sill by 100 mm. Water levels for the various load cases were as given in Section 7.3.4 under Hydrostatic Loads.

As shown on the drawings, the spillway structure has inclined upstream and downstream concrete rollways with small end walls at each end. There is no base slab connecting the upstream and downstream rollways. Accordingly, the section taken for analysis was a 1-m wide strip of the overflow with the critical sliding section along a plane connecting the end walls of the overflow. Sliding would thus be through the foundation material assumed to consist of inorganic silt and clay.

Analyses were performed using the following assumptions:

• foundation material	inorganic silt and clay
• friction angle	30°
• ultimate bearing pressure	0.580 MPa
• cohesion	0 kPa.

The concrete properties were taken as

$f_c' = 20$ MPa
$\gamma_{conc} = 23.50$ kN/m^3.

7.6.2 Discussion of Results

Detailed results of the stability analysis are found in Appendix F and are summarized in Table 7.6.

The results indicate that the overflow structure does not meet acceptance criteria in sliding during the winter condition and during the IDF case. As this is an overflow structure, it is possible that no ice forms against the stop logs in the winter due to constant flow over the logs. This should be verified.

Based on the results of the three boreholes, it has been assumed that the overflow is founded on the inorganic silt and clay. It is possible that the structure is founded on sand or the glacial till which could have angles of

Table 7.6

Stability Results – Harrington Dam

Section	Residual	Peak		Load Case	FOS Against Sliding				Location of Resultant	Minimum Base Friction Angle to Satisfy Sliding Criteria (deg)	Minimum % Bonded Area to Satisfy Sliding Criteria	Notes
					Residual Case		Peak					
	Phi (deg)	c (MPa)	Phi (deg)		Req'd	Actual	Req'd	Actual				
1-m strip of overflow	30	n/a	n/a	Normal	1.5	2.09	3.0	n/a	Within limits	22.6	n/a	1
				Normal with ice	1.5	1.40	3.0	n/a	Within limits	31.7	n/a	6
				Flood I	1.3	1.19	2.0	n/a	Within limits	32.3	n/a	6
				Flood II	1.3	1.19	2.0	n/a	Within limits	32.3	n/a	6
				Earthquake	1.0	1.30	1.3	n/a	Within limits	24.0	n/a	1
				Post-earthquake	1.1	1.40	2.0	n/a	Within limits	24.4	n/a	1

Notes:

uc = unstable crack

Note 1 = dam section satisfies dam safety criteria.

Note 2 = dam section satisfies dam safety criteria under peak strength assumptions.

Note 3 = dam section deemed to satisfy dam safety criteria for low hazard dams [Figure 7.1, Note (f) of the draft ODSG].

Note 4 = bearing stress at toe of dam exceeds criteria.

Note 5 = position of resultant does not satisfy criteria.

Note 6 = does not satisfy dam safety criteria for sliding stability.

Note 7 = rock anchor taken into account.

internal friction of 36° and 38°, respectively, greater than the required angle of 32.3° required for the structure to meet the acceptance factors of sliding. This should be verified during the proposed reconstruction at the dam to increase the discharge capacity.

Even though the structure is classified in the VERY LOW IHP category, the structure does not meet acceptance criteria during the winter and IDF flood conditions. UTRCA has proposed modifications to the dam to increase the discharge capacity at which time the deficiencies associated with the stability of the structure should be resolved.

8 Geotechnical Assessment

8.1 Geology

8.1.1 Regional Geology

The upland terrain is rolling, and relief is about 50 m. The regional physiography has developed as a result of the latest glaciation.

According to government geological mapping (Min. Nor. Dev., 1991; Ont. Div. Mines, 1973), the area is characterized by thick deposits of sediments. These were deposited during the Wisconsin glaciation which occurred in the Pleistocene era.

Silty to sandy silt till, known as the Tavistock Till, with minor clay content, predominates on the upland. Deposits of glaciofluvial sand and gravel outwash and ice contact stratified drift, glaciolacustrine silt and clay, and recent streambed alluvium exist throughout the area. These generally overlie the till. End moraines and eskers are also found locally.

Horizontally bedded sedimentary bedrock underlies the region, but is not exposed.

8.1.2 Site Geology

The dam is located in a rolling, cultivated area. Overburden comprising clay, silt, sand and some gravel forms the ground surface. No bedrock is seen in the area.

According to the drilling by Acres, the dam is founded on layers of clay, silt and sand, overlying sand/silt, and overlying silty glacial till in descending order. An artesian well is located just downstream of the dam. It shows a water level about 1.3 m above the ground.

Exploratory drilling was also done on the right bank close to the dam for the grist mill. This work was done in March 2002 by Atkinson, Davies Inc. (see bibliography). These boreholes showed a sequence of materials which generally correlate with Acres findings.

8.2 Spillway Structure

The spillway consists of a concrete-faced embankment. Deck elevation is 330.53. A short concrete apron is provided on the downstream side. The elevation of the top of this apron is at el 327.421 m. Sheetpiling extends to a depth of 0.8 m below the base of the apron. Shear keys exist at the toe of the upstream concrete face and downstream end of the apron.

8.2.1 Foundation and Foundation Shear Strength

The ground/foundation level of the embankment adjacent to the spillway was found to vary between el 326.37 and el 327.55. As discussed in further detail in Sections 5 and 8.4, the foundation stratigraphy comprises the following layers and materials in descending order. Angles of friction have been estimated for each layer. Zero cohesion has been assumed throughout.

Layers/Material	Angle of Friction (deg)
Silt, clay and sand (topsoil, organic)	25
Silt, clay and sand (nonorganic)	30
Sand/silt	36
Silty glacial till	38

In regards to the spillway, reference to old drawings by Kilborn Engineering Co. Limited in 1950 indicates that the original river/stream channel was incised about 1.5 m below the adjacent valley bottom level.

It appears very likely that the natural surface material, i.e., the organic silt, clay and sand (topsoil) which has relatively low shear strength, has been eroded away in the geological past and that the spillway embankment is, therefore, founded on the stronger nonorganic silt, clay and sand layer.

8.2.2 Bearing Capacity

An ultimate bearing capacity of 580 kN/m^2 was calculated for the spillway foundation assuming silt and clay material and a 30° angle of friction (Canadian Geotechnical Society, 1992).

8.3 Embankment Structure

8.3.1 Cross-Section Geometry

The upstream and downstream slopes of the embankment dam have been surveyed by Global Surveying Services. The resulting cross-section geometry is shown on Drawing 14504-HD-002. The downstream slope is noted to be unusually flat.

8.3.2 Foundation Preparation and Characteristics

There are no records of dam construction and of the foundation preparation. Based on the log of the boreholes, the presence of contaminating organics and topsoil indicates that poor quality materials were not removed prior to placement of the embankment fill.

8.3.3 Shear Strength Parameters

Results of the consolidated undrained triaxial shear strength tests indicated an angle of friction of 36° and zero cohesion for the clay, silt and sand embankment fill material.

The shear strength parameters for the main nonorganic silt, clay and sand foundation material were derived from 'N' values as per Bowles (1996). The 'N' values ranged from 8 to 17; accordingly, an angle of friction of 30° was selected, along with zero cohesion. The uppermost part of this layer, i.e., the surface of the foundation, was found to contain high organic content, particularly in BH-3 (up to 0.9 m). Blow counts ('N') varied from 1 to 5. Hence, it was necessary to downgrade the shear strength from that of the nonorganic equivalent. An angle of friction of 25° and zero cohesion were, therefore, estimated.

The shear strength of the sand/silt layer underlying the silt, clay and sand layer was estimated based on the 'N' value of 23. An angle of friction of 36° was estimated.

The shear strength of the silty glacial till, the lowermost part of the foundation, was estimated based on Acres experience with southern Ontario tills. An angle of friction of 38° and zero cohesion were selected.

8.3.4 Bearing Capacity

The allowable bearing capacity of the foundation is estimated to be approximately 130 kPa (Canadian Geotechnical Society, 1992). The embankment exerts a maximum total pressure of approximately 75 kPa and, hence, the foundation has adequate bearing capacity.

8.3.5 Settlement and Deformation

Harrington Dam exhibited no signs of settlement, indicating no differential vertical movements since construction. Provided the vertical loads are not significantly increased and given the low seismicity potential of the site area, settlement of the embankment fill is not likely to occur in the future. The same applies to the foundation.

Locally, the downstream slope shows 'bulging', i.e., deformation, caused possibly by relatively high groundwater levels in the dam.

8.3.6 Liquefaction

The soils that comprise the embankment and the upper part of the foundation are not considered to be susceptible to liquefaction due to their substantial clay and silt content and well-graded nature (Arumoli et al., 1999). The sand layer and the glacial till are also not considered liquefiable due to their well-graded nature. The low seismicity potential in the site area also reduces the risk of liquefaction to a negligible level.

8.3.7 Seepage and Uplift

The water table in a homogeneous fill dam of this height is normally gently sloping from the reservoir to just above the tailwater. The inspection indicated the downstream slope was soft (mushy) on the left side and groundwater was at the ground surface over a considerable area immediately downstream of the downstream toe. Small 'boil' deposits of silt/sand were noted in this area. This water and the artesian condition of the well downstream of the dam suggest that the sand/silt layer in the foundation may be under artesian pressure.

8.3.8 Instrumentation

The only instrumentation in this dam are the piezometers referred to above. These monitor the phreatic surface. No other instrumentation is recommended.

8.3.9 Embankment Stability

8.3.9.1 Left Embankment

8.3.9.1.1 Location of Section

Stability analyses were done for the left earth embankment. The section location taken for the stability analyses is through the highest portion of the dam at about its midpoint. At this section, the embankment is about 3.5 m high. Figure 8.1 shows the section used in the stability analysis.

8.3.9.1.2 Method of Analysis

Stability analyses were performed according to the limit equilibrium method of slope analysis utilizing the proprietary slope stability software SLOPE/W (GEO-SLOPE International Ltd.). All calculations were based on the effective strength method and analysis was performed according to the Morgenstern-Price method of slices with a half-sine function selected for the interslice force function. Several methods exist to perform slope stability calculations; however, the Morgenstern-Price method was selected since the appropriate factor of safety should be obtained from a slope stability method that satisfies both force and moment equilibrium.

8.3.9.1.3 Material Properties

Table 8.1 describes the properties for the various materials used in the stability analyses.

8.3.9.1.4 Phreatic Surface

It was deemed necessary to consider two piezometric pressure lines, as shown in Figure 8.1. A phreatic surface, deduced from piezometric readings, represents pore pressures within the embankment fill and the clay, silt and sand upper foundation material. The second piezometric pressure line applies to the underlying silt/sand layer and reflects its possible artesian condition (Section 8.3.7).

8.3.9.1.5 Seismic Parameters

The draft ODSG requires that dams withstand ground motions associated with a MDE. The MDE is selected based on the hazard potential classification and consequences of dam failure. In the case of the Harrington Dam, an earthquake event with 1:100-yr return period was selected as the design load case for stability assessment. This selection was on the basis that the dam has a VERY LOW IHP classification.

Probabilistic earthquake parameters for the damsite, up to 1:1000-yr return period, were established based on data obtained from the Geological Society of Canada, and are shown in Table 7.2. The horizontal PGA is 0.021 for the 1:100-yr return period.

The pseudostatic method of analysis requires an equivalent sustained ground motion, and hence, two thirds of the PGA is considered appropriate. A ground acceleration of 0.014*g* was, therefore, applied in the stability analysis.

Table 8.1

Stability Analysis of Earth Embankments

Item	Criteria	Calculated	Comments
General			
IHP		Very Low	
Flood Conditions			
IDF		50-yr flood	
Materials			
Embankment			
- embankment fill (CL)			
cohesion (kPa)		0	
ϕ (deg)		36	
moist unit weight (kN/m^3)		17.8	
saturated unit weight (kN/m^3)		19.0	
Foundation			
- silt (top soil, organics)			
cohesion (kPa)		0	
ϕ (deg)		25	
moist unit weight (kN/m^3)		17.8	
saturated unit weight (kN/m^3)		19.0	
- silt layer (nonorganic)			
cohesion (kPa)		0	
ϕ (deg)		30	
moist unit weight (kN/m^3)		18.5	
saturated unit weight (kN/m^3)		20.3	
- sand layer			
cohesion (kPa)		0	
ϕ (deg)		36	
moist unit weight (kN/m^3)		18.2	
saturated unit weight (kN/m^3)		19.5	
- glacial till			
cohesion (kPa)		0	
ϕ (deg)		38	
moist unit weight (kN/m^3)		18.5	
saturated unit weight (kN/m^3)		20.3	
Loads			
Normal water level (NWL)		330.00	
IDF water level		331.13	
Seismic, horizontal (S_h) PGA (*g*)		0.021*	* 2/3, i.e., 0.014g, was used in pseudostatic analyses
Load Combinations			
Upstream Slope			
Normal (NWL)	1.50	2.01	
Extreme (NWL, S_h)	1.10	1.85	
Extreme (IDF)	1.30	N/A	
Rapid Drawdown	1.20	N/A	
Downstream Slope			
Normal (NWL)	1.50	1.92	
Extreme (NWL, S_h)	1.10	1.78	
Extreme (IDF)	1.30	N/A	
Rapid Drawdown	N/A	N/A	

8.3.9.1.6 Load Cases

Load cases considered for the upstream and downstream slopes in the stability assessment are summarized in Table 8.1. The cases considered are normal, extreme (normal water level with earthquake) and rapid drawdown. However, the rapid drawdown case was deemed as being not applicable to this site based on the discharge facilities available. The case of the IDF was not considered as a load case in the stability analyses, due to the fact that under this condition the dam will be overtopped.

8.3.9.1.7 Results of Stability Analyses

The results of the stability analyses are provided in Table 8.1, together with the acceptance criteria and calculated factors of stability. Figures 8.2 to 8.5 graphically depict the cross sections analyzed and the minimum factors of safety established for both the upstream and downstream sections.

Both upstream and downstream slopes meet the acceptance criteria for all load cases.

8.3.9.2 Right Embankment

The right embankment downstream slope stands at 2H:1V which is considerably steeper than that for the left abutment. The toe of the downstream slope is supported by gabions and shows no seepage, suggesting the right embankment is in a reasonably drained condition. At the time of the site visit, there was geotextile netting in place on the slope surface in order to encourage vegetation and in turn, improve the slope surface stability. Although stability of the slope was not of immediate concern, the factor of safety of the slope was estimated, assuming dry slope conditions.

F.S. = $\tan \phi / \tan \alpha$

where,

ϕ = the angle of friction of the embankment material and was taken as 36°
α = the slope angle.

Appendix C

Acres International Limited Recommendations and Costs

July 2007

11 Recommendations and Costs

As a result of the 2002/2003 dam safety assessment, a number of recommended actions and maintenance activities were identified that are intended to ensure that the structure will satisfy current dam safety criteria within a 20-yr planning horizon. These ranged from routine monitoring to relatively major concrete rehabilitation works. In each case, an attempt was made to prioritize the remedial work requirements.

For each of the recommended issues, prefeasibility level cost estimates were developed based on an assessment of the general scope of work and typical unit price data from similar projects in Ontario. Note that the cost estimates that were developed were made on the basis of the actual estimated direct construction costs for the individual remedial action identified. As details of the contract packaging for a given dam are not known at this time, other costs (such as mobilization, control of water, increased access costs at remote damsites, contingency and engineering costs) were estimated on the basis of a percentage of the contract price according to the general guidelines summarized in Table 11.1.

Table 11.1

Summary of Additional Costs Associated With a Typical Remedial Repair Project

Item	Cost
Mobilization and demobilization	5% to 7% of capital cost
Control of water during construction	3% to 10% of capital cost (can vary significantly depending on complexity)
Barge access	10% to 15% of capital cost
Contingency	15% to 25% of capital cost
Engineering and supervision	8% to 15% of capital cost

In preparing cost estimates for repairing deteriorating concrete, it was generally anticipated that the scope of the repairs would include all of the deteriorated concrete and at least some of the concrete surrounding the repairs. It was usually assumed that, where necessary, the entire pier, upstream and downstream of the gains, would be repaired at one time. The actual timing of the repairs may, of course, vary. For example, it may be cost-effective where the extent of upstream deterioration is relatively minor to undertake these repairs under a separate, smaller contract, at a later date. There was no attempt made to address the timing of repair issues in this report. It is also noted that costs for repairing areas of relatively minor deterioration, that are not considered to require attention at this time, were not developed.

An explanation of the priority numbers and concrete repair classifications are shown in Tables 11.2 and 11.3. Details of the recommended action and associated costs for the Harrington Dam are summarized in Table 11.4. An overall cost summary of the remedial repairs, including allowances for engineering, permitting and environmental costs, is provided in Table 11.5.

Table 11.2

Explanation of Priority Numbers

Priority	Description
1	Immediate - Corrective action required immediately due to safety concerns.
2	High - Corrective action required within 2 years.
3	Medium - Corrective action required within 5 years.
4	Low - Corrective action required within 10 years.
5	Monitoring - Defect should be monitored with corrective action to be taken only when required.

Note: Each level reflects the relative importance or urgency associated with taking some form of action. In cases in which the defects were observed to be safety related (mostly Priority 1 items), action means actual construction. It is noted that some of the Priority 5 items may need to be reassigned a higher priority once the areas have been monitored and investigated and any defects have been identified.

Table 11.3

Concrete Repair Classification

	Description	Area (m^2)	Depth of Repair (mm)	Method
1	Sealing contraction joints (above water)	N/A	N/A	Remove existing cracked caulking by mechanical or other means. Clean joint of dirt and other residue. Apply backer rod if joint is deep. Apply primer. Apply polyurethane elastomeric sealant. Applicable to horizontal and vertical surfaces above waterline.
2	Sealing cracks and contraction joints below waterline	N/A	N/A	Requires diver. Remove existing sealant, if present. Clean joint of algae, etc, by wire brushing. Apply sealant such as Devclad 182 with ethafoam backing rod as required.
3	Bonding cracks (above waterline)	N/A	N/A	Required for structural bonding or to stop water leakage. Use epoxy injection for cracks less than 12 mm, cementitious injection for larger cracks. Where a crack is known to be damp or leaking water, use a water-reactive polyurethane resin.
4.1	Small vertical areas	0 – 2	1 – 50	Remove deteriorated concrete, saw cut, clean, trowel repair mortar
4.2	Horizontal areas	1 – 5	12 – 50	Remove deteriorated concrete, saw cut, pour free-flowing repair mortar
4.3	Large vertical areas	-	12 – 50	Remove deteriorated concrete, saw cut, shotcrete
4.4	Unlimited size vertical surfaces with deep deterioration	-	>75	Chip, saw cut, form and pour concrete. Dowels and rebar may be necessary.
4.5	Vertical areas with exposed rebar	-	12 – 50	Remove deteriorated concrete to 50 mm. Behind rebar, clean rebar of all rust, clean concrete and apply repair material.
4.6	Horizontal overlay with rebar	-	12 – 50	Remove deteriorated concrete to 50 mm. Behind rebar, clean rebar of rust, clean, apply overlay in accordance with manufacturer's directions.
4.7	Large areas of new reinforced facing concrete	-	>150	Roughen old concrete, dowel as required, place new rebar, form and pour concrete
5	Vertical grouting of masonry piers	-	-	Repoint masonry along wall faces. Drill vertically through pier from deck level. Grout using balanced, stable, cement-based suspension grouts to fill all voids and cracks in masonry.

Table 11.4

Estimated Remedial Repair Costs – Harrington Dam

Item No.	Structure	Component	Defect Description	Repair Description	Repair Type	Estimated Quantity	Estimated Construction Cost (2004 $)	Priority	Remarks
1	Embankment	Upstream slope	Some erosion occurring	Install riprap	-	90 m^2	5,000	2	1.5 m x 60 m = 90 m^2
2	Left and right embankments	Crest	Too low	Raise height of crests	-	35 m^3	2,000	1	Crest overtopped during IDF and inadequate freeboard. 0.5 H x 35 L x 2 W = 35
3	Entire dam	-	Lack of signage	Install signs	-	3	1,000	1	Install “Use at Own Risk” signs at each end of dam and “Danger – Keep Away – Fast Water” sign on upstream guardrail.
4	Spillway	Guardrail	Gaps in guardrail do not meet code requirements	Install mesh to reduce openings	-	-	-	1	By UTRCA.
5	Spillway	Low-level outlet	Gate may be inoperable	Repair to make operable	-	-	1,000	2	Contact manufacturer to inspect.
6	Spillway	Deck	Too low	New pedestrian bridge	-	-	5,000	1	Deck needs to be raised by 0.50 m.
7	Spillway	Abutments	Too low	Raise by 0.50 m	-	1 m^3	-	1	6 x 0.3 x 0.5 = 0.9 m^3. Included in Item 14.
8	Spillway	Deck	Excessive deflection	Support grating at closer spacing	-	-	-	2	Included in Item 6 above.
9	Left and right embankments	Adjacent to spillway	Repairs from 2000 flood not completed	Compact material next to spillway	-	-	-	1	Included in Item 2 above.
10	Spillway	Wingwalls and abutments	Freeze-thaw damage	Repair	4.1, 4.2	3 m^3	-	3	Concrete: 20 x 0.3 x 0.5 = 3 m^3. Included in Item 14.

Table 11.4
Estimated Remedial Repair Costs – Harrington Dam – 2

Item No.	Structure	Component	Defect Description	Repair Description	Repair Type	Estimated Quantity	Estimated Construction Cost (2004 $)	Priority	Remarks
11	Spillway	Wingwalls and abutments	Cracks	Bond and seal	3	5 m	-	3	Included in Item 14.
12	Spillway	Wingwalls and abutments	Erosion damage	Chip and trowel repair material	4.1	0.13 m^3	-	3	1.5 x 0.15 x 0.15 + 10 x 0.1 x 0.1= 0.13 m^3. Included in Item 14.
13	Left embankment	Downstream slope	Seepage	Install downstream seepage control	-	350 m^3	50,000	3	Cutoff wall along crest or berm on downstream toe.
14	Spillway	Sluices	Inadequate discharge capacity	Install two overshot gates from Springbank	-	-	70,000	1	Modifications to existing structure. Remote operation included.
15	Spillway	Sluices	Inadequate discharge capacity	Try to reduce the IDF by closer look at basin topographs	-	-	5,000	1	Engineering study using more detailed maps of the area.
16	Spillway	Stilling basin	-	Required downstream of overshot gates	-	-	50,000	1	Required for energy dissipation to prevent erosion.
17	Spillway	Entire structure	Assumed founded on inorganic sill and clay	Additional boreholes	-	2	15,000	2	Should be performed before any remediation work attempted.
18	Right embankment	Downstream slope	Does not meet factor of safety	Perform rigorous stability analysis	-	-	2,500	2	
							206,500		

Table 11.5

Budget Estimate Summary of Construction Costs for Maintenance Repairs for the Harrington Dam

Item No.	Description	Unit	Quantity	Unit Price ($)	Amount ($)
1	Mobilization and demobilization (5%)	LS	1	10,000	10,000
2	Repairs to dam and structures	LS	1	206,500	206,500
3	Control of water during construction	LS	1	20,000	20,000
4	**Subtotal (Construction Costs Without Contingency)**				**236,500**
5	Contingency on Construction Costs (25%)				59,125
6	**Total Estimated Construction Costs**				**295,625**
7	Engineering and Supervision	LS	1	25,000	25,000
8	**TOTAL ESTIMATED COST**				**320,625**

Appendix D

Naylor Engineering Associates Ltd. Report

February 20, 2008 (Project No. 7460G1)

INSPECTION REPORT

No.1

Project: Dam Safety Assessment

Location: Harrington Dam

Job No.: 7460G1

Inspection By: D. Kelly

Date: February 20, 2008

Time: 1:30 p.m.

Purpose/Type of Inspection: Visual inspection of Harrington Dam
Weather: Partly cloudy, -9°C

Inspection Comments:

1. At the time of the site visit the Harrington Dam was inspected. The ground surface was snow covered and the pond was covered with ice except against the stop logs.
2. In general the dam and spillway appear as described in the Dam Inspection Report from November 2002 that is contained in the Acres International Report.
3. It was noted that the ground surface is wet under the snow at the bottom of the downstream left (west) embankment. Three springs were noted.
4. A wet area was also observed in a small ditch that is located west of the dam and pond.
5. Strong flow was observed at the artesian well and the water is clear.
6. It is our opinion that the stability of the left earth embankment is marginal. Based on the results of the boreholes the embankment fill comprises very soft to soft clay, silt and sand with trace wood and organics. The SPT N-values range from 1 to 5 blows per 300 mm which indicated a very loose to loose relative density. The estimated angle of internal friction for this material is 25 to 30° but 36° was used in the geotechnical slope stability analysis that was done by Acres International. In comparison an internal friction angle of 31° was used for similar embankment material at the Embro Dam.

Inspection Comments:

7. If the lower friction angle is used then the embankment stability FS will be less than acceptable. Also the FS will be adversely affected by the springs and artesian groundwater pressure. Overall the safety of the Harrington Dam is questionable and an additional study should be done immediately.

8. The study must involve exploratory boreholes, piezometers, laboratory testing, surveying and geotechnical analysis.

Distribution:

1cc: Upper Thames River Conservation Authority
Attention: Mr. David Williams

Dennis Kelly, P.Eng.

TABLE 1

Harrington Dam
Dam Safety Assessment
Thames River Watershed

ITEM	COMMENTS
Foundation Soil	Alluvial material over sand
Core Material	Clay, silt and sand with organics and wood, loose
Construction Control	None
Design Parameters	None
Rip-Rap Erosion Protection	Upstream side, poor condition
Spillway	Concrete
Conduit Through Dam	None
Emergency Spillway	Right side, poor condition
External Erosion	Yes
Under Seepage	Yes
Artesian Conditions	Yes
Dam Distortion	Yes
Dam Settlement	Not known
Uplift Pressure	Yes