GEOTECHNICAL INVESTIGATION
HARRINGTON DAM EMBANKMENT STABILITY ASSESSMENT
COUNTY ROAD 28
MUNICIPALITY OF ZORRA, ONTARIO
for
UPPER THAMES RIVER CONSERVATION AUTHORITY

7608G1.R01
October 2008

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

DISTRIBUTION:
4cc: Client
1cc: Stratford Office

Naylor
Engineering
Associates Ltd.
CONSULTING ENGINEERS

25 Market Place
Stratford, Ont. N5A 1A4
519-273-0101, Fax 273-7188
www.nayloreng.com

7608G1.R01

October 29, 2008

Upper Thames River Conservation Authority
1424 Clarke Road
London, Ontario
N5V 5B9

Attention: Mr. Rick Goldt

Dear Sir:

Re: Geotechnical Investigation
Harrington Dam Embankment Stability Assessment
County Road 28
Municipality of Zorra, Ontario

Naylor Engineering Associates Ltd. is pleased to submit this report for the geotechnical investigation recently carried out for the above referenced project. The project involves the embankment stability assessment of the Harrington Dam in the Municipality of Zorra, Ontario.

The purpose of the geotechnical investigation was to review the structural integrity of the existing dam embankment and provide recommendations for rebuilding the embankment to meet current dam safety guidelines.

This Geotechnical Engineering Report provides details of the investigation methodology, summary of the subsurface soil and groundwater conditions, results of laboratory tests, engineering analysis, site plans, cross-sections, handhole logs, dam details and photographs.

We believe that this report has been completed within our terms of reference and trust that the information provided herein is sufficient for your present requirements. We would be pleased to be of further assistance during the rebuilding or removal of the Harrington Dam Embankment.

Yours truly,

Dennis Kelly, P.Eng.
Senior Geotechnical Engineer
cs

TABLE OF CONTENTS

Page

Table of Contents
(Continued)

1. Introduction

1.1 Background

Naylor Engineering Associates Ltd. was retained by the Upper Thames River Conservation Authority (UTRCA) to carry out an Embankment Stability Assessment for the Harrington Dam in the Municipality of Zorra, Ontario at the location shown on Drawing 1, appended. This work was authorized in a Contract Document dated May 5, 2008.

The Harrington Dam is located in the town of Harrington West on Harrington Creek, a tributary of Trout Creek, which flows into a reservoir for the Wildwood Dam. The Harrington Dam was built originally in 1846 with reconstruction in 1952. It should be noted that the dam was overtopped twice in summer of 2000 with subsequent repair work performed on the downstream embankment slopes near the spillways.

The Harrington Dam comprises east and west earth embankments separated by a three bay concrete spillway structure. The dam is a small earth dam approximately 90 m long and about 4.0 m high with a head of water of approximately 3.3 m acting across the dam. The dam contains water year round and the freeboard at the dam is approximately 1.1 m.

The purpose of the geotechnical investigation was to assess the geotechncial stability of the Harrington Dam embankment and to provide geotechnical recommendations to upgrade the dam embankment to meet current dam safety guidelines as required.

1.2 Dam Safety Assessment Objectives

A dam safety review according to the Ontario Dam Safety Guidelines (Draft), published by the Ministry of Natural Resources in 1999, involves;

"a phased process beginning with the collection and review of existing information, proceeding to detailed inspections and analyses and culminating with formal documentation."

The objectives of this investigation follow the general provisions stated in the Lakes and Rivers Improvement Act (Ontario Regulation 454/96);

"the protection of persons and of property by ensuring that dams are suitably located constructed, operated and maintained and are of an appropriate nature with regard to the purpose of clauses (a) to (e). 1998, C.18, Sched. I.S.23."

To accomplish this, a systematic evaluation of the dam will include:

- performing detailed site inspections
- classifying the dam based on hazard and flood potential
- assessment of current embankment stability
- assessment of current foundation stability
- assessment of seepage flow through the embankment and foundation
- recommending safe slopes of embankments
- recommending dam retrofitting and continuing maintenance requirements

2. Investigation Procedure

2.1 Previous Work

In October 2002 Acres International Limited (Acres) was retained by the UTRCA and Ausable Bayfield Conservation Authority to undertake an independent dam safety review of fifteen dams and control structures located in the Upper Thames and Ausable/Parkhill basins. An inspection of the Harrington Dam was conducted in November 2002 and the Dam Inspection Report is provided in Appendix A.

A Dam Safety Assessment Report for Harrington Dam was prepared by Acres in July 2007. The fieldwork for the investigation included comprehensive site inspections and condition assessments, and three exploratory boreholes (HT BH1, HT BH2 and HT BH3). Geotechnical laboratory testing consisted of two Atterberg Limits tests, three consolidated undrained triaxial compressive tests and three particle size distribution analyses. The report comprised a hydrotechnical assessment, civil/structural assessment, geotechnical assessment, operations maintenance and safety recommendations. The borehole logs, laboratory test results, and civil/structural and geotechnical assessment are provided in Appendix B of this report. The borehole locations are shown on Drawings 2 and 4, appended.

Acres Recommendations and Costs are provided in Appendix C of this report. The recommendations include: performing stability analyses; additional boreholes; install rip-rap on upstream slope; raise height of crest; install signs; compact material on the right and left embankments adjacent to the spillway; install downstream seepage control; install stilling basin; upgrade sluices to meet adequate discharge capacity; repair spillway by installing mesh to reduce openings in guardrail; a new deck for pedestrian bridge; raise abutments by 500 mm; and reconstruct wingwalls and abutments.

In February 2008 Naylor Engineering Associates Ltd. carried out a visual inspection of the Harrington Dam as requested by the UTRCA. The inspection report and preliminary recommendations are provided in Appendix D.

2.2 Field Program

2.2.1 Exploratory Boreholes and Handholes

The fieldwork for this investigation was carried out on June 10, 11 and 25, 2008 and involved the drilling of four boreholes (Boreholes 1 to 4) to depths ranging from 3.7 to 5.8 m and one handhole (Handhole 1) to a depth of 3.7 m at the locations shown on Drawing 2, appended. The boreholes were advanced with a CME-75 track mounted drillrig equipped with continuous flight solid stem augers supplied and operated by Geo-Environmental Drilling Inc.

Soil samples were recovered from the boreholes at regular 750 mm depth intervals using a 50 mm O.D. split spoon sampler driven into the soil according to the specifications for the Standard Penetration Test (SPT) (ASTM D1586). Vane Shear Tests (VST) (ASTM D2573) and pocket penetrometer tests were performed to assess the shear strength of the cohesive deposits. The VST and pocket penetrometer test results, and SPT N-values recorded are plotted on the borehole logs.

Thin walled (Shelby) tube sampling (ASTM D1587) was carried out to recover a relatively undisturbed sample of the silt and clay.

The handhole was advanced by driving a split spoon sampler using a 31.75 kg weight free-falling 760 mm. The blows to drive the sampler each 150 mm were recorded and have been converted to SPT N-values as indicated on the handhole log.

2.2.2 Piezometers and Monitoring Wells

Piezometers were installed in Boreholes 1 and 2 to determine the hydraulic head of the groundwater at specific stratigraphic levels. The piezometer installations comprised 19 mm diameter pipes with slotted and filtered screens that were surrounded with filter sand. Bentonite seals were provided to separate the screens of the piezometers from the screens of the monitoring wells, as well as seal the boreholes near the ground surface. Details of the installations and groundwater observations and measurements are provided on the borehole logs and the water level measurements are summarized in Table 1.

Four 50 mm diameter monitoring wells were installed in Boreholes 1 to 4 for the purpose of hydraulic conductivity testing. The wells had 0.76 to 1.52 m long screens, which were surrounded with a sand pack. Single well hydraulic response of slug tests were carried out at Boreholes 1 to 4. The slug tests consist of removing a volume of groundwater, then measuring the water level response back to static conditions in the well. The data was analyzed using the methods of Hvorslev and the results are provided on Table 2, appended.

The piezometers and monitoring wells were installed and tagged in accordance with R.R.0. 1990 Reg. 903 as amended to Ontario Reg. 128/03 under the Ontario Water Resources Act. Well records were submitted to the Ministry of Environment and the Owner. A licensed well technician must properly decommission the piezometers and wells within 6 months of last use (water level measurements or sampling).

2.2.3 Surveying

A total station survey was completed by R. J. Burnside & Associates Limited on May 22, 2008. The borehole and handhole locations were surveyed by Naylor Engineering Associates Ltd. The boreholes were located relative to existing site features, and the ground surface elevations are referred to the following temporary benchmark supplied by R.J. Burnside & Associates Limited.

TBM: Top centre of concrete base for east post of gate at location shown on Drawing 2.

Elevation: 50.00 m (assumed local datum)

A survey of the pond bottom near the dam was conducted by Naylor Engineering Associates Ltd. on September 11, 2008. The approximate depths of top of sediment and bottom of sediment are provided on Drawing 3, appended.

2.2.4 Spillway Coring

The concrete at the spillway structure was cored on July 29, 2008 at four locations shown on Drawing 2. The coring was done with a 100 mm diameter diamond bit core barrel. The cores were returned to our laboratory for compressive strength testing and the test results are summarized in Table 3, appended.

The fieldwork was supervised by our geotechnical engineering staff who directed the drilling and coring procedures; conducted SPT, VST and pocket penetrometer tests; documented the soil stratigraphies; monitored the groundwater conditions; installed the piezometers and monitoring wells; and, cared for the recovered soil samples.

2.3 Laboratory Testing

All soil samples secured during this investigation were returned to our laboratory for moisture content tests (ASTM D2216) (LS-701); the results of which are plotted on the borehole logs. The geotechnical laboratory tests carried out on selected samples of the major subsurface soils from this investigation comprised the following:

- one Atterberg Limits test (ASTM D4318) with results summarized in Subsection 3.4.3;
- five particle size distribution analyses (ASTM D422 or C139) with results plotted on Figure 1; and,
- one soil unit weight test (ASTM D2937) with results summarized in Subsection 3.4.4.

It is noteworthy that the particle size distribution analyses were conducted on soil samples from the split spoon sampler that excluded particles larger than 37 mm in diameter.

The soil samples will be stored for a period of four months from the date of sampling. After this time, they will be discarded unless prior arrangements have been made for longer storage.

3. Summarized Conditions

3.1 General

The Harrington Dam is located in the town of Harrington West on Harrington Creek in the Municipality of Zorra, Ontario. The dam and millrace were originally constructed for a water-powered grist mill at the site in 1846. Plans for a new dam and spillway were prepared by R. K. Kilborn and Associates in 1952 after a large section of a pre-existing spillway had been undermined and washed away. Work started in July 1952 and the project was virtually completed by the end of one year.

Harrington Pond has a surface area of approximately 3.0 ha and the dam controls a drainage area of approximately 12 km^2 comprising mostly rolling agricultural land. The dam is located at the north end of the pond where water releases from the dam spillway into Harrington Creek and flows in a northerly direction for approximately 100 m to the twin box culverts at County Road 28. The creek then flows easterly approximately 300 m before entering the Trout Creek which then empties to the Wildwood Lake.

The dam embankment is approximately 90 m long and 4.0 m high with side slopes inclined at between 2.0 and 6.0 horizontal to 1.0 vertical. The head of water acting across the dam is approximately 3.3 m and the freeboard on the pond side of the dam is approximately 1.1 m.

The creek channel immediately south of the dam is about 2.0 m wide and 300 mm deep, and situated within a park area with some bushes and trees along both sides. It is lined with riprap immediately downstream of the dam spillway. Minor erosion was evident along the sides of the creek channel and boulders are visible in the creek bed.

There are houses on both sides of the park. The houses located on the east bank include the old mill and are set several meters above the floodplain. The house on the west side of the park is located close to the same elevation as the floodplain.

Approximately 70 m downstream of the top of the dam there is an artesian well that stands 1.3 m above the ground. The artesian flow from the well discharges through a pipe into the creek near County Road 28. The depth of the well and the source aquifer are unknown. Photographs of the site conditions are provided in Appendix G.

3.1.1 Spillway

The discharge facilities at the dam consist of a three-bay concrete spillway structure with a short concrete apron at the toe of the spillway slope extending the full width of the bays. The spillway has concrete wing walls extending to the apron. It is understood that sheetpiling extends to a depth of 0.8 m below the base of the apron.

A 700 mm ID precast concrete outlet pipe passes through the left abutment and embankment of the spillway of the dam. This pipe was gated and closed at the time of the inspection. The overflow spillway has a trapezoidal concrete section with stop logs on the crest between two sets of steel stanchions. Hydrotechnical aspects of the dam structure are provided in Section 4.3.2 of the 'Dam Safety Assessment Report for Harrington Dam' produced by Acres in July 2007.

The area of contact between the crest of the dam and the spillway shows signs of erosion and undercutting. Depressions exist in the walkway in the immediate vicinity of the spillway probably as a result of overtopping in 2000. Gabions placed on the area of contact between the downstream embankments and the spillway demonstrate minor signs of scour erosion and undercutting.

The results of compressive strength testing carried out on concrete cores taken from the dam spillway are provided on Table 3, appended. The compressive strengths are 37.6 MPa for the west wing wall, 48.7 and 57.1 MPa for the main spillway walls, and 81.9 MPa for the spillway apron.

No emergency spillway was observed but an old concrete millrace is evident at the east embankment. The millrace is partially filled with soil and extends northwards to the mill building.

3.1.2 West Embankment

The west (left) embankment is approximately 65 m in length spanning the distance from the concrete spillway to the grassed and treed park area on the west side of the pond. A raised access road embankment extends from the end of the west embankment to the parking lot in the park.

The dam embankment crest is 2.0 to 3.0 m wide and is vegetated with grass. The crest showed no signs of cracking, sinkholes or settlement at the time of fieldwork.

The upstream (pond side) of the west embankment is protected with numerous cobbles and boulders at the water line leaving the rest of the slope vegetated with grasses and marsh type vegetation. Wave scour erosion has occurred up to 500 mm resulting in irregular slope patterns. No displacement settling or sinkholes were noticed on the upstream slope.

The downstream slope of the west embankment is inclined at between 5.0 and 6.0 horizontal to 1.0 vertical with the bottom half flatter than the top half. The majority of the downstream slope is vegetated with grass with the exception of twenty large trees present at the toe of the dam and a small patch of overgrown brush on the slope.

The downstream slope is soft/wet in many areas indicating active seepage. There is a 5.5×6.0 m area showing bulging which may be caused by active seepage. There are smaller areas of possible leakage present on the west embankment and intermittent ponded water is present at the toe of the dam in the park area.

The area of contact between the embankment fill and the concrete spillway showed signs of erosion, possibly caused by overtopping or seepage.

Typical cross-sections of the dam embankments are shown on Drawing 3.

3.1.3 East Embankment

The east (right) embankment is approximately 15 m in length extending from the spillway to behind the old grist mill. The crest is part of a grassed pathway connecting the dam to Victoria Street. It is 2.0 to 4.0 m wide and shows no signs of cracking, displacement, sinkholes or settlement.

The pond side of the east embankment is protected with cobbles and boulders at the water's edge but there are several trees at the waterline. Wave scour erosion has occurred up to 0.3 m resulting in an irregular slope pattern. No displacement, settling or sinkholes were noted on the upstream slope at the time of fieldwork.

The downstream slope of the east embankment is inclined at approximately 2.0 horizontal to 1.0 vertical. The downstream slope is vegetated with grasses, weeds and small bushes. It is noteworthy that previous attempts to grow more grass were completed by placing geotextile netting. The downstream slope shows no signs of cracking, sinkholes or settlement although they may have been concealed by vegetation at the time of fieldwork. No seepage was seen in the slope.

There is an abandoned millrace channel located on the east embankment that appears to be in the same position as the old concrete trough that previously conveyed flows to the old grist mill. The longitudinal profile of the emergency spillway seems to rise above the concrete spillway deck level, thus preventing any discharge through the channel. Also the channel is partially filled with soil and debris.

3.2 Pleistocene Geology

The Harrington Dam is situated on Harrington Creek which flows north eventually entering Trout Creek and Wildwood Lake. The dam is located within the physiographic region of Southern Ontario known as the Oxford Till Plain. The region is occupied by a drumlinized till plain with glacial meltwater valleys. The dominant soil material is Tavistock Till which is a gritty clayey silt till. Deposits of glaciofluvial sand and gravel, outwash and ice contact stratified drift, glaciolacustrine silt and clay, and recent streambed alluvium and peat exist throughout the area.

The region is underlain by Devonian carbonate formations. The predominant rock type is limestone of the Lucas Formation. The soil cover is approximately 30 m thick at Harrington West and the bedrock is exposed in the river valleys in St. Marys. The bedrock is approximately 400 million years old and was formed in an inland shallow sea.

3.3 Dam Classification

The current dam is approximately 4.0 m high and impounds a total estimate storage of $2.0\times10^4\ m^3$. This classifies the structure as a SMALL dam on the basis of height and a SMALL dam on the basis of storage impounded, and results in an overall classification of SMALL.

The Harrington Dam is classified overall as a VERY LOW incremental hazard potential (IHP) structure for a dam failure during a flood event. There is low potential for loss of life and damage from a dam breach would not inflict major economic or social losses as well as environmental impacts (see *Figure 1-7: Hazard Potential Classification for Dams* in Appendix E). There is one house on the west side of the channel and one house downstream of the Road 28 culvert that is located at or near the same elevation as the floodplain. The houses could be partly inundated if there is a breach in the dam, and/or erosion could occur at the Road 28 culvert.

The size of the dam is governed by a minimum inflow design flood of a 50 year, 3-day summer storm event. The inflow design flood is the largest flood that was selected for the initial design of the dam (see *Figure 4-1: Minimum Inflow Design Flood for Dams* in Appendix E). At this time there have been no large changes in development to justify changing these original classifications.

3.4 Subsoil Conditions

We refer to the appended borehole logs for detailed soil descriptions and stratigraphies; results of SPT, VST and pocket penetrometer testing; moisture content profiles; groundwater observations and measurements; and details of piezometer and monitoring well installations. We also refer to Drawing 3 for geological cross-sections of the subsurface stratigraphy.

In general, the subsurface stratigraphy at the site comprises fill overlying peat, topsoil, sand, sand and gravel, and glacial till. Descriptions of the soil deposits encountered are provided in the following subsections.

3.4.1 Pond Sediment

Sediment was encountered below the pond at thicknesses ranging from 300 to 700 mm on the gentle slope of the embankment. The measurements are provided on Drawing 3, appended.

3.4.2 Fill

Fill material was encountered in all the boreholes and handholes that were drilled on and around the dam. The fill is 1.2 to 4.7 m thick and extends below the termination depth of Handhole 1. The fill typically comprises dark brown silty sand with trace gravel and clay changing to grey silt with some clay and sand. The results of four particle size distribution analyses carried out on samples of the fill are plotted on Figure 1 and are provided in the following table:

Borehole Number	Sample Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
1	2.29-2.79	1	21	67	11
2	0.76-1.22	6	54	34	6
2	2.29-2.74	6	30	50	14
HT BH2	1.52-2.89	0	36	51	13

SPT N-values recorded in the non-cohesive sandy silt fill typically ranged from 3 to 13 blows per 300 mm indicating a very loose to compact relative density. SPT N-values recorded in the cohesive clayey silt fill typically ranged from 1 to 4 blows per 300 mm indicating a very soft consistency. Shear strengths determined in the silt fill ranged from 25 to 100 kPa.

The results of two Atterberg Limits test carried out on a sample of the fill indicates that the deposit has a low degree of plasticity with results provided on the following table:

Borehole Number	Sample Depth (m)	Water Content (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Liquidity Index
2	2.29-2.74	22	14	19	5	1.6
HT BH1	3.05	20	17	29	13	0.2

The moisture content of the fill ranges from 10 to 35% indicating that the deposit is drier than the plastic limit to wetter than the plastic limit or moist to saturated.

3.4.3 Topsoil

Topsoil was encountered beneath the fill in Borehole HT BH3 that was completed at the base of the west earth embankment. The topsoil is 900 mm thick and comprises black silt with some organics.

3.4.4 Peat

Peat was encountered from 3.8 to 4.8 m depth in Borehole 2, and from 1.8 to 2.1 m below existing grade in Borehole 3. Both of these boreholes were drilled on the west embankment of the dam near the spillway. The peat comprises black amorphous peat. The result of a soil unit weight test carried out on a sample containing peat from Borehole 1 indicates a unit weight of 13.4 kN/m^3. The insitu moisture content of the peat is 90% indicating that the peat is saturated.

3.4.5 Sand

Sand was encountered beneath the fill in Boreholes HT BH1, HT BH2 and HT BH3 at depths of 6.1, 6.0 and 1.4 m respectively. Sand was also encountered in Borehole 3 at a depth of 2.1 m. The sand material comprises fine to coarse sand with some silt and gravel, and is 0.6 to 1.6 m thick.

SPT N-values of the sand ranged from 20 to 33 blows per 300 mm penetration of the split spoon sampler, indicating a compact to dense relative density. The moisture content of the sand indicated saturated conditions at the time of fieldwork.

3.4.6 Sand and Gravel

Sand and gravel was encountered beneath the fill in Borehole 4 at depth of 1.2 m. The sand and gravel extends below the termination depth of Borehole 4.

The sand and gravel generally comprises a brown sand and gravel with some silt and trace clay. The results of one grain size distribution analysis carried out on samples of the sand and gravel soil are provided on Figure 1 and show the sample contains 3% clay, 11% silt, 39% sand, and 47% gravel.

The native sand and gravel has a compact relative density, based on SPT N-values of 15 to 24 blows per 300 mm penetration of the split spoon sampler. The natural water contents of the samples of the granular strata ranged from 9 to 20% indicating saturated conditions.

3.4.7 Silt and Clay

Silt and clay was encountered at 4.0 m depth in HT BH2. The deposit is 2.0 m thick and comprises a stiff, tan coloured silt and clay, as described by Acres.

SPT N-values of the silt and clay range from 1 to 14 blows per 300 mm penetration of the split spoon sampler, indicating a very soft to firm consistency.

3.4.8 Glacial Till

Glacial till was encountered beneath the fill, sand or peat in Boreholes 1 to 3, and beneath the sand in Boreholes HT BH1, HT BH2 and HT BH3. The glacial till extends below the termination depths of the boreholes.

The glacial till texture ranges from grey silt with some clay and trace sand and gravel to a brown sandy silt with some gravel. A particle size distribution analysis for a sample of the glacial till is plotted on Figure 1, and shows the sample contains 4% sand, 80% silt, and 16% clay. The presence of cobbles and boulders can always be expected in the glacial till deposits due its deposition process.

SPT N-values recorded in the silt till deposit typically ranged from 7 to 28 blows per 300 mm, indicating a loose to compact relative density. The insitu moisture contents of the glacial till soils range from 9 to 21%, indicating that the deposit ranges from moist to saturated or about the plastic limit to wetter than the plastic limit.

3.5 Groundwater

We refer to the appended borehole logs, and Table 1 for groundwater observations and measurements carried out in the piezometers and monitoring wells.

Groundwater occurs in the dam fill in Boreholes 1 and 2 at Elevation 51.7 m or approximately 1.3 m below the top of the dam embankment. The groundwater level in the fill is approximately 0.2 m below the pond water level (Elevation 51.9 m) and is 3.4 m above the existing creek level on the downstream side of the dam (Elevation 48.3 m).

The horizontal hydraulic gradient is towards the north and wet areas were noted on the north side of the dam at the time of the fieldwork. An upward hydraulic gradient was noted in the lower piezometers at Boreholes 1 to 4, indicating subartesian pressure.

Artesian flow occurs at the well located approximately 60 m downstream from the dam. The depth of the well is not known, but the flow at the well is likely from a confined aquifer below a layer of till.

The hydraulic conductivity of the subsurface soils has been estimated using the single well hydraulic response of slug tests provided on Table 2. The inferred hydraulic conductivity of the fill ranges from 2.7×10^{-6} to 1.3×10^{-5} m/s, and the hydraulic conductivity of the sand and gravel is 1.7×10^{-5} m/s.

4. Dam Structure and Stability

4.1 General

The project involves the geotechnical assessment of the Harrington Dam in the Municipality of Zorra, Ontario. The Harrington Dam and Pond were originally built in 1846 then reconstructed in 1952. The Harrington Dam is a SMALL fill dam approximately 90 m long and 3.0 to 4.0 m wide at the crest. The sides of the dam embankments are inclined at between 2.0 and 6.0 horizontal to 1.0 vertical, and the freeboard on the pond side is approximately 1.1 m. The freeboard is within allowable limits as shown in *Figure 4-2: Minimum Freeboard for Low Hazard Potential Dams* found in Appendix E.

The Harrington Dam embankment comprises silt and sand fill material placed over topsoil, peat, silt, sand, sand and gravel, and native glacial till. Groundwater occurs in the dam fill soil at 1.3 m below the top of the earth embankment.

The embankment dam is approximately 3.5 m high and impounds a total estimated storage volume of 2.0×10^{4} m^{3}. This classifies the structure as a SMALL dam on the basis of height and a SMALL dam on the basis of storage volume impounded. The dam is classified as a VERY LOW incremental hazard potential structure for dam failure during a flood event.

The discharge facilities at the dam consist of a three bay reinforced concrete spillway and a 900 mm low-level precast concrete outlet pipe. There is an overflow spillway present at the top of the main spillway with stop logs and two sets of steel stanchions.

The following subsections of this report contain geotechnical recommendations pertaining to the existing dam including soil strength parameters, bearing capacity, settlement, liquefaction, seepage, uplift and dam stability. A similar geotechnical assessment was carried out by Acres International Limited and we have provided the results of their work in Appendices A, B and C.

4.2 Soil Parameters

Using the results from the exploratory boreholes, slug testing and geotechnical laboratory testing, engineering parameters where determined for the different soil types in and below the dam embankment. These parameters contribute largely to the understanding of the soil characteristics and their subsequent behaviour. Soil parameters pertaining to Harrington Dam are provided in the following table:

Soil Type	Hydraulic Conductivity (m/s)	Cohesion (kPa)	Friction Angle (Degrees)	Unit Weight (kN/m^3)
Fill	1.3×10^{-5} to 2.7×10^{-6}	0	25	18.0
Peat	1.0×10^{-7}	10	20	13.4
Sand & Gravel	1.7×10^{-5}	0	35	21.0
Sand	1.7×10^{-5}	0	33	18.0
Glacial Till	5.5×10^{-8}	5	28	20.3

4.3 Bearing Capacity

The founding soils are not considered to have a suitable bearing capacity for the existing embankments. Soft peat, soft clay, topsoil and/or sand layers with subartesian pressure were contacted beneath the west embankment at Boreholes 2, 3, HT BH2 and HT BH3.

4.4 Settlement

The Harrington Dam embankments showed no cracking, sink holes or settlement at the time of the fieldwork. This may indicate no differential vertical movements have occurred recently but the buried peat layers and very loose fill provide a high possibility of past settlement in the embankments which may not be currently obvious. Due to the low strength of the soils, future creep settlement is probable especially if any new fill is added to the existing embankments.

There is bulging at the downstream embankment on the west side of the dam spillway. This could be caused by high water levels in the embankment in addition to intermittent seepage occurring through the dam.

4.5 Liquefaction

Liquefaction is the process of soil liquefying often inflicting vast damage on the surrounding area. At the Harrington Dam liquefaction of the embankment and subsoil is a concern. The fill has a very loose relative density (SPT N-values of 1 to 4 blows per 300 mm) and the founding soil comprises organic peat, topsoil and soft clay. There is subartesian pressure and the embankment fill is saturated. The Site Classification for Seismic Site Response would be 'E' (which is the minimum) from Table 4.1.8.4.A. of the National Building Code.

4.6 Seepage and Uplift

Seepage and uplift in a dam structure are caused by excessive porewater pressure through the embankment, thus leading to high instability. There are signs of seepage and uplift at the Harrington Dam and there is subartesian pressure in the soil below the dam. An area west of the dam spillway on the downstream embankment slope is locally bulging and possible minor seepage areas along the west embankment were noted. Vegetation including small bushes and trees along the downstream slopes of the dam may conceal additional minor seepage occurring throughout the embankment. It is noteworthy that the dam fill comprises silt and sand that is susceptible to seepage and soil piping erosion, and high water levels were measured in the piezometers installed in the dam berm.

4.7 Results of Stability Analysis

The long-term stability of the dam embankment must meet the requirements of the Canadian Dam Safety Association and Ministry of Natural Resources. In order to evaluate the safety of this relatively homogeneous berm, the engineering properties of the major soil components were estimated as noted in Subsection 4.2.

Stability analyses were carried out using the Slope/W computer program and three different scenarios were evaluated for the dam configuration, as follows:

1. The long term stability of the embankment under full reservoir head.
2. Rapid (i.e. unplanned) drawdown of the reservoir at a rate significantly in excess of the rate at which pore pressures in the embankment fill are able to dissipate.
3. A pseudostatic horizontal seismic load was incorporated into the stability analysis using a seismic coefficient of 0.04g, a conservative value for this area of Canada (Canadian Foundation Engineering Manual, 1992). The results of these analyses are summarized in the following table:

Loading Conditions	Slope	Minimum Factor of Safety	Calculated Factor of Safety
Steady State Seepage with maximum storage pool	Downstream	1.5	0.8 to 2.4
Full or partial rapid drawdown	Upstream	1.2 to 1.3	0.4 to 0.9
Horizontal seismic load	Downstream and Upstream	1.3	0.3 to 2.4

Based on the stability evaluation, it is concluded that satisfactory factors of safety are not maintained for undrained and drained (long-term) cases, and that the embankment has low stability under seismic conditions (*Refer to Figure 6-1: Factors of Safety Static Assessment* in Appendix E and Appendix F for *Geo-Slope Modelling Results*).

4.8 Assessment

The existing dam does not meet current standards and is not considered stable under existing conditions. The problems with the dam are the low strength of the embankment fill, wave erosion that is occurring on the pond side, seepage occurring through the north face of the west embankment, soft foundation soils, and subartesian pressure. The seepage could eventually cause soil piping erosion at the downstream toe of the dam. Remedial/retrofit work on the existing embankments is not recommended because of the low strength and unstable nature of the existing materials. Reconstruction is recommended if economic conditions allow.

5.0 Dam Reconstruction

5.1 General

The project involves the geotechnical assessment of the Harrington Dam embankments in the Municipality of Zorra, Ontario. The Harrington Dam and Pond were built originally in 1846 and reconstructed in 1952. The Harrington Dam is a SMALL earth fill dam approximately 90 m long and 3.0 to 4.0 m wide at the crest.

The existing dam is considered metastable, and does not meet dam safety requirements. The embankment fill is loose and wet, the founding soil is organic and soft, and there is upward groundwater pressure. It is our opinion that remedial work on the existing dam is not a viable option, because it could destabilize the existing materials. Either the dam should be completely reconstructed using new materials and modern engineering principles or it should be decommissioned. The existing dam should not be incorporated into a new dam.

The following subsections of this report contain geotechnical information pertaining to the dam reconstruction including materials, compaction, blanket drains, outlet pipe, emergency spillway, stability of the dam and construction sequence. We would be pleased to prepare a detailed design for the new dam but it is beyond the scope of the current assignment.

5.2 New Dam Construction

It would be recommended that the new dam be designed with a relatively impermeable clay core surrounded by a sand and gravel shell. The new dam could be constructed in the park area on the downstream side of the existing dam (approximately where the existing trees are situated).

The existing dam could be left in place during the construction of the new dam to provide a relatively dry working environment. The existing dam must be closely monitored during construction to ensure safe conditions on the downstream side.

Prior to construction of the new dam the existing organics (topsoil, peat, and fill) must be removed from beneath the plan area of the dam to the level of the native soil between Elevation 48.0 and 49.0 m. The organics are between 1.0 and 2.0 m thick in the park. The organics would have to be removed with a tracked hydraulic excavator because of the wet conditions in the valley. The excavations to remove the organics will extend up to 1.0 m below the existing groundwater table and rapid inflow should be expected. The groundwater may be controlled by positive dewatering and/or temporary drainage ditches to move the water away from the work area.

Following the removal of the organics the subgrade should be inspected by Naylor Engineering Associates Ltd. The purpose of the inspection is to confirm satisfactory subgrade conditions for dam fill placement and compaction.

After preparing the subgrade the new dam may be constructed as a composite type earth fill structure with a relatively impermeable clay core and a sand and gravel shell. The top of the core should be at least 3 m wide to allow for construction traffic. The core fill should be placed in 300 mm thick lifts and compacted with a heavy sheepsfoot roller to minimum 95% standard Proctor maximum dry density (SPMDD). The initial lift will may have to be up to 600 mm thick to ensure trafficability and minimize compaction problems. We recommend that the core be constructed with maximum slopes of about 2.0 horizontal to 1.0 vertical to allow for proper compaction.

Distribution of material throughout the core shall be such that the fill is free of lenses, pockets, or layers of material differing substantially in texture or gradation from the surrounding material. The dam must not be constructed during freezing weather and frozen material shall not be used.

The shell material should be compacted to minimum 95% SPMDD and sloped at an inclination of at least 4.0 horizontal to 1.0 vertical in order to lessen the potential for erosion. The shell material should comprise imported sand and gravel meeting defined drainage characteristics to prevent the migration of fines during drawdown. The drainage characteristics of the core and shell materials must be tested prior to use to confirm that the materials are suitable for use in the new dam.

The foundation for the earth dam will comprise native sand, sand and gravel or silt till that is suitable to support a 5 m high embankment structure without undergoing shear failure. (All fill, topsoil, peat and soft clay will be removed). The load on the native mineral soil material below the center of the new dam would be approximately 100 kPa. The native mineral soil may undergo minor compression settlement beneath the dam but the settlement is expected to be less then 50 mm. The factor of safety against sliding for the embankment constructed on the native soils is greater than 2.0. Also, since the resulting force acts within the middle third of the embankment, adequate safety against overturning is ensured.

The new dam will impound water for long time periods and therefore steady state seepage conditions will exist below and within the dam. The subsurface conditions underlying the dam structure will comprise permeable sand and sand and gravel overlying relatively impermeable silt till. The vertical hydraulic gradient in the granular deposits is upwards. The hydraulic conductivity of the native granular soil is about 10^{-3} cm/s and the calculated exit velocity at the downstream end of ultimate dam core will be in the order of 10^{-4} to 10^{-5} m^3/s per m of length. In order to control this seepage and prevent piping erosion at the downstream toe of the dam, it is recommended that a clay key or a blanket drain be installed. The blanket drain should outlet at a sufficient distance from the dam to prevent erosion.

The finished slopes on the north side must be topsoiled and vegetated and the south slopes must be covered with rip-rap to minimize surface erosion. Some routine maintenance of the dam surfaces will likely be required to address minor long term weathering and erosion.

It is recommended that additional exploratory boreholes be drilled if a new dam is to be constructed at the site. The boreholes would be drilled along the alignment of the new dam and would extend at least 2.0 m into the basal till deposit.

5.3 Outlet Pipes and Weir Structure

It is anticipated that the new dam would have a spillway crest at Elevation 51.9 m (current pond level) and the outlet pipe will exit north of the dam.

The outlet pipe and weir structure must be carefully designed for the flows expected. The backfill should comprise clay placed in intimate contact with the complete circumference of the outlet pipe and the concrete. In places where proper compaction may be difficult to achieve lean concrete backfill should be used. We also recommend seepage collars be provided on the pipe to reduce the risk of piping along the pipe/soil interface. The pipe should be placed before construction of the clay core, not cut in afterwards.

If an outlet headwall structure is proposed, then the support for this structure must be derived from the native glacial till deposits encountered at approximately Elevation 47.5 m at Boreholes 2 and 3. An allowable bearing pressure of 150 kPa is available in this deposit. The headwall and wingwalls should be backfilled using free-draining granular material and may be designed using an active earth pressure coefficient of 0.35 and a unit weight of 21 kN/m^3. Any footings must be protected with a minimum 1.2 m of earth cover or equivalent insulation to provide protection against potential frost damage (concrete headwall as per OPSD 804.030).

The top profile of the earth berm will include an overflow channel to handle the largest regional storm expected. The overflow channel should be lined with concrete or rip-rap sized depending on the velocities expected (rip-rap treatment as per OPSS 511 and OPSD 810.01. The rip-rap must be pre-approved and comprised of well-graded good quality angular broken rock placed carefully to form an interlocking surface. The rip-rap must be placed over filter fabric conforming to OPSS 1860 for geotextile.

5.4 Construction Sequence

Based on our understanding of the project and the subsurface soil and groundwater conditions, the following construction sequence for the Harrington Dam Reconstruction work is suggested:

- lower pond levels as much as possible prior to construction;
- excavate topsoil and peat from below new dam area;
- install dewatering system;
- prepare subgrade founding soil;
- construct berm core with imported clay fill as required to achieve minimum 2.0 horizontal to 1.0 vertical slopes;
- install blanket drain;
- construct berms shell with imported sand and gravel as required to achieve minimum 4.0 horizontal to 1.0 vertical slopes;
- place filter cloth and rip-rap protection as required;
- construct spillway;
- cover downstream side with topsoil;
- conduct a condition survey of the completed berm;
- remove all existing dam materials and concrete spillway;
- monitor berm during reservoir fill; and,
- monitor outlets and conduct berm inspections after construction (*see Figure 3-1: Minimum Suggested Frequency for Dam Safety Review and Maintenance Inspection* in Appendix B).

5.5 Construction Inspection and Testing

Geotechnical inspections and insitu density testing must be conducted during new dam construction in order to verify that all organic materials have been stripped from the subgrade and to ensure that all fill materials meet the specifications and are being adequately compacted. Naylor Engineering Associates Ltd. should be represented on-site at all times during reconstruction of the dam.

Appropriate laboratory and field testing of the dam components must be conducted during all phases of construction. The laboratory testing should be carried out by Naylor Engineering Associates Ltd.

5.6 Material Quantities

For the recommended dam remedial work, material quantity estimates were developed based on an assessment of the general scope of work. As details of the final design are not known the quantities should be considered approximate.

Item	Quantity
Topsoil and Peat Removal	3300 m^3
Earth Removal	880 m^3
Berm Core Fill (Clay)	10000 m^3
Berm Shell Fill (Sand and Gravel)	10000 m^3
Blanket Drain Pipe	110 m
Blanket Drain Sand Fill	1500 m^3
Filter Cloth	1600 m^2
Wire Mesh	390 m^2
Rip Rap	500 m^3
Existing Dam Removal	4000 m^3
Topsoil	300 m^3

It should be noted quantities above are expected to vary substantially depending on final design, and do not include the outlet pipe and weir.

6. Dam Decommissioning

A dam decommissioning is relevant according to the Canadian Dam Association – Dam Safety Guidelines when;

"A dam has reached the stage in its life cycle when both its construction and its intended use have been permanently terminated in accordance with a decommissioning plan"

The feasibility of the dam decommissioning as a management option should be discussed when financial and/or environmental loss becomes too great to justify further operations of the dam. In order to decommission any dam a plan must be approved by Lakes and Rivers Improvement Act and other Regulatory Ministry involvement.

To accomplish a decommission of a dam the owner must complete the following as outlined by the Ontario Dam Safety Guidelines (DRAFT 1999):

- prepare a detailed plan for withdrawal of the dam from service;
- indicate measures necessary for site safety;
- check possibility of exposure to remaining structures to loads/combination of loads not foreseen in the original design;
- ongoing surveillance and maintenance should be determined prior to decommissioning;

A detailed plan should include the following aspects:

- consequences on downstream development including operation and safety of downstream dams and reservoirs;
- physical characteristics of sediment that has accumulated in reservoir upstream of the dam (i.e. grain size, depth of material, and volume);
- sediment transport in both long and short term;
- how to empty the reservoir before demolition;
- chemical stability before, after and during demolition;
- historical and/or archeological impact;
- permitting requirements;
- flood protection of local infrastructure; and,
- preliminary cost estimate.

At the Harrington Dam the decommissioning work would involve some sediment removal, rechannelizing of the creek in the former pond and downstream of the dam, removal of the spillway structure, full or partial removal of the embankment soil and land restoration. Off-line ponds could be constructed as part of the decommissioning. The work must ensure a sustainable creek and floodplain complex after the dam is removed.

7. Conclusions

The subsurface conditions at the site have been investigated by means of borings, monitoring wells, piezometers, and geotechnical laboratory tests. On the basis of the results, the following conclusions can be drawn:

1. The dam at Harrington Pond comprises loose silt and sand fill over peat, topsoil, clay, silt, sand, sand and gravel, and till.

2. Groundwater was measured within the fill in the dam and there is subartesian pressure in the soil below the dam.

3. The Harrington Dam Embankment does not meet current standards, and is not considered stable under existing conditions.

4. The dam is not considered suitable for retrofit work.

5. A new dam and spillway would be required but the financial cost would be significant.

6. Dam decommissioning is a viable option due to significant financial cost for a new dam construction and future operation.

It is important to know that the geotechnical investigation involved a limited sampling of the site gathered at specific testhole locations and the conclusions in this report are based on the information gathered. The subsurface conditions between and beyond the testholes will differ from those encountered at the testholes. Should subsurface conditions be encountered which differ materially from those indicated from the testholes we request that we be notified in order to assess the additional information and determine whether or not changes should be made as a result of the conditions.

Respectively submitted,

Montana Brown, B.Sc.

LICENSED PROFESSIONAL ENGINEER
D.J. KELLY
Oct 29 2008
PROVINCE OF ONTARIO

Dennis Kelly, P.Eng.
Senior Geotechnical Engineer

LIST OF ABBREVIATIONS

The abbreviations commonly employed on the borehole logs, on the figures, and in the text of the report, are as follows:

Sample Types	
AS	auger sample
CS	chunk sample
RC	rock core
SS	split spoon
TW	thin-walled, open
WS	wash sample

Soil Tests and Properties	
SPT	Standard Penetration Test
UC	unconfined compression
FV	field vane test
ø	angle of internal friction
γ	unit weight
w_p	plastic limit
w	water content
w_l	liquid limit
I_L	liquidity index
I_p	plasticity index
PP	pocket penetrometer

Penetration Resistances	
Dynamic Penetration Resistance	The number of blows by a 63.5 kg (140 lb.) hammer dropped 0.76 m (30 in.) required to drive a 50 mm (2 in.) diameter 60 ° cone a distance 0.30 m (12 in.). The cone is attached to 'A' size drill rods and casing is not used.
Standard Penetration Resistance, N (ASTM D1586)	The number of blows by a 63.5 kg (140 lb.) hammer dropped 0.76 m (30 in.) required to drive a standard split spoon sampler 0.30 m (12 in.)
WH	sampler advanced by static weight of hammer
PH	sampler advanced by hydraulic pressure
PM	sampler advanced by manual pressure

Soil Description

Cohesionless Soils	**SPT 'N' Value**	**D_r (%)**
Relative Density (D_r)	(blows per 0.30 m)	
Very Loose	0 to 4	0 to 20
Loose	4 to 10	20 to 40
Compact	10 to 30	40 to 60
Dense	30 to 50	60 to 80
Very Dense	over 50	80 to 100

Cohesive Soils	Undrained Shear Strength (C_u)	
Consistency	**kPa**	**psf**
Very Soft	less than 12	less than 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very Stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

DTPL	Drier than plastic limit
APL	About plastic limit
WTPL	Wetter than plastic limit

TABLE 1

GROUNDWATER LEVEL MEASUREMENTS

Harrington Dam Embankment Stability Assessment
County Road 28
Township of Zorra, Ontario

Borehole Number	Ground Elevation (m)	June 17, 2008		June 23, 2008		June 25, 2008		July 25, 2008	
		Groundwater Depth (m)	Groundwater Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)
BH 1 upper	53.05	1.27	51.78	1.34	51.71	1.39	51.66	1.40	51.65
BH 1 lower	53.05	1.45	51.60	1.39	51.66	1.44	51.61	1.65	51.40
BH 2 upper	52.91	1.27	51.64	1.33	51.58	1.32	51.59	1.32	51.59
BH 2 lower	52.91	2.29	50.62	2.16	50.75	2.20	50.71	2.28	50.63
BH 3	50.12	0.66	49.46	0.81	49.31	0.79	49.33	0.40	49.72
BH 4	50.39	0.33	50.36	-0.04	50.43	0.09	50.30	0.09	50.30

Notes:

Elevations referenced to TBM supplied by R.J. Burnside Associates Ltd.

TBM: Top centre of concrete base for the east fence gate post at location shown on Drawing 2.

Elevation: 50.00 m (assumed)

Pond water level at Elevation 51.91m at time of fieldwork

Creek water level at Elevation 48.13 m

Naylor Engineering Associates Ltd
CONSULTING ENGINEERS

TABLE 2

SUMMARY OF SLUG TEST ANALYSIS

Harrington Dam Embankment Stability Assessment
County Road 28
Municipality of Zorra, Ontario

Monitoring Well Number	Screen Depth (m)	Soil Type	Hydraulic Conductivity (m/s)
1	1.98-2.74	Fill	2.7×10^{-6}
2	1.22-2.75	Fill	1.3×10^{-5}
3	2.13-3.05	Silt Till/Sand	5.5×10^{-8}
4	1.52-3.05	Sand and Gravel	1.7×10^{-5}

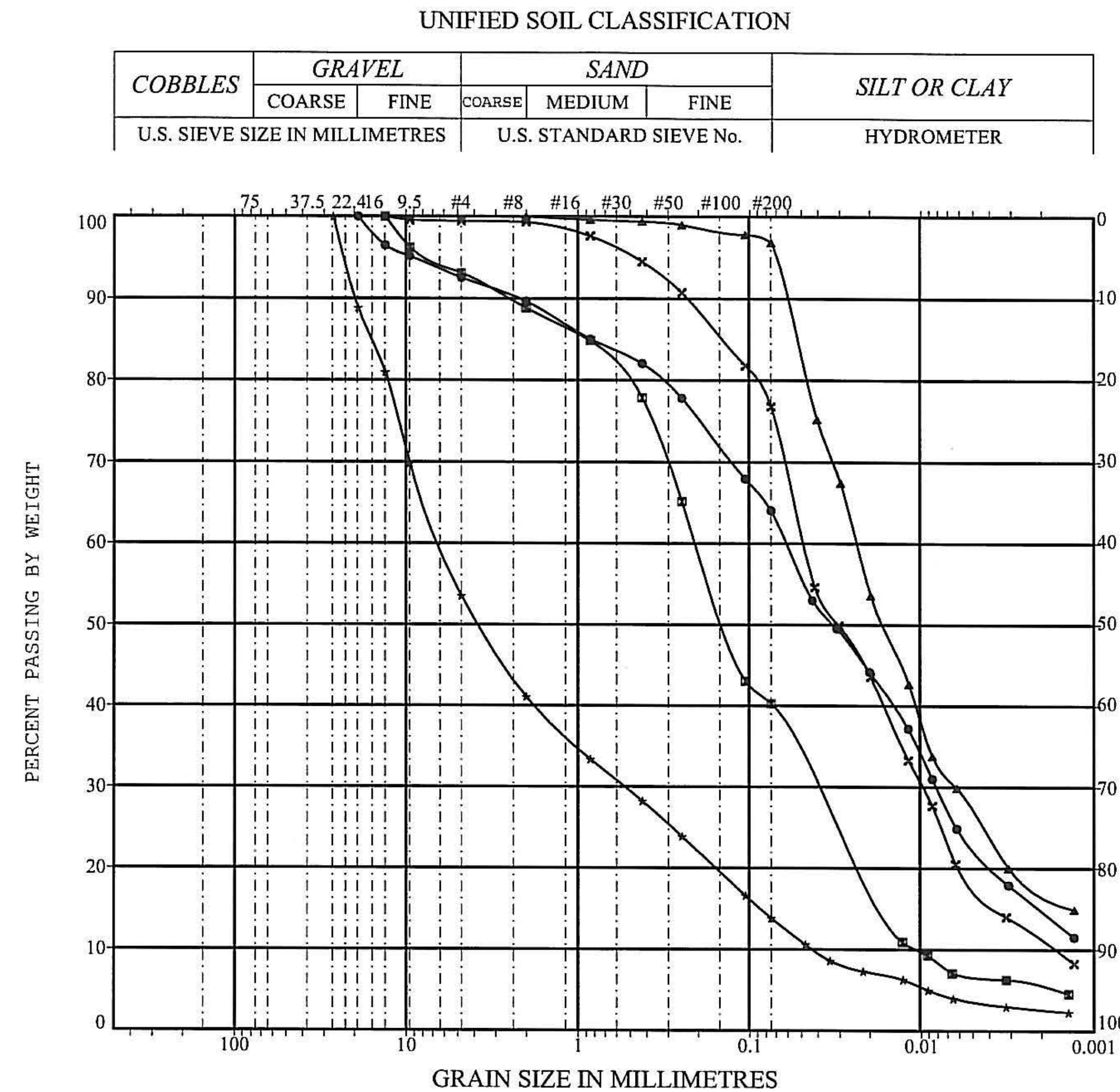
TABLE 3

CONCRETE CORE COMPRESSIVE STRENGTH TEST RESULTS

Harrington Dam Embankment Stability Assessment
County Road 28
Harrington, Ontario

Core Number	Core Location	Density (kg/m^3)	Strength * (MPa)
1	Top of east wall of spillway	2428	57.1
2	Top of west wall of spillway	2358	48.7
3	Top of west wingwall	2299	37.6
4	Spillway apron	2460	81.9

Notes: Concrete cored on July 29, 2008.
* Corrected compressive strength of 100 mm diameter cores.



PROJECT Harrington Dam Embankment Stability Study

LOCATION County Road 28, Harrington, Ontario JOB NO. 7608G1

CURVE ID	BOREHOLE/ TEST PIT	SAMPLE NO.	DEPTH (m)	SOIL DESCRIPTION
●	BH2	Sa4	2.29-2.74	Sandy SILT, some Clay, trace Gravel
⊠	BH2	Sa2	0.76-1.22	Silty SAND, trace Gravel and Clay
▲	BH2	Sa8	5.33-5.79	SILT TILL, some Clay, trace sand
★	BH4	Sa4	2.29-2.74	SAND and GRAVEL, some Silt and trace Clay
✖	BH1	Sa4	2.29-2.74	Sandy SILT, some Clay

REMARKS

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Figure No. 1

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Borehole Number: 1

Ground Elevation: *53.05 m*

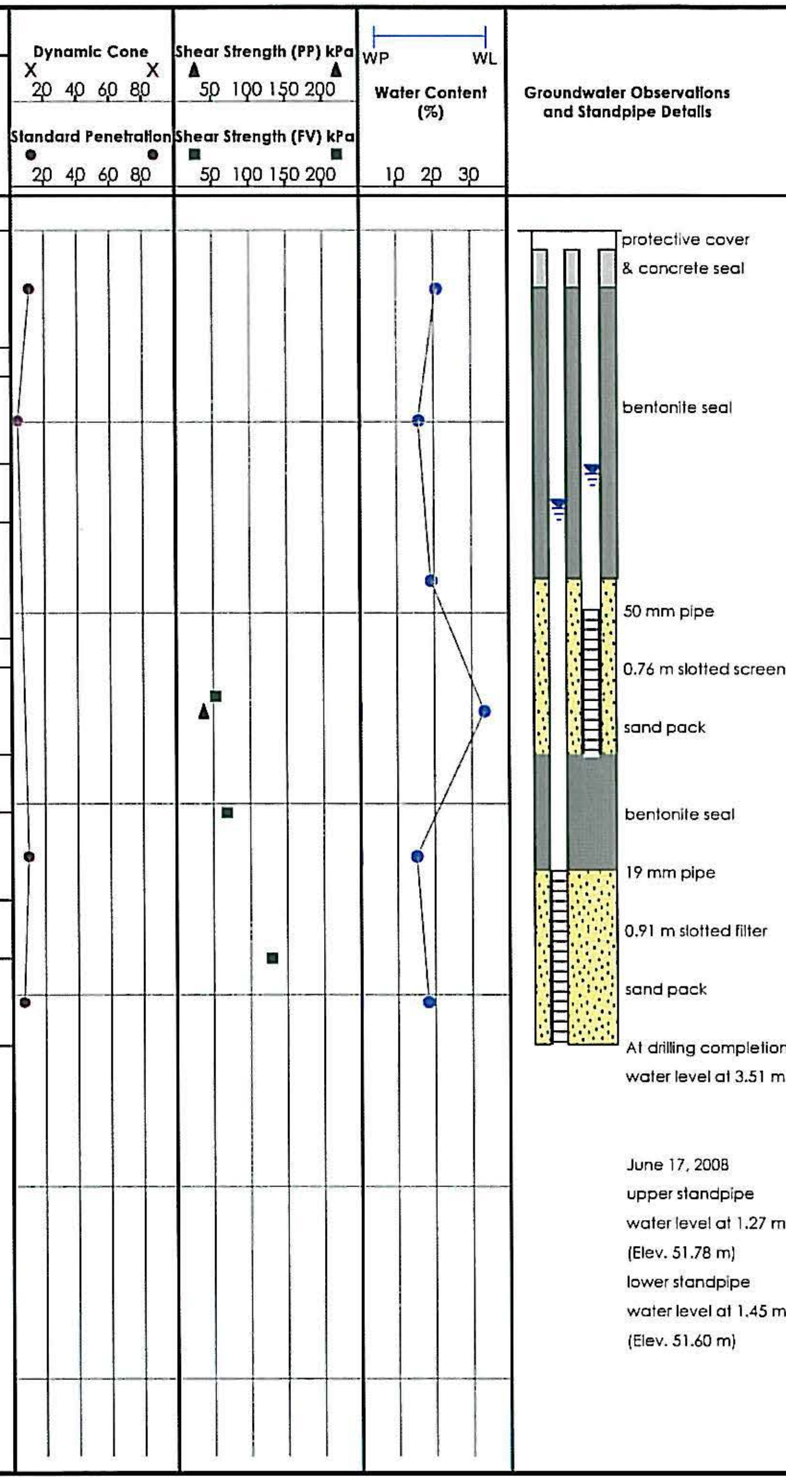
Project: *Harrington Dam Embankment Stability Assessment*

Location: *County Road 28, Harrington, Ontario*

Job No.: *7608G1*

Drill Date: *June 10, 2008*

Depth (m)	SOIL PROFILE – Description	Symbol	Elevation (m)	SAMPLE – Number	Type	N-Value
0.00	Ground Elevation		53.05			
	FILL: dark brown silt (topsoil), wet					
	loose to compact brown silt, trace sand and clay, very moist			1	SS	11
1.00	very loose brown fine to medium sand, some silt, very moist		52.00	2	SS	4
	soft brown clayey, silt, some topsoil, WTPL			3	SS	*
2.00			51.00			
	dark brown sandy silt, some clay, saturated			4	TW	
3.00			50.00			
	SILT TILL: stiff grey silt, some clay, trace sand and fine gravel, APL			5	SS	10
4.00	some silt and sand layers, saturated		49.00	6	SS	7
	Borehole terminated at 4.27 m					
5.00			48.00			
6.00			47.00			



Reviewed by: *DK*

Drill Method: *Hollow Stem Auger*

Notes: **Sampler fell under weight of hammer*

Field Tech.: *RM*

Sheet: *1 of 1*

Drafted by: *AP(01a)*

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Borehole Number: 2

Ground Elevation: *52.91 m*

Project: *Harrington Dam Embankment Stability Assessment*

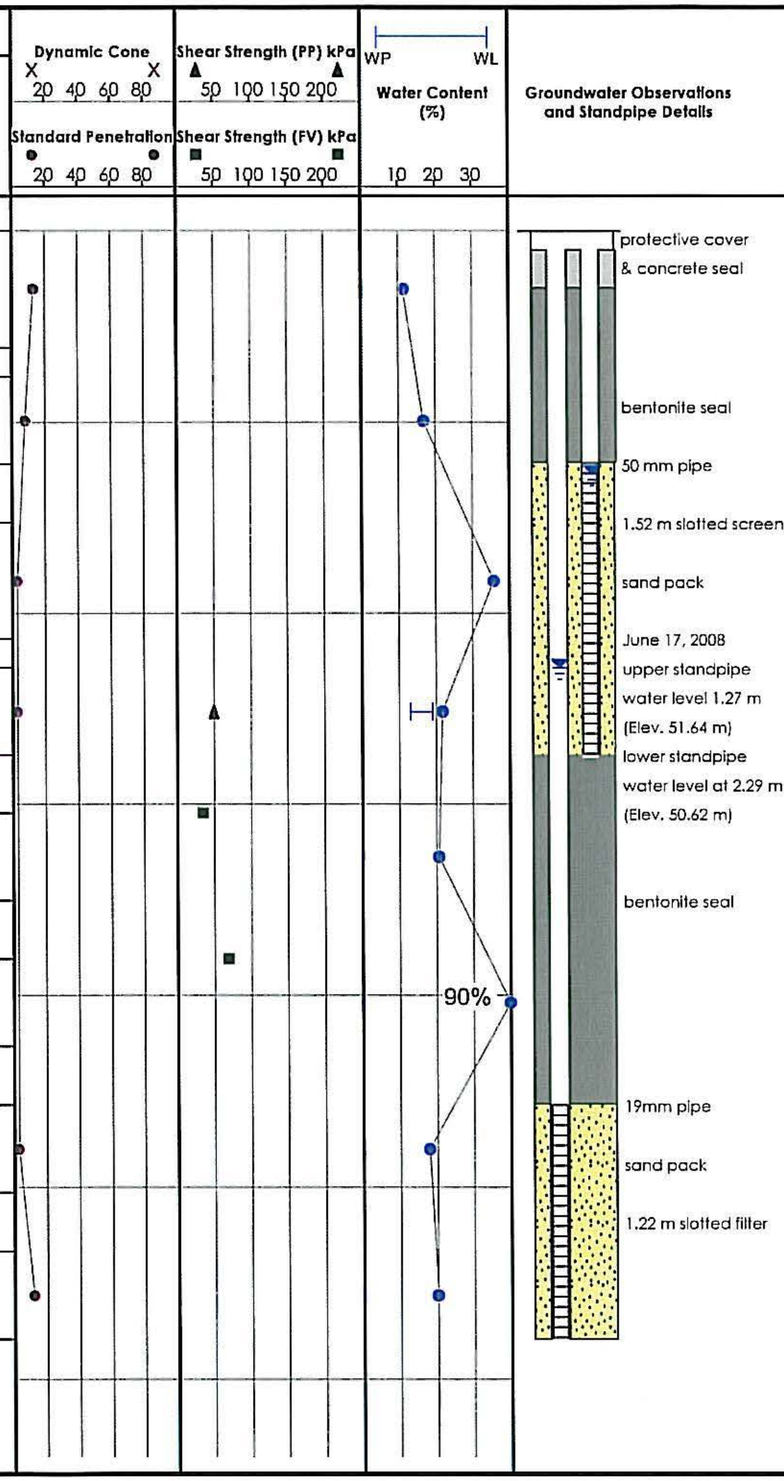
Location: *County Road 28, Harrington Ontario*

Job No.: *7608G1*

Drill Date: *June 11, 2008*

Depth (m)	SOIL PROFILE – Description	Elevation (m)
0.00	Ground Elevation	52.91
	FILL: dark brown silt (topsoil), some brown silt and sand, very moist	
	compact brown silt, trace sand and clay, very moist	
	loose grey/brown silty fine to medium sand, trace gravel and clay, saturated	
1.00		52.00
	soft black topsoil and grey silty clay, WTPL	
2.00		51.00
	soft grey sandy silt, some clay, trace gravel, WTPL	
3.00		50.00
	soft grey silt, some clay and sand, wet	
	PEAT: black amorphous peat, WTPL, wood	49.00
4.00		
	SILT TILL: firm to stiff grey silt, some clay and trace sand, WTPL	
5.00		48.00
	Borehole terminates at 5.79 m	
6.00		47.00

SAMPLE Number	Type	N-Value
1	SS	13
2	SS	8
3	SS	3
4	SS	3
5	TW	*
6	SS	*
7	SS	3
8	SS	12



Reviewed by: *DK*

Drill Method: *Hollow Stem Auger*

Notes: **Sampler fell under weight of hammer*

Field Tech.: *RM*

Sheet: *1 of 1*

Drafted by: *AP(01a)*

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Borehole Number: 3

Ground Elevation: *50.12 m*

Project: *Harrington Dam Embankment Stability Assessment*

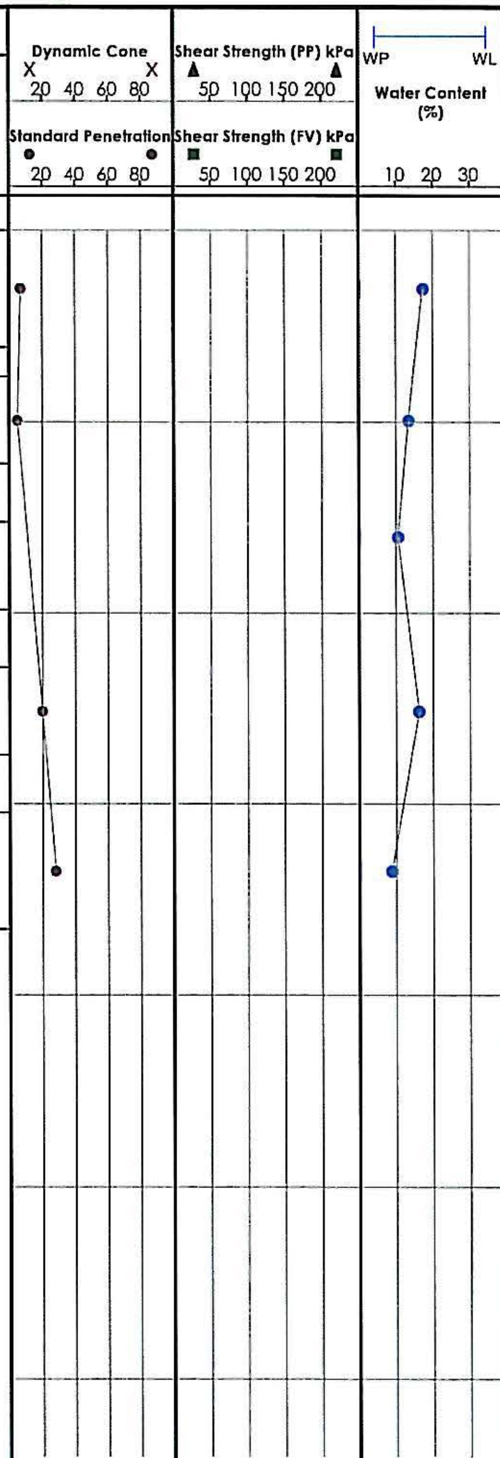
Location: *County Road 28, Harrington Ontario*

Job No.: *7608G1*

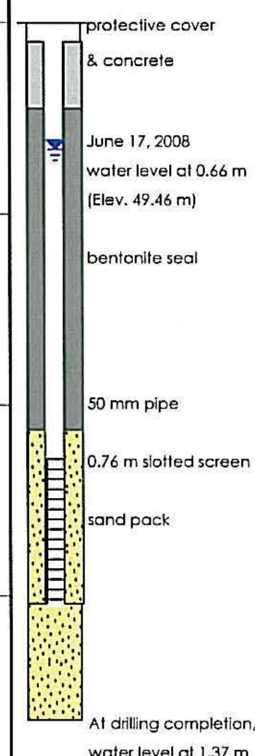
Drill Date: *June 11, 2008*

SOIL PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Number	Type	N-Value
	Ground Elevation	50.12			
0.00	**FILL:** dark brown silt (topsoil), some brown silt, sand and gravel, very moist		1	SS	7
	loose brown silt, some clay, sand and gravel, moist				
	occassional boulders				
1.00	loose dark brown silt (topsoil), some sand, gravel and pieces of brick, moist	49.00	2	SS	5
	some black silty sand, saturated		3	SS	*
2.00	**PEAT:** brown fibrous peat, saturated	48.00			
	SAND: compact grey fine to coarse sand, some silt and gravel, saturated		4	SS	20
3.00	**SILT TILL:** compact brown sandy silt, some gravel, moist	47.00	5	SS	28
	Borehole terminates at 3.66 m				
4.00		46.00			
5.00		45.00			
6.00		44.00			



Groundwater Observations and Standpipe Details



Reviewed by: *DK*

Drill Method: *Hollow Stem Auger*

Notes: **Sampler bouncing on wood*

Field Tech.: *RM*

Sheet: *1 of 1*

Drafted by: *AP(01a)*

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Borehole Number: 4

Ground Elevation: *50.39 m*

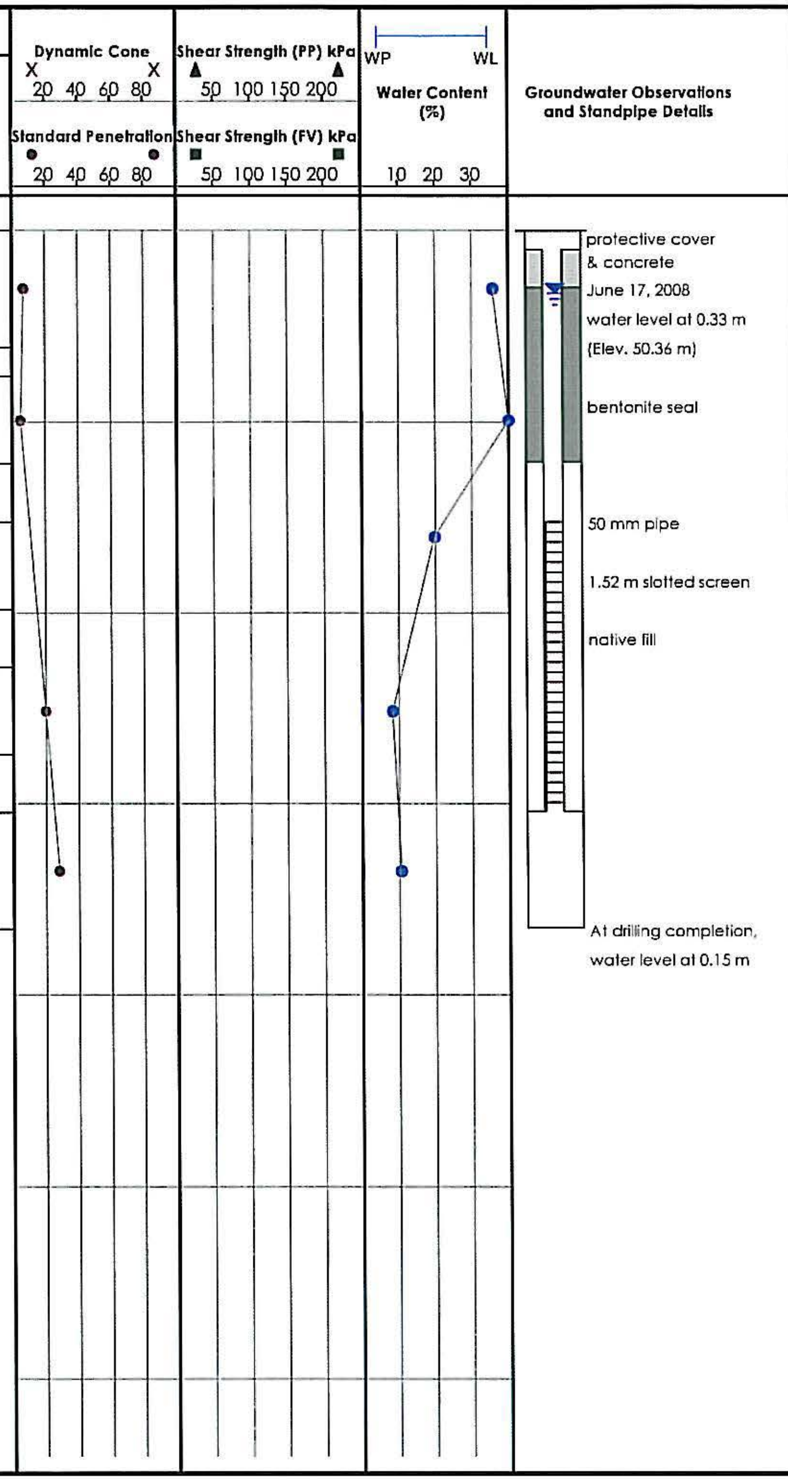
Project: *Harrington Dam Embankment Stability Assessment*

Location: *County Road 28, Harrington Ontario*

Job No.: *7608G1*

Drill Date: *June 11, 2008*

Depth (m)	SOIL PROFILE – Description	Symbol	Elevation (m)	SAMPLE Number	Type	N-Value
	Ground Elevation		50.39			
0.00	***FILL:*** dark brown silt (topsoil), wet: some peat and grey silty clay, WTPL		50.00	1	SS	2
1.00				2	SS	10
	SAND AND GRAVEL: loose brown sand and gravel, some silt and trace clay, saturated		49.00	3	SS	15
2.00			48.00	4	SS	24
3.00			47.00	5	SS	24
	Borehole terminates at 3.66 m					
4.00			46.00			
5.00			45.00			
6.00			44.00			



Reviewed by: *DK*

Drill Method: *Hollow Stem Auger*

Notes:

Field Tech.: *RM*

Sheet: *1 of 1*

Drafted by: *AP(01a)*

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Handhole Number: HH1

Ground Elevation: *52.58 m*

Project: *Harrington Dam Embankment Stability Assessment*

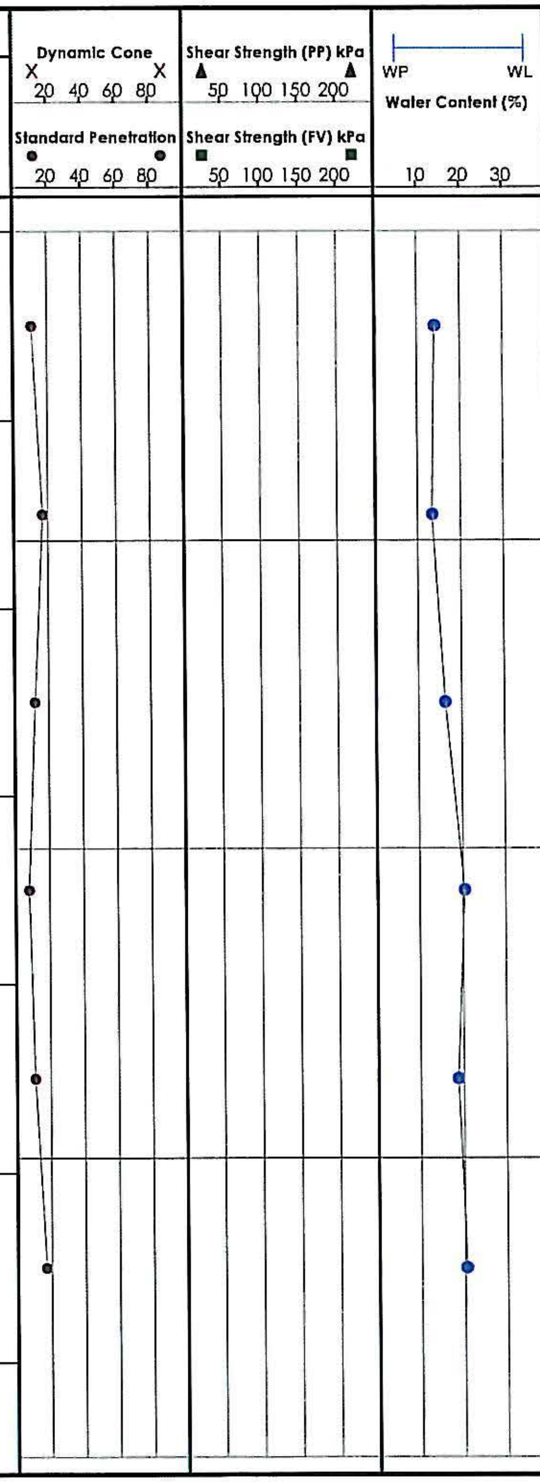
Location: *County Road 28, Harrington Ontario*

Job No.: *7608G1*

Drill Date: *June 25, 2008*

SOIL PROFILE / SAMPLE

Depth (m)	Description	Elevation	Number	Type	N-Value
	Ground Elevation	52.58			
0.00	**FILL:** compact brown silt, some sand and clay, trace gravel, very moist		1	SS	11
	root from ash tree	52.00	2	SS	17
1.00		51.00	3	SS	12
2.00			4	SS	8
	grey, saturated	50.00	5	SS	11
3.00	wood		6	SS	17
	Borehole terminates at 3.66 m	49.00			



Groundwater Observations and Standpipe Details

bentonite seal

native backfill

At drilling completion, dry cave at 2.44 m

Reviewed by: *DK*

Field Tech.: *NM*

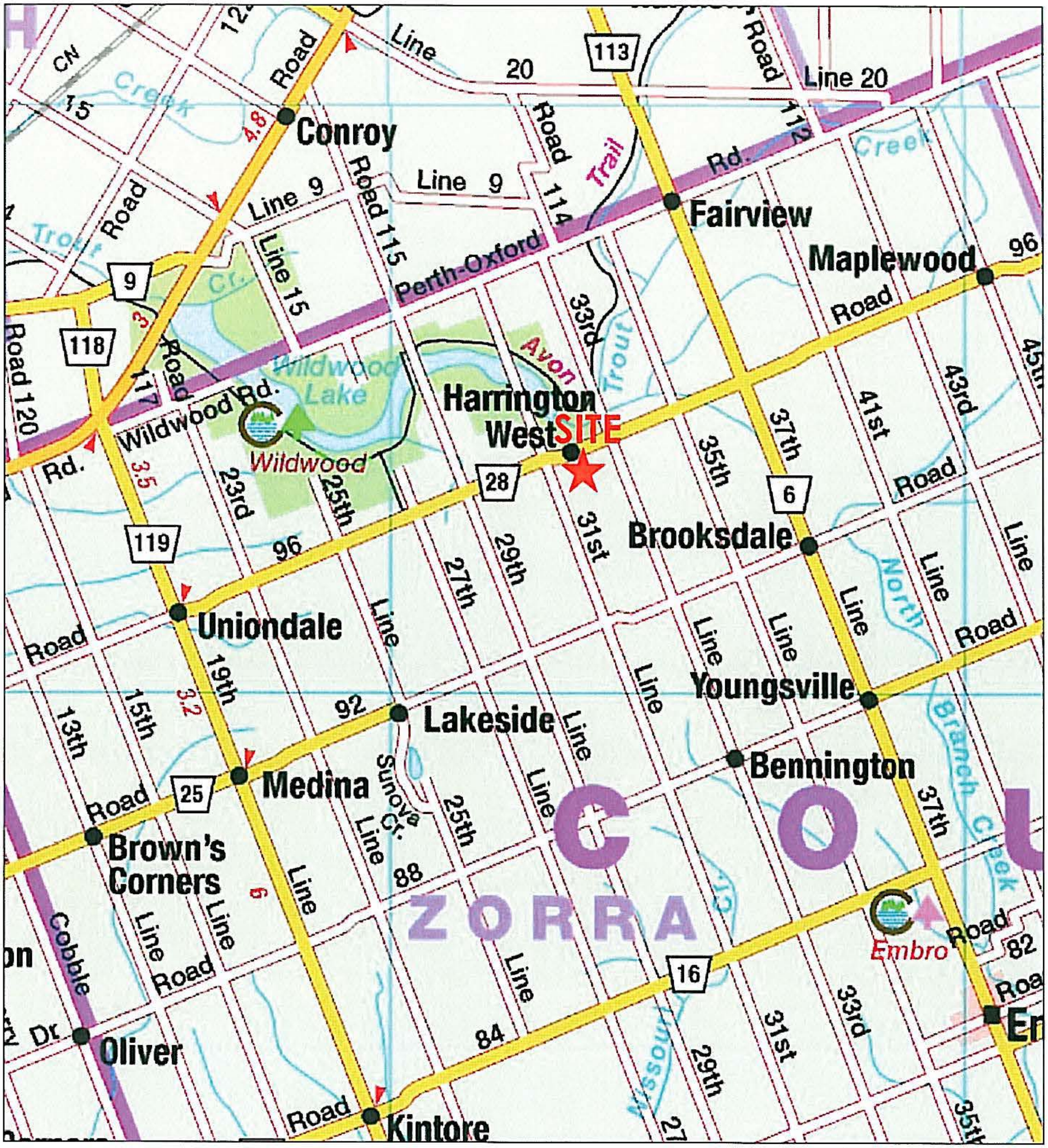
Drill Method: *Solid Stem Auger*

Sheet: *1 of 1*

Notes:

Drafted by: *Ap(00a2)*

No.	Revisions	Date
0	**Report Issued**	**Oct. 2008**
1		
2		
3		



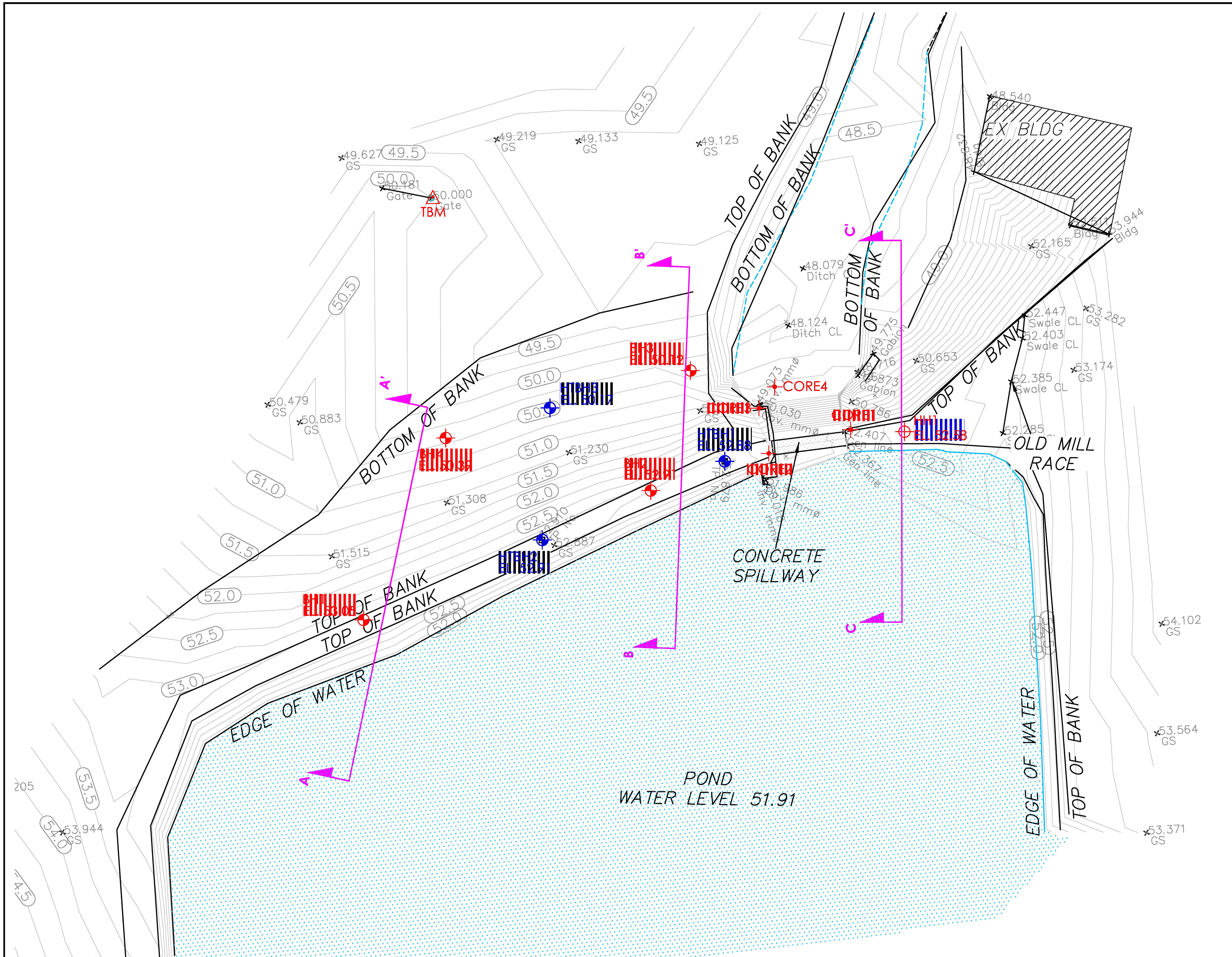
Drawing Reference: Base drawing from page 15 of MapArts Ontario Road Atlas

0 100 200 300 400 m

SCALE 1:100000

Harrington Dam Embankment **Stability Assessment** **County Road 26** **Harrington, Ontario**	Naylor Engineering Associates Ltd. CONSULTING ENGINEERS	**LOCATION PLAN**			
		Date	Scale	Job No.	Drawing No.
		Oct. 2008	**1:100000**	**7608G1**	**1**

F:\7608\7608G1\7608G1_01.DWG
June 18, 2008 (AP)



Drawing Reference: Base drawing provided by R.J. Burnside Associates Ltd.

No.	Revisions	Date
0	**Report Issued**	**Oct. 2008**
1		
2		
3		

Legend

- Borehole Location (Current Investigation)
- Handhole Location (Current Investigation)
- Approximate Borehole Location (Acres International)
- Corehole Location (Current Investigation)
- EL. 52.58 — Ground Surface Elevation (m)
- Temporary Benchmark: Top of centre concrete base for east post of gate, Elevation 50.00 m (assumed local datum)
- A A' — Geologic Cross-Sections (Refer to Drawing No. 3)

0 5 10 15 20 25 m

SCALE 1:400

NORTH

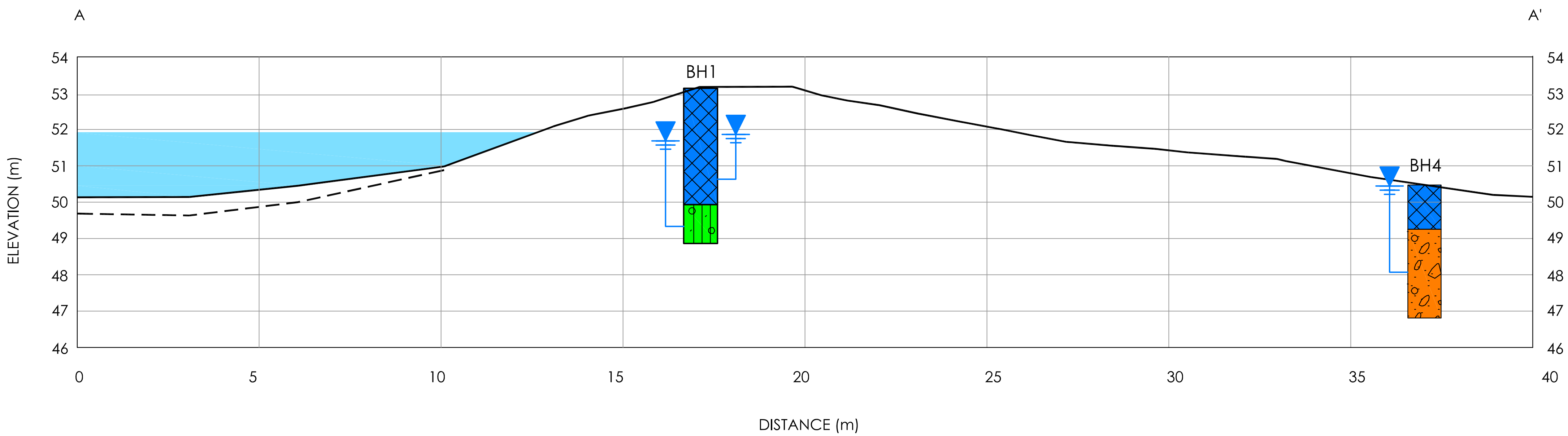
Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

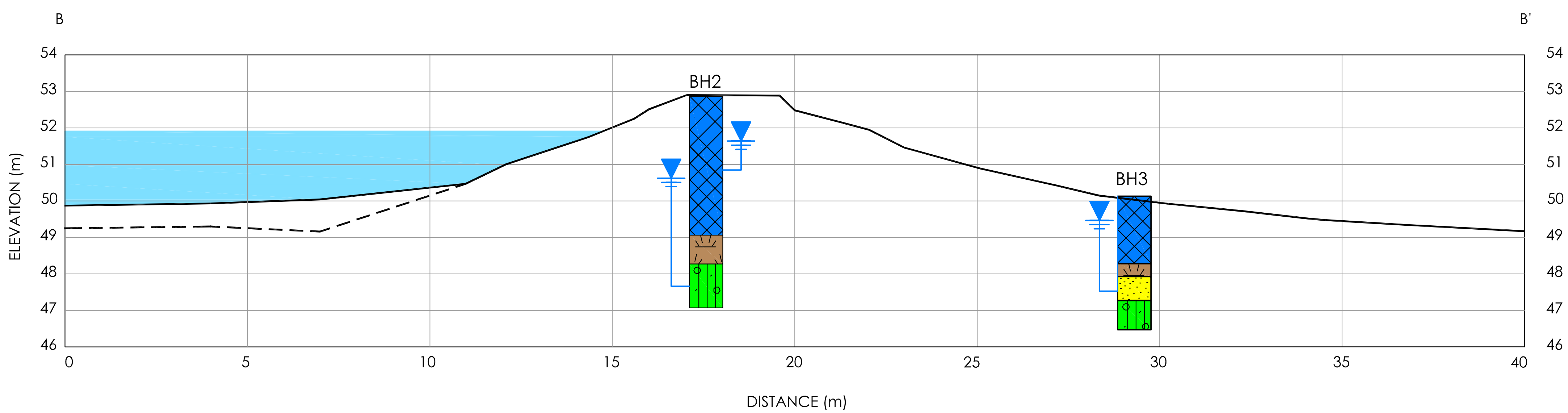
Harrington Dam Embankment
Stability Assessment
County Road 28
Harrington, Ontario

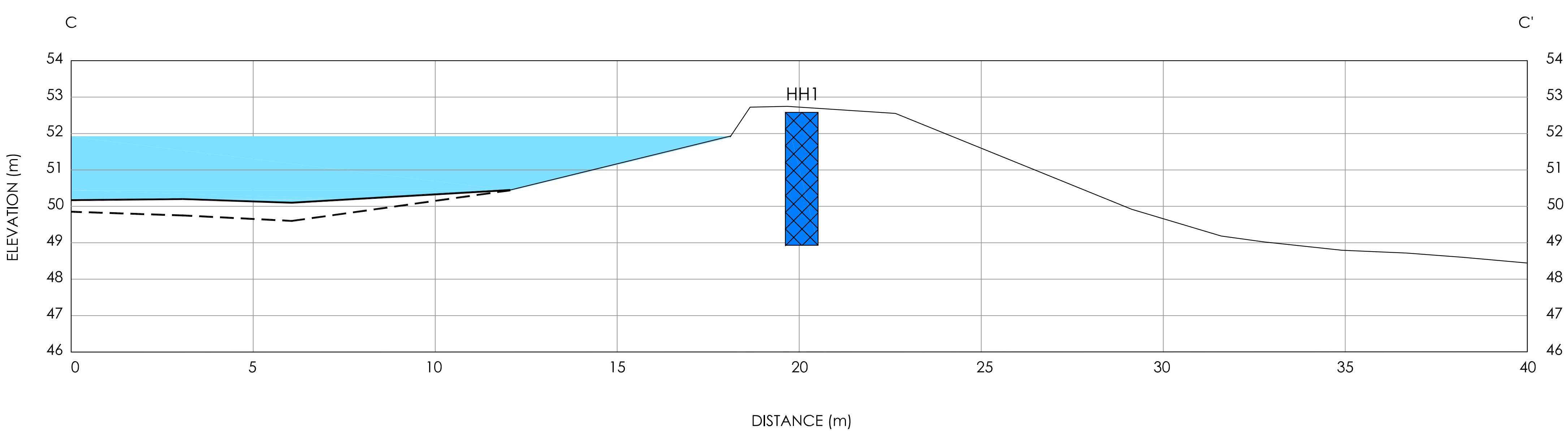
BOREHOLE LOCATION PLAN

Date	Scale	Job No.	Drawing No.
Oct. 2008	**1:400**	**7608G1**	**2**

No.	Revisions	Date
0	**Report Issued**	**Oct. 2008**
1		
2		
3		







Legend

- Fill
- Sand
- Peat
- Sand & Gravel
- Silt Till
- Water Level June 17, 2008
- Centre of Screen Location
- Existing Grade
- Approximate Top of Sediment
- Approximate Bottom of Sediment

Notes:

Seasonal fluctuations in groundwater levels would be expected.

The inferred stratigraphy shown on this cross-section is based on the subsurface stratigraphy contacted at the boreholes. The subsurface conditions between the boreholes will vary.

The ground surface under the water is based on depth (to refusal) measurements taken with a steel survey rod.

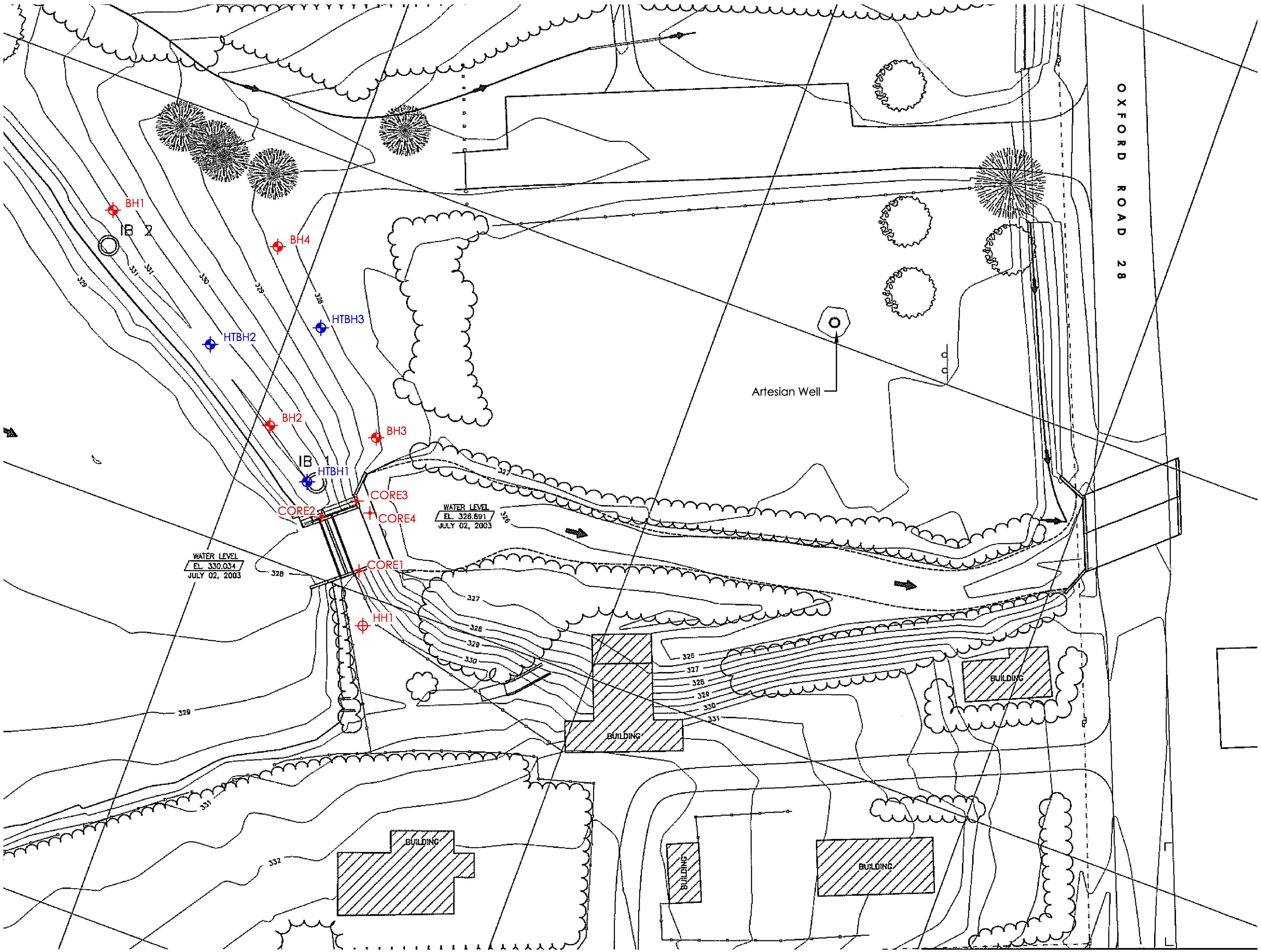
Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Harrington Dam Embankment
Stability Assessment
County Road 28
Harrington, Ontario

CROSS SECTIONS A-A', B-B', C-C'

Date	Scale	Job No.	Drawing No.
Oct. 2008	**1:150**	**7608G1**	**3**

F:\7608\7608G1\7608G1_03.dwg
July 31, 2008 (pp)



No.	Revisions	Date
0	**Report Issued**	**Oct. 2008**
1		
2		
3		

Legend

- Borehole Location (Current Investigation)
- Handhole Location (Current Investigation)
- Approximate Borehole Location (Acres International)
- Corehole Location (Current Investigation)

0 5 10 15 m

SCALE 1:500

NORTH

Naylor Engineering Associates Ltd.
CONSULTING ENGINEERS

Harrington Dam Embankment
Stability Assessment
County Road 28
Harrington, Ontario

SITE CONDITIONS PLAN

Date	Scale	Job No.	Drawing No.
Oct. 2008	**1:500**	**7608G1**	**4**

Drawing Reference: Base drawing by Acres International, Drawing No. 14504-HD-002, Dated April 1, 2004.

F:\7608\7608G1\7608G1_04.dwg
October 27, 2008 /ks