

FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED HUMBER RIVER ARCH CULVERT EXTENSION
HIGHWAY 400 INTERIM WIDENING
VAUGHAN, ONTARIO
W.P. 192-00-00, CENTRAL REGION

Submitted to:

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Pavements and Foundations Section
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FIGURES

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RECORD OF BOREHOLE SHEETS

NOTES OF BOREHOLE LOGS

RECORD OF BOREHOLE SHEETS Borehole Numbers: HR 1 to HR 3

APPENDIX 'A'

Borehole Logs extracted from report prepared by Golder Associates Limited dated May 2001
GEOCRE5 File No. 30M13-51

APPENDIX 'B'

Site Photographs

1.0 INTRODUCTION

AMEC Earth & Environmental Limited, Consulting Geotechnical, Materials Quality Control and Environmental Engineers, has been retained by the Ministry of Transportation to conduct a foundation investigation for the extension of the existing Humber River Arch Culvert in King Township, Regional Municipality of York, Ontario. The site location is as shown on the Key Plan of Drawing No. 1. This project is in conjunction with the proposed interim widening of Highway 400 from Major Mackenzie Drive to South Canal Road.

The purpose of this investigation is to determine the sub-surface conditions at the site of the proposed culvert extension by means of a number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations are provided on the geotechnical aspects of foundation design. Comments are also provided on anticipated construction issues where they may affect the geotechnical design of the proposed culvert extension. The work carried out for this geotechnical investigation was completed in accordance with AMEC's proposal (ref. P-22280, dated 20 June 2002).

The proposed plan and profile for the proposed culvert extension were provided to us by the Ministry of Transportation.

Existing subsurface information and laboratory testing results contained in the following report was used to supplement this investigation

- Golder Associates Limited. *Preliminary Foundation Investigation and Design Report, Humber River Arch Culvert Extension, Highway 400 Widening from North of Major Mackenzie Drive to South Canal Road, G.W.P. 222-97-00, Agreement No. 2005-A-000106, dated May 2001 – GEOCRE File No. 30M13-51.*

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

2.1 SITE DESCRIPTION

The existing Humber River Arch Culvert is located about 1.5 km south of Ling Road in King Township, Regional Municipality of York. We understand that the culvert has been assigned as MTO Structure Site 37-094.

The river banks at the structure location are at about Elevation 255 to 256m, while at the crest of the valley the ground rises to Elevation 274 to 275m. Highway 400 grade over the culvert is about Elevation 267m. The Humber River flows to the west through the culvert.

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The existing concrete arch culvert is about 12 m wide with a maximum height at the center of about 6.1m. The original culvert was constructed in the late 1940s under Contract 46-11 to a length of about 43.7m. The culvert was extended about 29m to the east in the late 1950s. The total length of the existing culvert is about 72.7m. The culvert is founded on spread footings at about Elevation 252.4m, and about 2.4m wide (ref. MTO Drawing TWP No. 74-94-2-8, dated March, 1955). The wingwall retaining walls are also founded at the same elevation. The stream bed is at about Elevation 254.1 within the culvert, overlain by about 0.1 to 0.2m of sediment. Below the sediment the stream bed within the culvert has been infilled with grout or concrete. Photographs of the site are included in Appendix C.

2.2 PHYSIOGRAPHY

Based on available geologic information, the site is located in the South Slope of the Oak Ridges Moraine. Generally after the last glacial withdrawal, glacial till deposits of the Halton Till formation (clayey silt to silty clay till) were deposited over the ice contact and glacial outwash sediments (sands, silts and gravels) of the Oak Ridge Moraine (ORM). The glacial till and ORM were then eroded by the Humber River and recent alluvial deposits and organics were laid down along watercourses. The entire interbedded sequence of sands, silts and gravels of the ORM is generally in the order of about 100 m in thickness in the area of the site. Shale bedrock is generally in the order of about 160 m below existing grade. The cohesionless sands and silts in the Oak Ridges moraine is a water bearing aquifer that is used as a source of water for domestic, industrial and municipal water supply and is known to be under excessive hydrostatic pressure.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the current investigation was carried out on 16 to 18 September 2002, and consisted of drilling and sampling three boreholes (Boreholes HR1, HR2 and HR3) to depths of 11.3 to 12.7m below the existing ground surface. Dynamic cone penetration tests were carried out adjacent to each borehole to depths of 11.3 to 18.3m.

Also referenced in this report in Appendix A, is one borehole (Borehole 90) advanced by Golder Associates in 2001 for a preliminary foundation investigation (GEOCRE No. 30M13-51, referenced in Section 1.0).

Borehole 90, advanced by Golder Associates in 2001, was advanced to a depth of 12.5 m and extended to a depth of about 16.8m by the dynamic cone penetration test method.

The plan locations of the boreholes advanced in the current and previous investigations, and

selected stratigraphic sections are shown on Drawing No. 1. Details of sub-surface conditions encountered at each borehole location advanced by AMEC, including the results of in-situ testing, are presented on the Record of Borehole sheets.

The boreholes for the current investigation were advanced, using hollow stem continuous flight augers with a track-mounted power auger drill rig (CME 75) owned and operated by Master Soil Investigation Limited, under the full-time supervision of experienced geotechnical personnel from AMEC Earth & Environmental Limited.

Sampling in the AMEC boreholes were carried out at regular intervals of depth (0.75 to 1.5m) by the Standard Penetration Test Method, as specified in American Standards for Testing and Materials Method Number: D-1586. This consists of freely dropping a 63.5 kilogram hammer for a vertical distance of 0.76 m to drive a 51 mm outside diameter split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground for a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-values of the soil, and this gives an indication of the consistency or the relative density of the soil deposit.

The dynamic cone penetration test was carried out from the ground surface adjacent to Boreholes HR1, HR2 and HR3, and at the bottom of Borehole 90. This test consists of continuously driving a 60° point, 50 mm diameter cone attached to the drill rod, into the undisturbed ground with a driving energy of 475 kJ (63.5 kilogram hammer free falling for a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded, providing an indication of the relative changes in the soil density with depth.

The soil samples recovered by AMEC were transported to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative soil samples from the current investigation. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets and also on Figure Nos. 1 to 11, inclusive.

Groundwater conditions in the current investigation open boreholes were observed throughout and immediately after the drilling operations. A standpipe piezometer was installed in Boreholes HR1 and HR2, to permit long term monitoring of groundwater levels at the site. All boreholes were adequately backfilled with auger cuttings and bentonite on completion of the fieldwork.

The borehole locations for the current investigation were initially established in the field by our field personnel based on existing features. The borehole locations in terms of northing and easting co-ordinates, and elevations were surveyed by Holding Jones Vanderveen Inc. We

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understand that these elevations are referenced to the geodetic datum. The locations and co-ordinates of the boreholes are shown on Drawing No. 1; the co-ordinates and elevations are indicated on the Record of Borehole Sheets.

4.0 SUB-SURFACE CONDITIONS

Details of sub-surface conditions encountered at each borehole location for the current and previous investigations by others, including the results of in-situ testing, groundwater observations and laboratory test results are presented on the Record of Borehole Sheets and in Appendix A.

4.1 Alluvial Sands with Organics

All boreholes encountered brown to grey alluvial sands underlying the thin (less than 0.1m thick) vegetation cover on the ground surface to depths ranging from 0.6 to 2.2m below existing ground surface. This deposit ranged in composition from sand to silty sand with varying amounts of gravel, clay, rootlets, and interbedded organics. Measured 'N'-values range from 3 to 47 blows per 0.3m, indicating a very loose to dense relative density, but generally loose.

4.2 Sands and Silts

Below the surficial alluvial sands, all boreholes encountered a sand to silt deposit. Boreholes HR1, HR2 and 90 were terminated within this deposit at depths of 11.3 to 12.5m below existing ground surface (or Elevation 242.5 to 243.3m). The deposit is generally composed of fine sand with trace to some silt, with interbeds of silt, silty sand and sandy silt. Measured 'N'-values range from 3 to 47 blows per 0.3m, but generally between 10 and 20 blows. It should be noted that after every auger advance and prior to split spoon sampling, between 1.5 to 4.6 m below existing ground surface in various boreholes, sand was "blown" into the hollow stem augers. This disturbance has likely resulted in lower 'N'-values.

Dynamic cone penetration test results adjacent to Boreholes HR1, HR2 and HR 3 range from 10 to 30 blows per 0.3m to a depth of about 5 to 6m. Adjacent to Boreholes HR1 and HR2 the blows increase to 70 blows per 0.3m at a depth of 7m and reach over 100 blows per 0.3m at depths of 10 to 11m. Adjacent to Borehole HR3 the blows increase to 70 blows per 0.3m at a depth of about 11m. The dynamic cone penetration tests adjacent to the boreholes were terminated at depths of 18.3m (or Elevation 236.6m) 11.3m (Elevation 243.3m) and 12.2m (Elevation 242.5m). Based on the measured 'N'-values and the dynamic cone test results, the sands and silts can be considered to have a relative density of loose to compact to a depth of about 5 to 6m (Elevation 249 to 250m), increasing to compact to dense to a depth of about 10 to 11m (Elevation 244 to 245m).

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Laboratory tests carried out on the sands and silts are summarized on the various Record of Boreholes, in Appendix A and in Figure Nos. 1 to 11, inclusive. The laboratory results are summarized below.

Natural Moisture Content 17 to 23%

Grain Size Distribution

Fine Sand, trace Silt (7 samples) – Figure Nos. 1 to 5 and Appendix A

Gravel	0 to 1%
Sand	83 to 96%
Silt	4 to 10%
Clay	0 to 6%

Silty Sand (6 samples) – Figure Nos. 6 to 11

Gravel	0 to 8%
Sand	50 to 78%
Silt	14 to 43%
Clay	5 to 8%

4.3 Clayey Silt Glacial Till

Underlying the sands and silts, a grey clayey silt glacial till deposit was encountered in Borehole HR3 at a depth of 11.6m (Elevation 241.1m) and the borehole was terminated within this deposit at a depth of 12.7m (or Elevation 252.0m). A measured 'N'-value of 60 blows per 0.3m was obtained within this till indicating a hard consistency. The glacial till is a heterogeneous mixture of clay and silt with sand and trace gravel size particles. The dynamic cone penetration test was terminated within this deposit with blows ranging from 75 to 95 blows per 0.3m.

It should be noted that cobbles and boulders may be encountered within this deposit due to its nature of formation.

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4.4 Groundwater Conditions

Groundwater levels in the open boreholes were observed during drilling and upon completion of each borehole. To permit long term monitoring of groundwater levels at the site, standpipe piezometers were installed in Boreholes HR1 and HR2.

Based on the observations in the piezometers and from tactile and visual observations of the recovered soil samples, in our opinion, the groundwater table at the culvert location is at a depth of about 0.6m, or at about Elevation 254m \pm .

In Borehole HR2, the morning after completion of the borehole, when removing the hollow stem augers, groundwater was overflowing the top of the augers. Artesian groundwater conditions were not observed during drilling. After installation of the standpipe piezometer, the groundwater level within the pipe stabilized at about 0.3m above ground surface. Groundwater measurements about seven weeks later indicate a groundwater level 0.2m above ground surface. Excessive hydrostatic pressure (artesian condition) was observed from the piezometer indicates an upward groundwater flow gradient.

It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events.

5.0 DISCUSSION AND RECOMMENDATIONS

This report contains the findings of our geotechnical investigation and those carried out by others, together with our recommendations and comments. These recommendations and comments are based on factual information and are intended only for use of the Design Engineers. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. Sub-surface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The anticipated construction conditions are also discussed, but only to the extent that they may influence design decisions. Construction methods discussed, however, express our opinion only and are not intended to direct the Contractors on how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all the factors that may have an effect upon the construction.

Based on the Preliminary Design Study plan (Exhibit No. 6-4, prepared by McCormick Rankin Corporation dated June 2002), the existing arch culvert that crosses beneath Highway 400 is about 72.7m in length. The proposed design consists of the extension of the existing structure 14.4m to the east and west with new wingwalls. The existing concrete arch culvert is about 12 m wide with a maximum height at the center of about 6.1m.

In general, the subsoils along the proposed culvert extension consist of upper loose to compact alluvial sands with interbedded organic, clay and gravel to a depth of about 1 to 2m, underlain by loose to compact sands and silts to a depth of about 5 to 6m, with increasing relative density from compact to very dense at a depth of about 10 to 11m. The water bearing sands and silts are under artesian pressure at depth, and the groundwater table is generally within 1m of the ground surface.

5.1 Culvert Foundations

5.1.1 Spread Footings

Based on the data obtained from the boreholes, the proposed culvert widening can be supported on spread footing foundations placed on the undisturbed, dewatered compact sands and silts. The existing culvert foundations are founded at Elevation 252.4m, and about 2.4m wide and will therefore control the founding level of the proposed extension adjacent to the existing structure. The footing should also be placed below the anticipated scour depth.

The following highest founding depths and soil resistances are recommended at the borehole locations for 2.4m wide spread footings placed in undisturbed, competent natural soils.

TABLE 1

BOREHOLE / LOCATION	EXISTING GROUND SURFACE ELEVATION (m)	RECOMMENDED HIGHEST BASE (bottom) OF FOOTING ELEVATION (m)	DEPTH BELOW EXISTING GROUND SURFACE TO FOOTING BASE (m)	BEARING RESISTANCE AT S.L.S. (kPa)	FACTORED BEARING RESISTANCE AT U.L.S. (kPa)	SUBGRADE MATERIAL
90 North Side of Humber River – East Side of Hwy 400	255.0	252.7	2.3	150	300	Compact Sand
HR1 South Side of Humber River – East Side of Hwy 400	254.9	252.4	2.5	150	300	Compact Sand
HR2 South Side of Humber River - West Side of Hwy 400	254.6	252.4	2.2	100	200	Loose to Compact Sand
HR3 North Side of Humber River – West Side of Hwy 400	254.7	252.4	2.3	100	200	Compact Sand

The factored bearing resistance at ULS given in the above table incorporated a resistance factor of 0.5 as per Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00.

The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

It should be noted that settlement of the existing and the proposed structure footings will be influenced by the additional loadings imposed by the embankment fill which will cause further settlement of the underlying sands and silts. A finite element analysis using a computer programme, Sigma/W, was carried out for the proposed embankment widening over the existing and proposed culvert. Total settlements of up to 80 mm were determined over the proposed extension and existing culvert, and the settlements induced by the proposed additional loads by the embankment widening are presented in Figure No. 12. Anticipated settlements adjacent to

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the existing culvert are presented in Figure No. 13. These settlements are in addition to the above noted settlements induced by the loading at S.L.S.

Under inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Section 6.7.2 of the C.H.B.D.C. (CAN/CSA-S6-00).

The unfactored horizontal resistance against sliding between concrete and the compact sand surface can be calculated using a friction angle of 24 degrees.

Due to the settlement that will be induced by the additional fill (up to 12m of fill) for the embankment widening, the proposed structure will undergo up to about 80 mm of total settlement. These settlements will create differential settlement of the Highway 400 embankment. As there is not sufficient space at the site to preload or surcharge the area, therefore the following three options may be considered to reduce the anticipated settlements or changes to the proposed design.

Option A: Design the footings of the culvert to accommodate the anticipated settlements and allow for maintenance of the culvert and highway embankment.

Option B: Drive sheet piles adjacent to the proposed footings to allow for excavation below the groundwater table. Dewater the cofferdam and excavate to Elevation 250.0m. Replace the excavated sands with compacted fill to Elevation 252.4m and construct the spread footings. The S.L.S and U.L.S. for design purposes on compacted fill are 300 and 700 kPa, respectively.

Option C: Design the embankment widening using lightweight fill to reduce the settlement associated with the fill embankment. Additional analysis using Sigma/W was carried out for embankment widening consisting of compacted flyash and Styrofoam instead of compacted earth fill. Total settlements would be limited to about 60 mm for flyash and less than 40mm for Styrofoam.

Option A is considered to be unfeasible due to the magnitude of settlements anticipated. Option C can be further discussed during detail design of the highway embankment widening. Option B is further discussed in detail below.

Option B

Prior to fill placement, the proposed foundation area should be enclosed by driving steel sheet piles to at least Elevation 245m. The enclosure should then be dewatered using deep wells within the enclosure. The cofferdam enclosure should then be excavated down to Elevation

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250.0m. Footing inspection should be conducted in accordance with SP902S01 – Excavating and Backfilling to Structures. In this case, when the excavation reaches the required depth, the subgrade should be evaluated and approved by the Quality Verification Engineer appointed by the Contract Administrator. The surface of the subgrade should be compacted to at least 98% Standard Proctor Maximum Dry Density.

The grade within the sheet pile enclosure should then be raised to Elevation 254.4m by placing the excavated sands in compacted layers. The fill material should be placed in thin layers not exceeding approximately 200 mm when loose. Oversize particles (cobbles and boulders) larger than 100 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used, to at least 98% of its Standard Proctor Maximum Dry Density. The fill used for engineered fill should not be frozen and should be placed at a moisture content within 2% of the optimum value for compaction. The engineered fill should not be performed during winter months when freezing ambient temperatures occur persistently or intermittently. The sheet piles should be cut-off at the proposed founding level of the culvert footings and left in place.

For compacted fill subgrade prepared as indicated above, an S.L.S. value of 300 kPa and U.L.S. of 700 kPa can be used for design purposes. The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. The proposed embankment widening should induce less than 25 mm of total settlement, in addition to the above noted settlements at S.L.S.

It is recommended that instrumentation in the form of settlement monitoring be carried out for the existing culvert and highway embankment during the driving of the sheet piles. Such monitoring is necessary to confirm that any settlement/ movement associated with the proposed construction would be within tolerable limits.

5.1.2 Pile Foundations

It is not recommended to support the proposed culvert extension on driven piles as this may cause detrimental damage to the existing spread footings and/or disturbance to the founding subgrade of the existing culvert.

Augered-in-place (caisson) foundations and micro-piles are also considered not feasible due to the artesian water conditions within the sands and silts.

5.2 Retaining Wall Foundations

Retaining walls are proposed for the wingwalls of the culvert extension, which would be parallel to Highway 400, and about 2 to 3m in height. Based on the results of the investigation, a number of design options in constructing the proposed retaining wall were considered at this site, such as a reinforced concrete retaining wall, gabions and RSS walls.

The following three options, which are considered feasible for this site, are listed below:

- Option 1: Reinforced Concrete Retaining Wall
- Option 2: Gabion (gravity) Retaining Wall
- Option 3: Retained Soil System (RSS) Wall

The above options are feasible for construction in terms of geotechnical engineering. The selection of a preferred option will primarily depend on the construction cost, aesthetic requirements, and construction staging factors.

The following section presents a discussion of the suitability of the three options and is intended only to assist the designer in choosing the most feasible option for the site. Summary of the different options are presented below in Table 2.

Table 2: Summary of Retaining Wall Options

No.	Options	Description	Advantages	Disadvantages	Cost Comparison
1	Reinforced Concrete Retaining (R.C.) Wall	'L' or inverted T-shaped cantilever wall, which has a vertical or inclined slab monolithic with a base slab. Generally, up to 8m high.	<ul style="list-style-type: none"> -use of conventional construction methods -features can be incorporated into the finished face -durable, provided it is designed properly -can practically retain soil up to 15 m 	<ul style="list-style-type: none"> -construction of spread footings normally requires large excavation area with sometimes limited available space -stringent requirements on selected fill to be used for drainage -reasonably good foundation is required -curing time is required before becoming effective -generally costly 	Medium to High
2	Gabion Retaining Wall	A gravity retaining wall consisting of baskets filled with rockfill. Generally, up to 5 to 6 m high.	<ul style="list-style-type: none"> -Simple to construct and maintain -Permits construction on weaker foundations -Flexible structure, hence tolerate higher differential settlements than R.C. retaining walls. -Relatively easy to demolish in part or in full -More economical for relatively long walls 	<ul style="list-style-type: none"> -self draining filling required -cost may be high for small quantities -Less durable than conventional R.C.wall 	Low
3	Retained Soil System Wall (RSS)	Vertical wall panels or facing are supported by anchored strips within the backfill behind the wall. RSS walls are normally used for true or false abutments, wall/slope and roadbase embankment.	<ul style="list-style-type: none"> -easy to construct -RSS walls higher than 4 m are often more economical than conventional walls. -permits construction on weaker foundations -good appearance 	<ul style="list-style-type: none"> -need larger space behind the retaining wall and may extend beyond the ROW -generally applicable for walls greater than 2 m 	Low to Medium

Details of each proposed option are discussed below.

5.2.1 Option 1: Reinforced Concrete Retaining Wall

This conventional wall is an 'L' or inverted 'T' shaped cantilever wall, which has a vertical or inclined concrete slab monolithic with a concrete base slab. This type of retaining wall is impermeable and usually requires the provision of drainage holes (weep holes) through the walls to prevent build-up of hydrostatic pressure behind it. In addition, it normally requires granular backfill such as Granular 'B'. The advantages and disadvantages of this option are presented in Table 2. The main advantage of this option, in our opinion, is its durability; while its main disadvantage could be its relatively high cost and requirement of large excavation area behind the wall. For the design of footings for the concrete retaining wall, the geotechnical recommendations are presented in Section 5.2.4 of this report.

5.2.2 Option 2: Gabion Retaining Wall

Gabion retaining wall is a gravity retaining wall consisting of baskets filled with rockfill. This type of wall is flexible and can tolerate higher differential settlements than the concrete retaining wall. This wall is permeable and does not require granular backfill or weeping holes for drainage.

For gabion retaining wall, the same geotechnical parameters recommended for the conventional concrete retaining wall could be used.

For gabion structure, the active earth pressure should be used in the design. A geotextile separator (non-woven, Class II, Filtration Opening Size of 75 to 150 μm) should be placed behind the wall to prevent migration of fine particles into the wall due to seepage pressure. A back batter of 1H to 6V at the back face of the retaining wall is recommended to reduce the earth pressure acting on the wall and even out ground bearing pressures. Frost cover is also not normally required for this structure because of its flexibility.

The main advantages of this option are that they are simple to construct and are generally cost effective.

5.2.3 Option 3: Retained Soil System

A retained soil system wall is a type of wall that utilizes reinforced earth system where vertical wall panels or facing are supported by anchored strips within the backfill behind the wall. These walls are also flexible and easy to construct. However, this option requires relatively large construction area behind the wall (i.e., required width behind the wall in the order of about 0.7 times the height of the wall) and is normally cost effective for walls higher than 2 m. The RSS wall will need to be designed to accommodate the settlements induce by the embankment widening.

For retained soil system, the following should be included in the Contract Documents:

- S identify longitudinal extent in plan of the Retained Soil System
- S identify in plan transverse space constraints (top of wall and bottom of wall)
- S identify elevation of top of wall and bottom of wall
- S include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and moderate to high appearance.

5.2.4 Retaining Wall Foundations

From the findings of the boreholes, the footings for the retaining wall can be founded on the native compact sands and silts.

For design purposes the following C.H.B.D.C. bearing resistances may be used for spread footings of about 1m wide:

BOREHOLE / LOCATION	EXISTING GROUND SURFACE ELEVATION (m)	RECOMMENDED HIGHEST BASE (bottom) OF FOOTING ELEVATION (m)	DEPTH BELOW EXISTING GROUND SURFACE TO FOOTING BASE (m)	BEARING RESISTANCE AT S.L.S. (kPa)	FACTORED BEARING RESISTANCE AT U.L.S. (kPa)	SUBGRADE MATERIAL
90 North Side of Humber River – East Side of Hwy 400	255.0	252.7	2.3	150	300	Compact Sand
HR1 South Side of Humber River – East Side of Hwy 400	254.9	252.4	2.5	150	300	Compact Sand
HR2 South Side of Humber River - West Side of Hwy 400	254.6	252.4	2.2	100	200	Loose to Compact Sand
HR3 North Side of Humber River – West Side of Hwy 400	254.7	252.4	2.3	100	200	Compact Sand

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The factored bearing resistance at ULS given in the above table incorporated a resistance factor of 0.5 as per Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00.

The serviceability condition is based on the premise that total settlements will not exceed 25mm.

Under inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Section 6.7.2 of the C.H.B.D.C. (CAN/CSA-S6-00).

The sliding resistance between concrete and the sand is 24 degrees. For gabions, the sliding resistance over the sand is 19 degrees.

Based on the finite element analysis carried out for the above site, the proposed embankment widening will induce up to 25 mm of settlement after fill placement.

Footing inspection should be conducted in accordance with SP902S01 – Excavating and Backfilling to Structures. In this case, when the excavation reaches the required depth, the subgrade should be evaluated and approved by the Quality Verification Engineer appointed by the Contract Administrator. If necessary, the excavation may need to be deepened to the surface of a sufficiently competent soil.

5.2.5 Recommended Options

Based on the above, we recommend the options in the following order of priority:

For the retaining wall:

1. Option 3: Retained Soil System

This option involves construction of a soil reinforced structure. There is adequate space to construct this option and is less expensive than Option1.

2. Option 1: Reinforced Concrete Retaining Wall

This option involves construction of concrete footings. The wall is durable and requires little maintenance which is desirable due to the limited access to the site.

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3. Option2: Gabion Retaining Wall

This option involves construction of a flexible structure, however due to the limited durability of this structure and the limited access to the site, it is the least favourable option.

Considering the height of the proposed wall (maximum 2 to 3 m high), and the limited access for maintenance to the site, Option 3, Retained Soil System, is considered the most suitable option at this site.

5.3 Lateral Earth Pressures and Backfilling

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with Ministry of Transportation of Ontario's standards.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with C.H.B.D.C. (CAN/CSA-S6-00). For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'

- Unit Weight = 22 kN/m^3
- Coefficient of Lateral Earth Pressures:
 $K_a = 0.27$ (active condition)
 $K_o = 0.43$ (at-rest condition)

Compacted Granular 'B'

- Unit Weight = 21 kN/m^3
- Coefficient of Lateral Earth Pressures:
 $K_a = 0.31$ (active condition)
 $K_o = 0.47$ (at-rest condition)

The above design parameters assume level ground surface and backfill behind the retaining structure.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at-rest pressures should be used as per Section 6.9.2.1 of the C.H.B.D.C. (CAN/CSA-S6-00). The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9.3 of the C.H.B.D.C. (CAN/CSA-S6-00).

.../...

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current Ministry of Transportation of Ontario's practice.

Backfilling arrangements around the culvert should be carried out as per OPSD 803.02. Backfill to the culvert should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular 'A' or 'B'. The excavated soil is suitable for backfill, provided organic inclusions and/or clay layers are removed and the sands are reconditioned prior to reuse (i.e. spread out and dried, or mixed with drier soils). All granular fill should be placed in loose lifts not exceeding 200 mm thick and be compacted to at least 98% of the material's Standard Proctor Dry Density.

Heavy equipment should not be used adjacent to the walls and roof of the culvert. The height of the backfill to the culvert walls should be maintained equal on both sides of the structure during all stages of backfill placement.

5.4 Embankment Stability and Widening

Slope stability analyses were performed on the existing conditions for the west and east embankment slopes. The stability of the proposed embankment widening was analyzed by the limit equilibrium method using a computer programme Slope/W. The following assumed soil parameters were adopted.

SOIL TYPE	FRICTION ANGLE (degrees)	UNIT WEIGHT (kN/m ³)
EARTH FILL, compacted	30	21
Alluvial Sands, loose to compact	28	18
Upper Sand to Silty Sand, loose to compact	30	19
Lower Sand to Silty Sand, compact to dense	33	21

The groundwater table was assumed to be at Elevation 254.0m, about 1m below the ground surface adjacent to the river. The results of the slope stability analysis are presented in Figure No. 14. The embankment slope stability results indicate that a slope of 2H to 1V is considered stable.

The embankment widening over this Site should include the excavation of all the topsoil and organic soils at the toe of the existing embankment to expose the native, competent ('firm'

.../...

bottom) soils, over the full width of the proposed embankment widening as per O.P.S.D. 203.030.

Embankments of granular/earth fill with a side slope inclination of 2 horizontal to 1 vertical (2H:1V), would be stable against surficial stability, provided that the subgrade is properly prepared by removing all surficial topsoil, loose existing fill, organic and otherwise unsuitable materials as per MTO Standards before placing the new fill. Appropriate benching (as per OPSD 208.010) at the sloping subgrade level should be considered to allow the new fill to key into the existing slope. Berms of 2 m in width should be provided as per current MTO practice (mid-height berm for every 8 m of embankment height). The berm gradient should be sloped (about 20H:1V) to drain away surface water from the embankment.

The fill materials used for construction of the earth fill embankment, or for the purposes of backfilling, should consist of approved, clean earth fill. The fills should be placed in accordance with OPSS501 and with lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the Standard Proctor Maximum Dry Density of the materials. The degree of compaction within the top 0.6 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under a geotechnical control programme.

In general, we recommend that, in as much as practicable, for the construction of the widened sections the same materials as the existing embankment be used. That is, if the existing embankment was constructed of earth fill, the widening should preferably be implemented using earth fill.

If the widening parts of the embankments are constructed using earth fill, the existing embankment slopes should be stripped. The existing embankment slopes should also be benched in accordance with OPSD 208.010 to ensure that the new embankment is keyed into the existing embankment and to minimize differential settlements.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (Ontario Provincial Standards Specifications Number: 572).

5.5 Scour Protection and Erosion Protection to River Channel

Based on the existing base of the culvert, the streambed within the culvert should be filled with a cement-grout mixture or lean concrete to maintain uniform scour protection to the footings. At the culvert ends, the embankment slope around the culvert inlet and outlet, and the riverbank adjacent to the culvert, should be protected against erosion by placement of a rip-rap (rock)

.../...

blanket, to an Elevation of 1m above the high water level of the river. Rip-rap should also be provided for an adequate distance from the culvert, both upstream and downstream, taking into account the anticipated flow rates, culvert gradient and scouring caused by the river. The scour and erosion protection design should be carried out in accordance with the MTO Drainage Management Manual.

The rip-rap blanket should consist of 100 to 200 mm size rocks placed by hand or machinery in a 0.3m thick layer. The rock should be blocky and angular in shape and even in size. Any voids in the rockfill should be filled with smaller size rock or gravel of similar type to prevent underlying materials from washing out. A geotextile separator (non-woven, Class II FOS 75 to 150 μm) should be placed behind the rockfill to prevent migration of fine particles into the rockfill due to seepage pressure. A toe for the filter and rip-rap protection should be provided at the edge of the lining and protective cover to key the lining into the natural ground to provide protection to erosion and scour.

For a better design of scour protection, if required, it is recommended that further study be carried out to define the following parameters for erosion control:

- Flow rate
- Water Depth
- Type of transported sediments
- Detailed cross section survey
- Stream pattern and alignment
- Channel gradient

For the proposed culvert extensions, we recommend the use of cutoff walls around the proposed culvert to prevent erosion of the enclosing soil.

5.6 Construction Comments

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations (i.e. Occupational Health and Safety Act O.Reg. 213/91).

The boreholes show that the excavations for the structure foundations can be expected to extend through surficial alluvial sands and silts and below the groundwater table within the fine sands. Open cut excavations are not expected to stand temporarily below the water table.

Due to the high groundwater table at the site, and the adjacent Humber River, dewatering the sands will be difficult. In order to divert the river during construction and to assist in dewatering, sheet piles should be driven to Elevation 245.0m adjacent to the existing structure and

.../...

encompass the proposed excavation to create a water-tight cofferdam. Dewatering within this cofferdam can be carried out using temporary filtered sumps or wells. Upon completion of the footing construction, the sheet piles should be cut at the footing level and left in place.

Water removed by pumping should not be discharged directly into the river. The effluent from dewatering operations should be filtered or passed through sediment traps to prevent turbidity. During construction, temporary runoff controls such as sediment traps, interceptor drains, dikes and/or silt fence s should be p rovided and installed t o p revent u ncontrolled water f low and sediment into the river.

Excavation below the groundwater table prior to dewatering is not recommended as this will cause loosening of the founding subgrade and necessitate deeper excavations to undisturbed subgrade.

Allowance should be made to place an approximately 150 mm thick layer of lean concrete on the subgrade surface, i.e. excavation base, within four hours of preparation and acceptance of the bearing soil, as the fine sands and silts are easily disturbed. It should be pointed out that if the foundation soil is disturbed, excessive settlements could occur after structural loads are applied. Care should also be exercised to minimize disturbance to the sandy subgrade during excavation.

5.7 Frost Protection

Design frost penetration for the general area is 1.2 m. Therefore, a permanent soil cover of 1.2 m or its thermal equivalent is required for frost protection of foundations.

5.8 Construction Inspection

It is recommended that a quality control programme of inspection and testing be carried out during the construction phase of the project to confirm that the conditions encountered are consistent with design assumptions; and to confirm that the various project specifications and material requirements and handling are being s atisfied. Regular checking by surveying of foundation movement may be required.

6.0 CLOSURE

We recommend that once the details of the structures are finalized, our recommendations should be reviewed for their specific applicability.

Sincerely,



Andrew Drevininkas, P. Eng.
Assistant Manager
Geotechnical Services



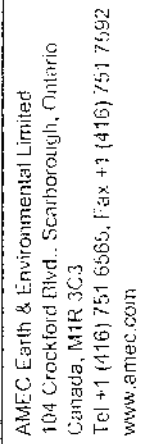
Kai-Sing Ho, Ph.D., P.Eng.,
Principal Geotechnical Consultant,
MTO Designated Contact.



K:\GEO-TRANSPORT\PROJECTS\2002\TT22852 - humber - final - design.doc

FIGURES

amec



UNIFIED SOIL CLASSIFICATION SYSTEM

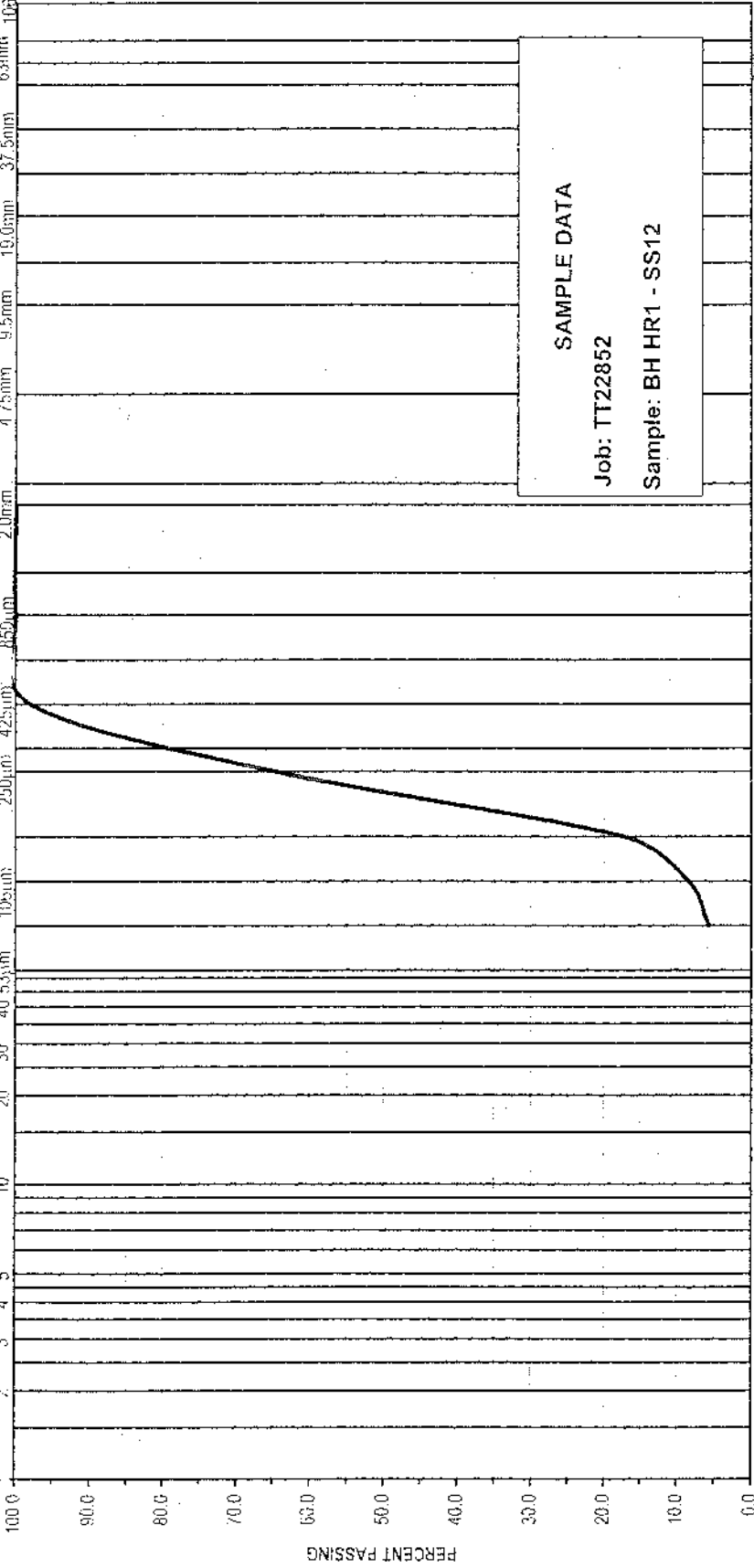


CLAY & SILT		SAND		GRAVEL	
		Fine	Medium	Coarse	Coarse

MINISTRY SIEVE DESIGNATION (Metric)

GRAIN SIZE IN MICROMETERS

1	2	3	4	5	10	20	30	40	53.0mm	75.0mm	150µm	250µm	300µm	425µm	600µm	850µm	1.18mm	2.36mm	4.75mm	9.5mm	19.0mm	25.0mm	37.5mm	53.0mm	75.0mm	106.0mm
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SAMPLE DATA

Job: TT22852

Sample: BH HR1 - SS12

MINISTRY SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

SAND
trace Silt

Client :- Ministry of Transportation Ontario

Project:- Highway 400 Interim Widening

Location:- Vaughan, Ontario

Depth :-

Date :- 30-Sep-2002

AMEC Earth & Environmental Limited
104 Crockford Blvd., Scarborough, Ontario
Canada, M1R 3C3
Tel +1 (416) 751 6565, Fax +1 (416) 751 7592
www.amec.com

UNIFIED SOIL CLASSIFICATION SYSTEM

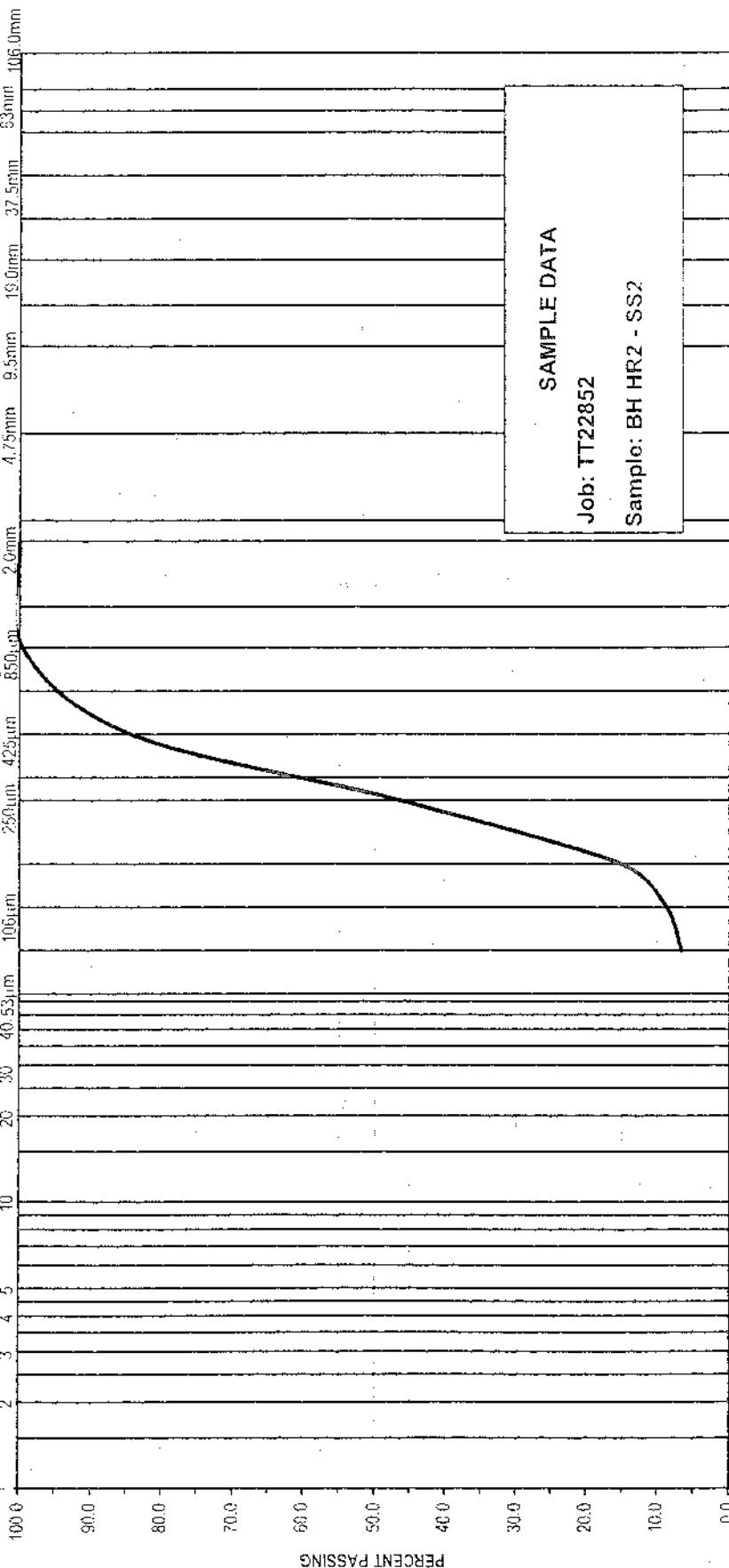


CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

MINISTRY SIEVE DESIGNATION (Metric)

GRAIN SIZE IN MICROMETERS

75µm	150µm	300µm	600µm	850µm	2.0mm	4.75mm	9.5mm	19.0mm	25.0mm	53.0mm	75.0mm
2	3	4	5	10	20	30	40	60	75	100	150



SAMPLE DATA

Job: TT22852

Sample: BH HR2 - SS2

MINISTRY SIEVE DESIGNATION (Imperial)

2.5"

3"

4"

1"

1.5"

3/4"

3/8"

1/2"

1/4"

1/8"

1/16"

1/32"

1/64"

1/128"

1/256"

1/512"

1/1024"

1/2048"

1/4096"

1/8192"

1/16384"

1/32768"

1/65536"

1/131072"

1/262144"

1/524288"

1/1048576"

1/2097152"

1/4194304"

1/8388608"

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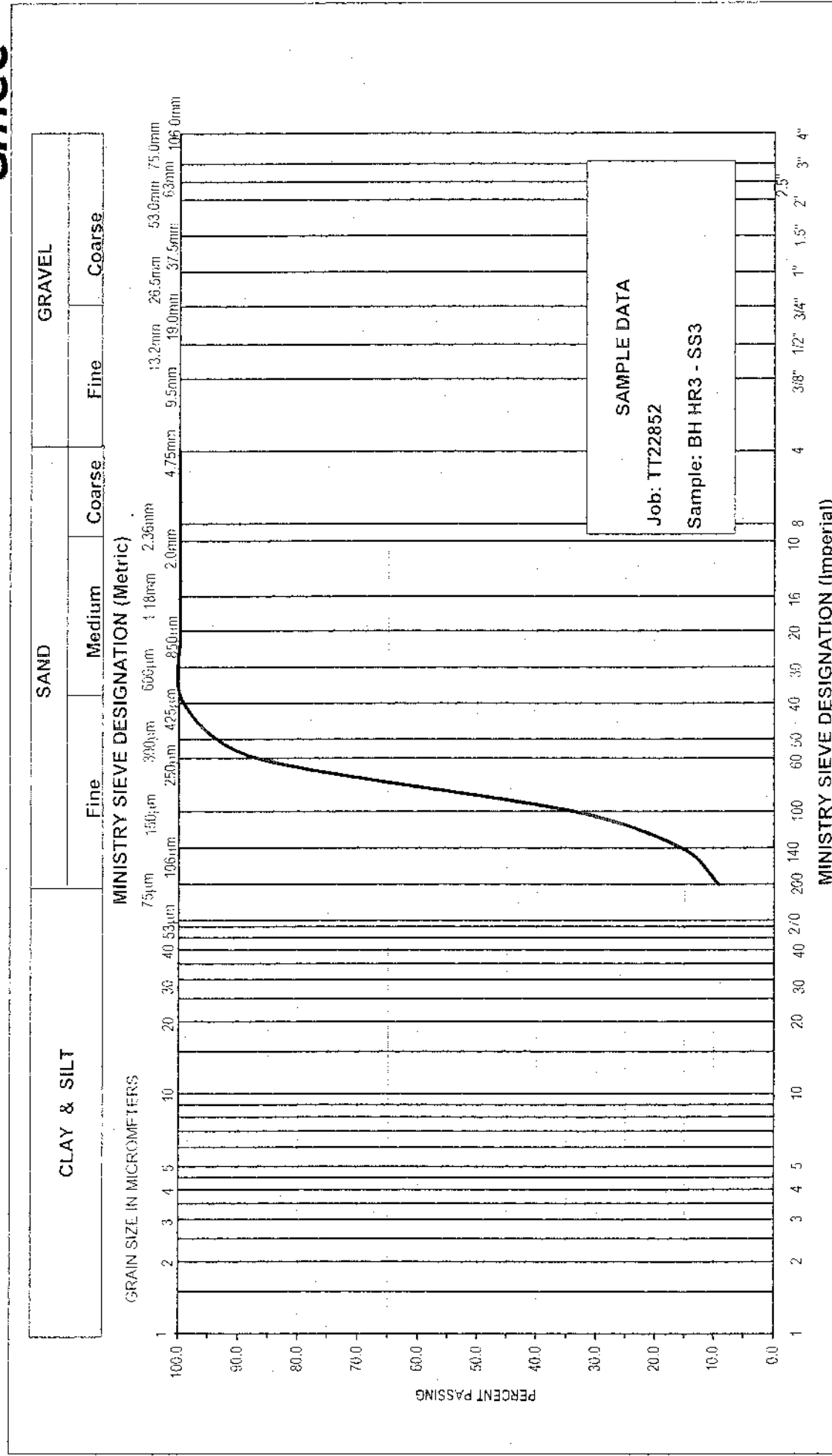
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UNIFIED SOIL CLASSIFICATION SYSTEM



AMEC Earth & Environmental Limited 104 Crockford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6565, Fax +1 (416) 751 7592 www.amec.com	GRAIN SIZE DISTRIBUTION		Client :- Ministry of Transportation Ontario	
	SAND trace Silt		Project:- Highway 400 Interim Widening	
			Location:- Vaughan, Ontario	
			Depth :-	Date :- 30-Sep-2002

UNIFIED SOIL CLASSIFICATION SYSTEM

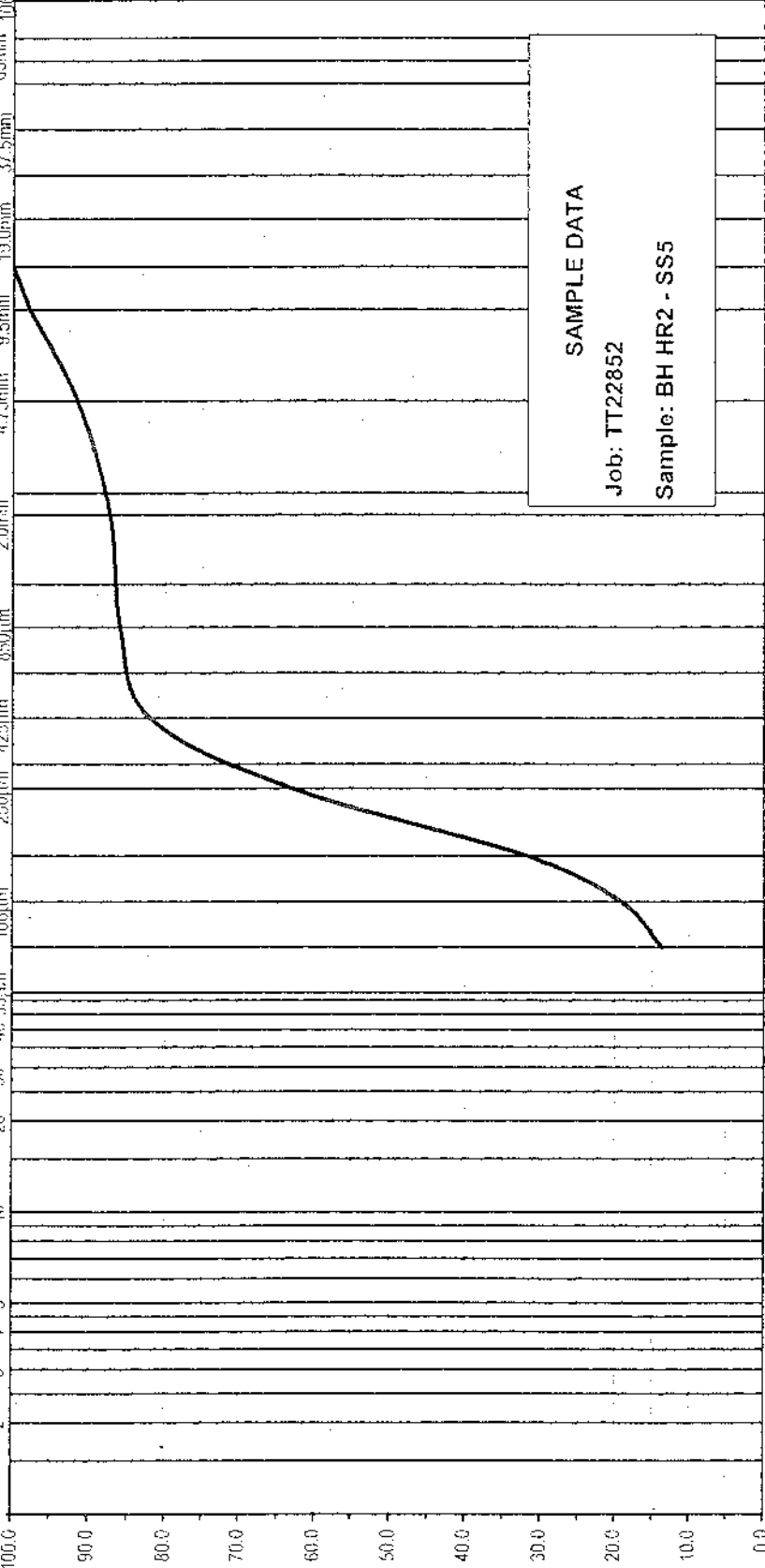


CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse

MINISTRY SIEVE DESIGNATION (Metric)

GRAIN SIZE IN MICROMETERS

1	2	3	4	5	10	20	30	40	53µm	75µm	106µm	150µm	250µm	300µm	425µm	600µm	850µm	1180µm	2.36mm	4.75mm	9.5mm	13.2mm	20.0mm	25.0mm	37.5mm	53.0mm	75.0mm	106.0mm
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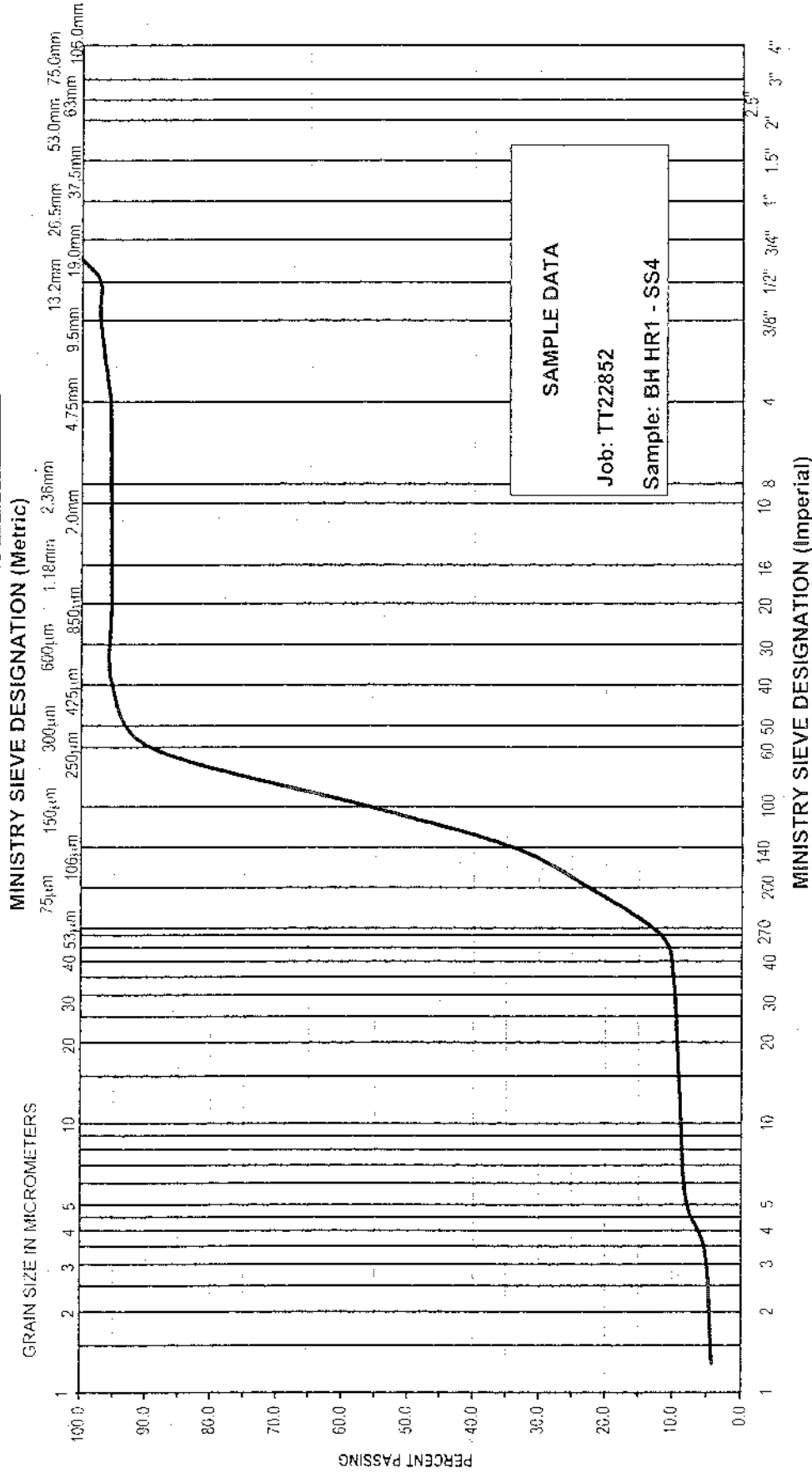
MINISTRY SIEVE DESIGNATION (Imperial)

AMEC Earth & Environmental Limited 104 Crockford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6565, Fax +1 (416) 751 7592 www.amec.com	GRAIN SIZE DISTRIBUTION		Client :- Ministry of Transportation Ontario
	SAND		Project:- Highway 400 Interim Widening
	some Silt trace Gravel		Location:- Vaughan, Ontario
		Depth :-	Date :- 30-Sep-2002

UNIFIED SOIL CLASSIFICATION SYSTEM

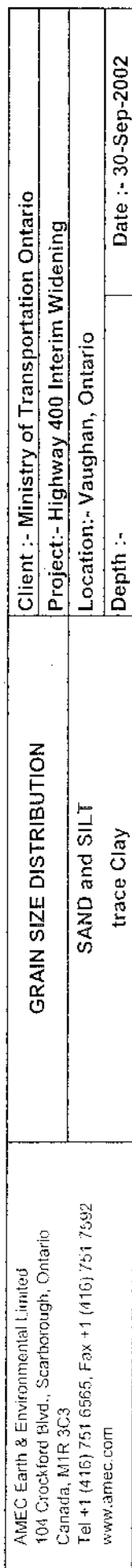


CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



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	SAND	Project:- Highway 400 Interim Widening
	some Silt trace Gravel and Clay	Location:- Vaughan, Ontario
		Depth :-
		Date :- 30-Sep-2002

amc



UNIFIED SOIL CLASSIFICATION SYSTEM

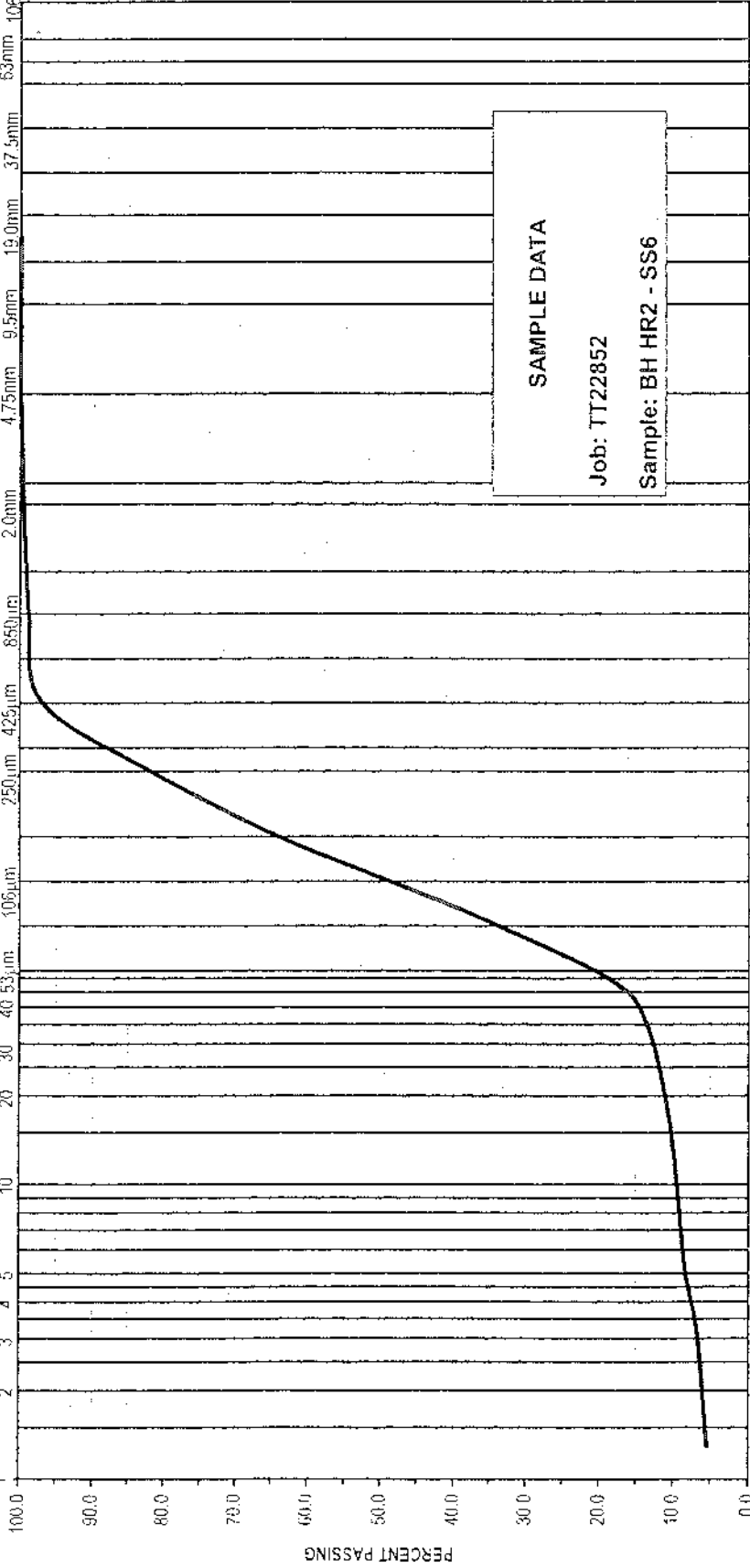


CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse

MINISTRY SIEVE DESIGNATION (Metric)

GRAIN SIZE IN MICROMETERS

1	2	3	4	5	10	20	30	40	53	75	106	150	250	300	425	600	850	1180	2000	2360	4.75mm	9.5mm	19.0mm	26.5mm	53.0mm	75.0mm	106.0mm
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SAMPLE DATA

Job: TT22852

Sample: BH HR2 - SS6

MINISTRY SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

SAND

with Silt trace Clay

Client :- Ministry of Transportation Ontario

Project:- Highway 400 Interim Widening

Location:- Vaughan, Ontario

Depth :-

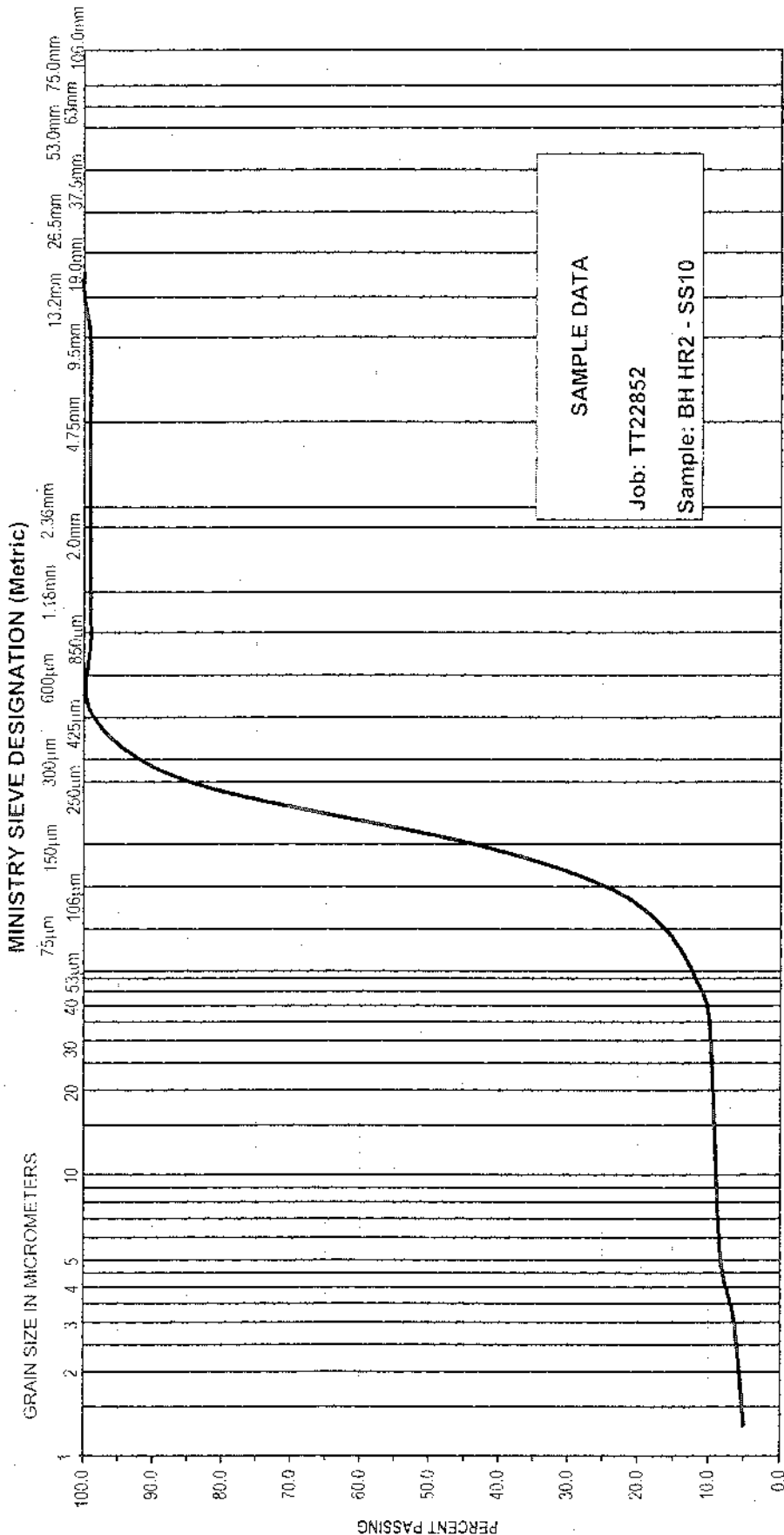
Date :- 30-Sep-2002

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Canada, M1R 3C3
Tel +1 (416) 751 6565, Fax +1 (416) 751 7592
www.amec.com



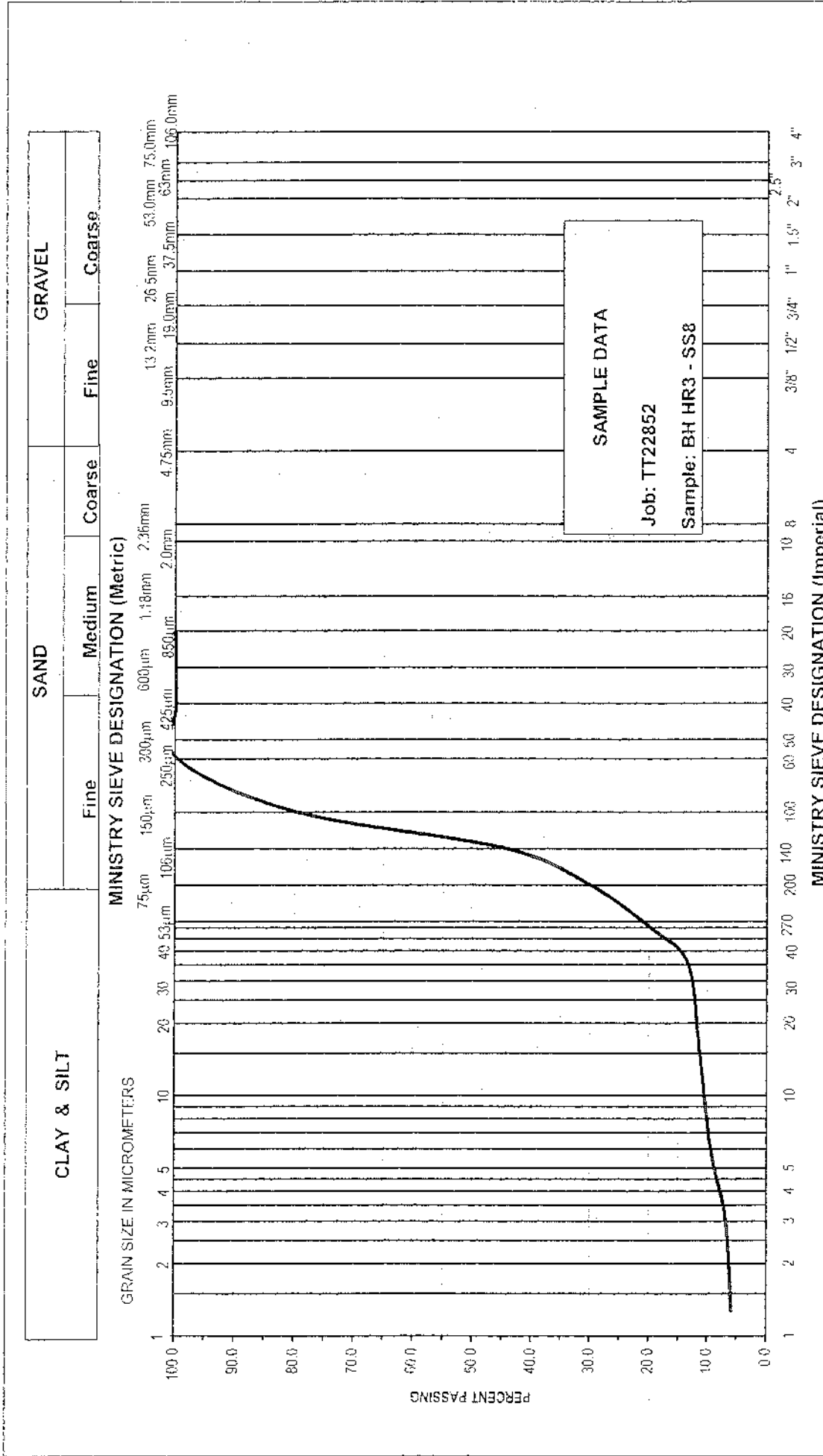
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse



AMEC Earth & Environmental Limited 104 Crockford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6565, Fax +1 (416) 751 7392 www.amec.com	GRAIN SIZE DISTRIBUTION		Client :- Ministry of Transportation Ontario
	SAND		Project :- Highway 400 Interim Widening
some Silt trace Gravel and Clay		Location :- Vaughan, Ontario	Date :- 30-Sep-2002

UNIFIED SOIL CLASSIFICATION SYSTEM



AMEC Earth & Environmental Limited 104 Crockford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6565, Fax +1 (416) 751 7592 www.amec.com	Client :- Ministry of Transportation Ontario	
	Project:- Highway 400 Interim Widening	
	Location:- Vaughan, Ontario	
	Depth :-	Date :- 30-Sep-2002

UNIFIED SOIL CLASSIFICATION SYSTEM

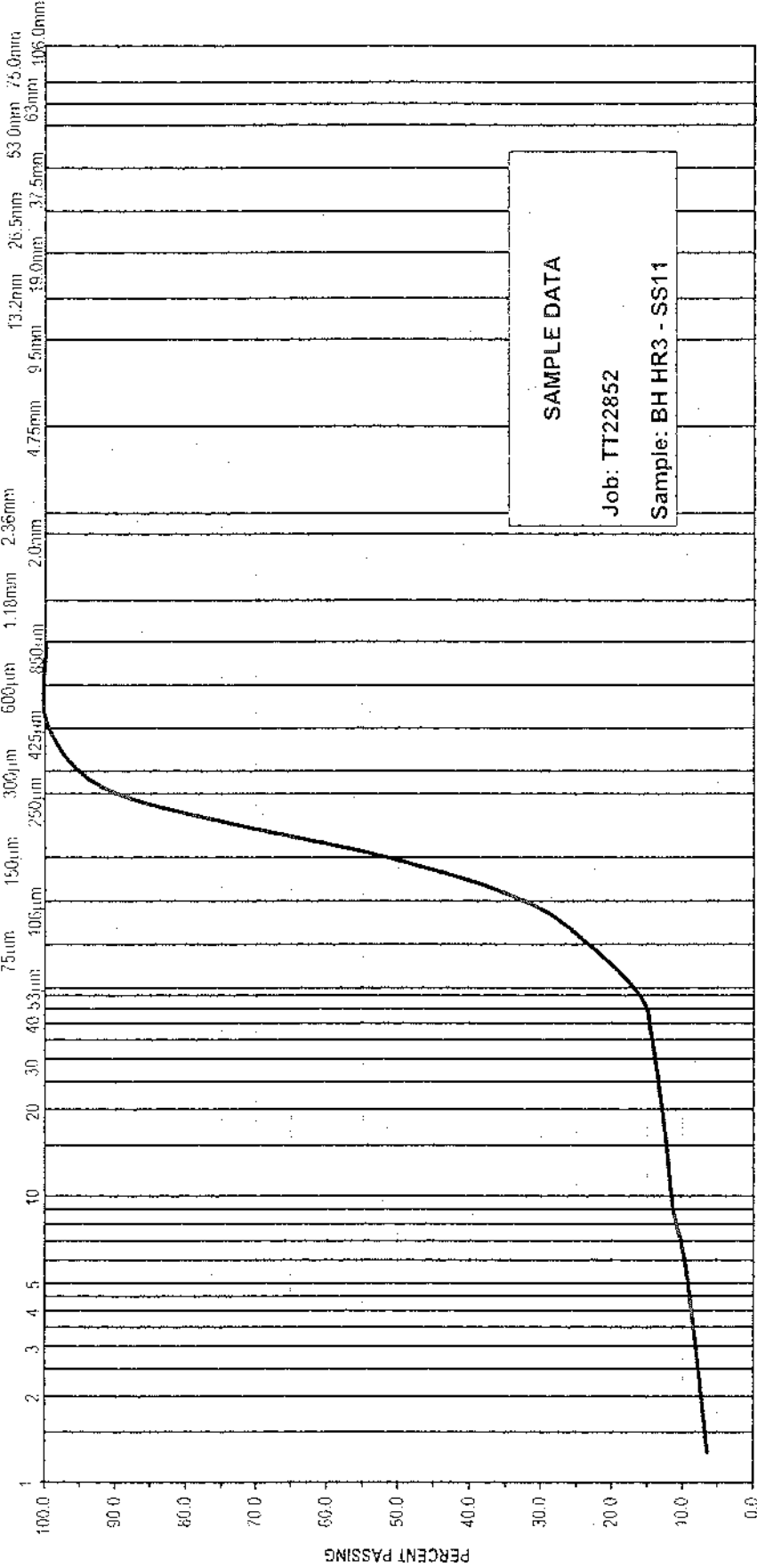


CLAY & SILT		SAND		GRAVEL	
		Fine	Medium	Coarse	

MINISTRY SIEVE DESIGNATION (Metric)

75.0µm 150µm 300µm 600µm 850µm 2.0mm 4.75mm 9.5mm 13.2mm 25.0mm 53.0mm 75.0mm
 106µm 200µm 250µm 425µm 600µm 850µm 1180µm 1320µm 1900µm 2500µm

GRAIN SIZE IN MICROMETERS



MINISTRY SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

SAND

some Silt trace Clay

Client :- Ministry of Transportation Ontario

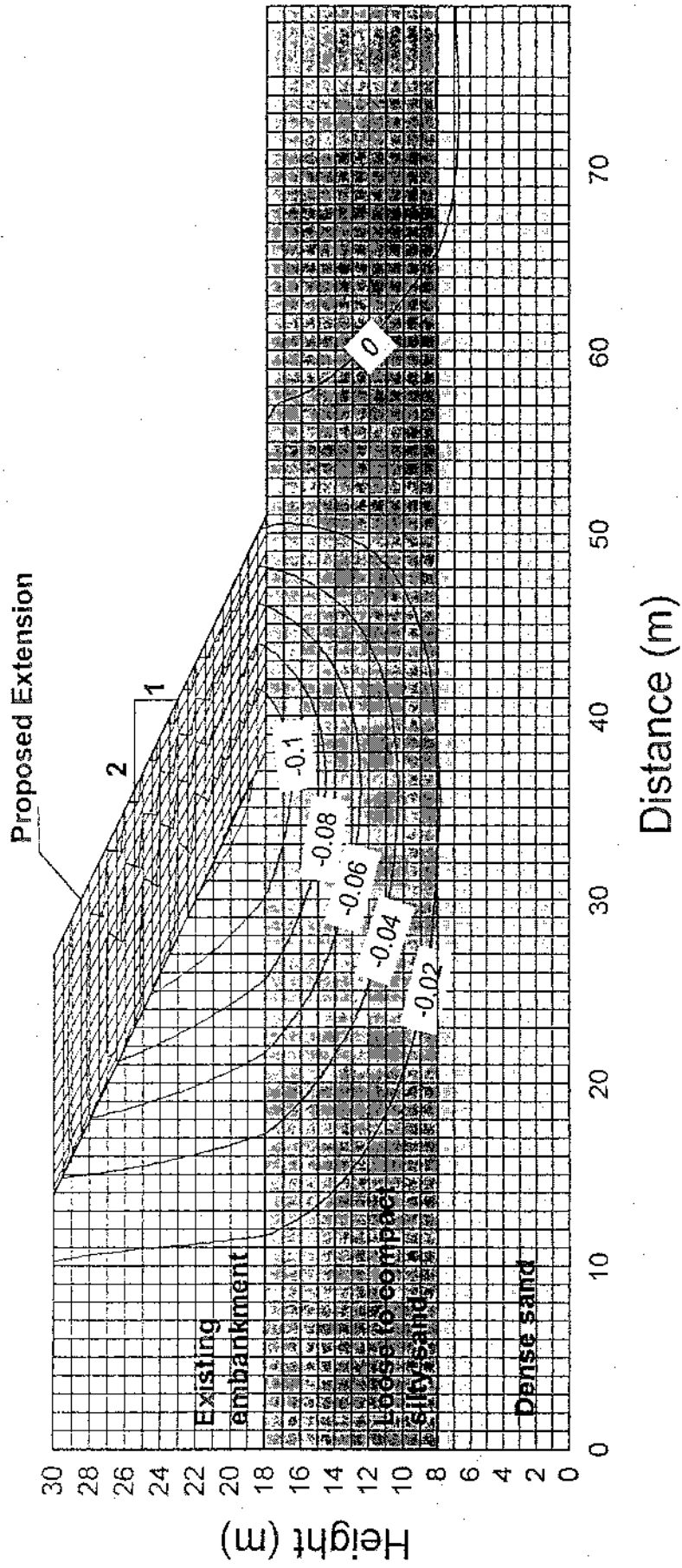
Project:- Highway 400 Interim Widening

Location:- Vaughan, Ontario

Depth :-

Date :- 30-Sep-2002

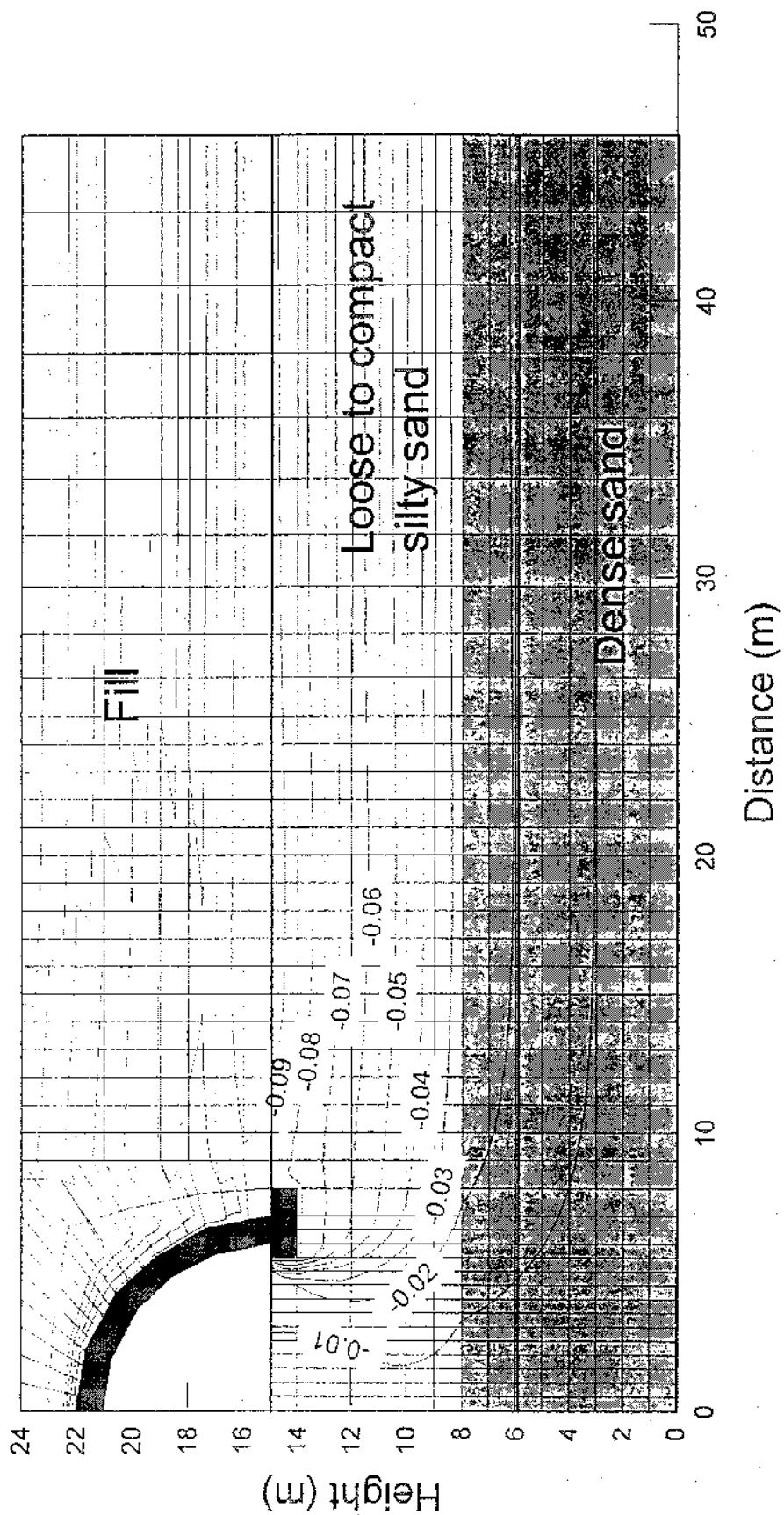
AMEC Earth & Environmental Limited
 104 Crockford Blvd., Scarborough, Ontario
 Canada, M1R 3C3
 Tel +1 (416) 751 6565, Fax +1 (416) 751 7592
 www.amec.com



AMEC EARTH AND ENVIRONMENTAL LIMITED

Project No. TT2852B Humber River Arch Culvert Extension, Highway 400, King Township

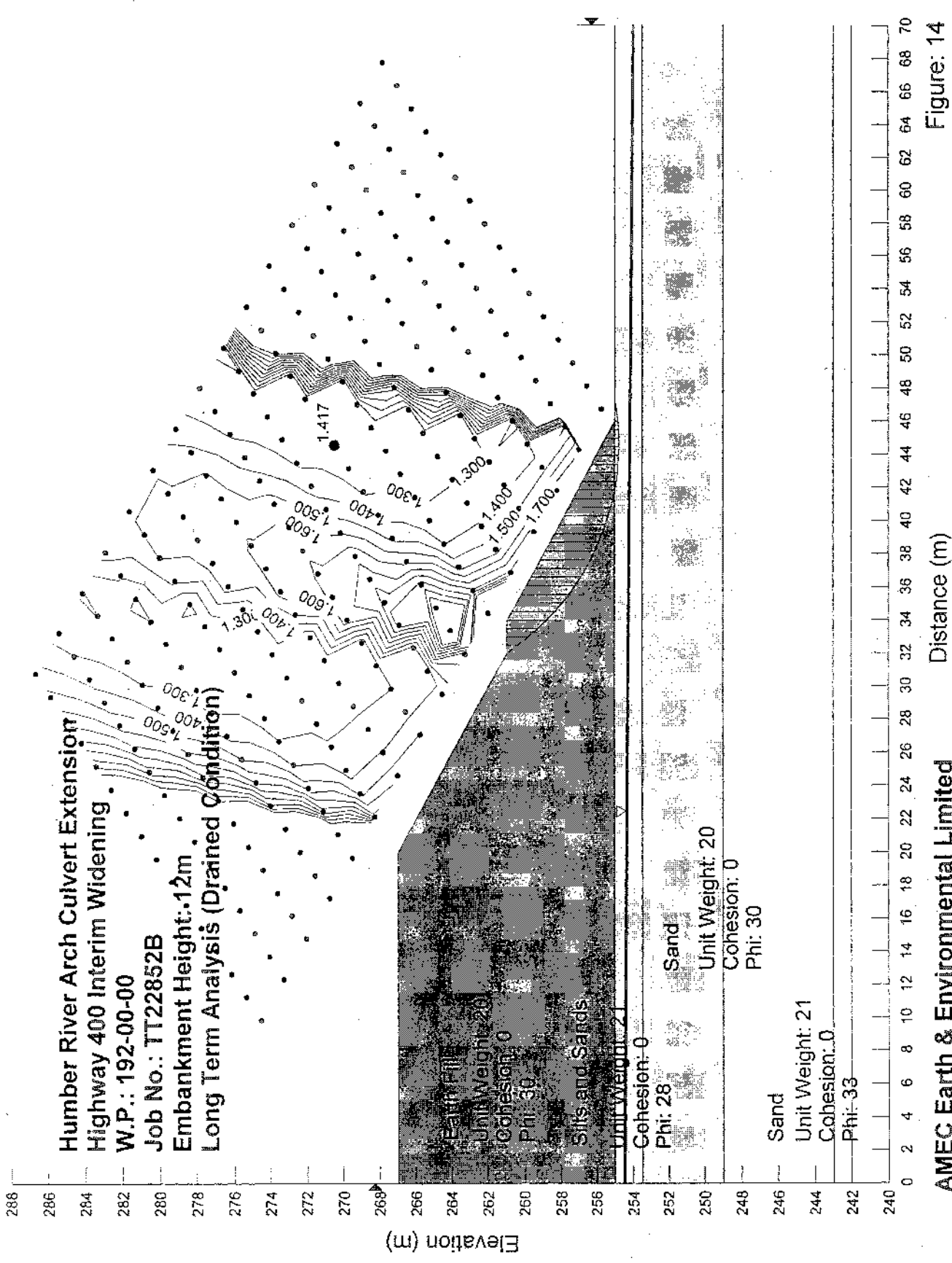
Figure 12 - Foundation settlement due to widening



AMEC EARTH AND ENVIRONMENTAL LIMITED

Project No. TT22852B Humber River Arch Culvert Extension, Highway 400, King Township

Figure 13 - Settlement near the culvert



Humber River Arch Culvert Extension
 Highway 400 Interim Widening
 W.P.: 192-00-00
 Job No.: TT22852B
 Embankment Height: 12m
 Long Term Analysis (Drained Condition)

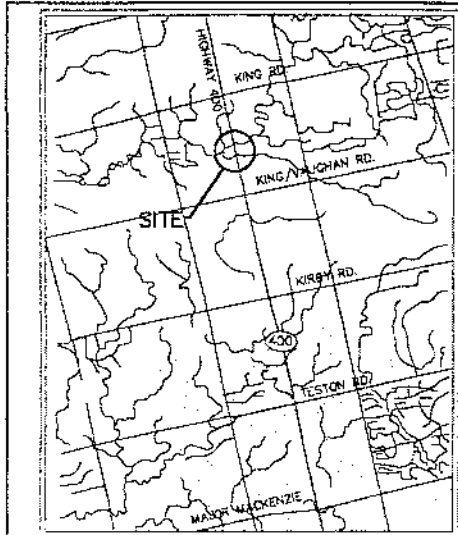
Figure: 14
 K:\GEO-TRANSPORT\PROJECTS\2002\TT22852\SLOPE

AMEC Earth & Environmental Limited

DRAWINGS

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES - METRES.

AMEC Earth & Environmental Limited



- LEGEND
- Bore Hole -- AMEC Investigation
 - ⊕ Bore Hole & Core -- previous investigations done by others
 - 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
 - CONC Blows/0.3m (90° Cone, 475 J/blow)
 - ↓ WL at time of investigation--
 - ↓ WL in Piezometer
 - ⊥ Piezometer
 - ⊥ End of Borehole

No	ELEV	CO-ORDINATES	
		NORTH	EAST
90	255.0	4 863 549	299 699
HR1	254.9	4 863 533	299 710
HR2	254.8	4 863 528	299 818
HR3	254.7	4 863 515	299 816

NOTE: The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

DATE	BY	DESCRIPTION

HWY No 400

SUBMITTAL CHECKED AD

DRAWN VK

DIST

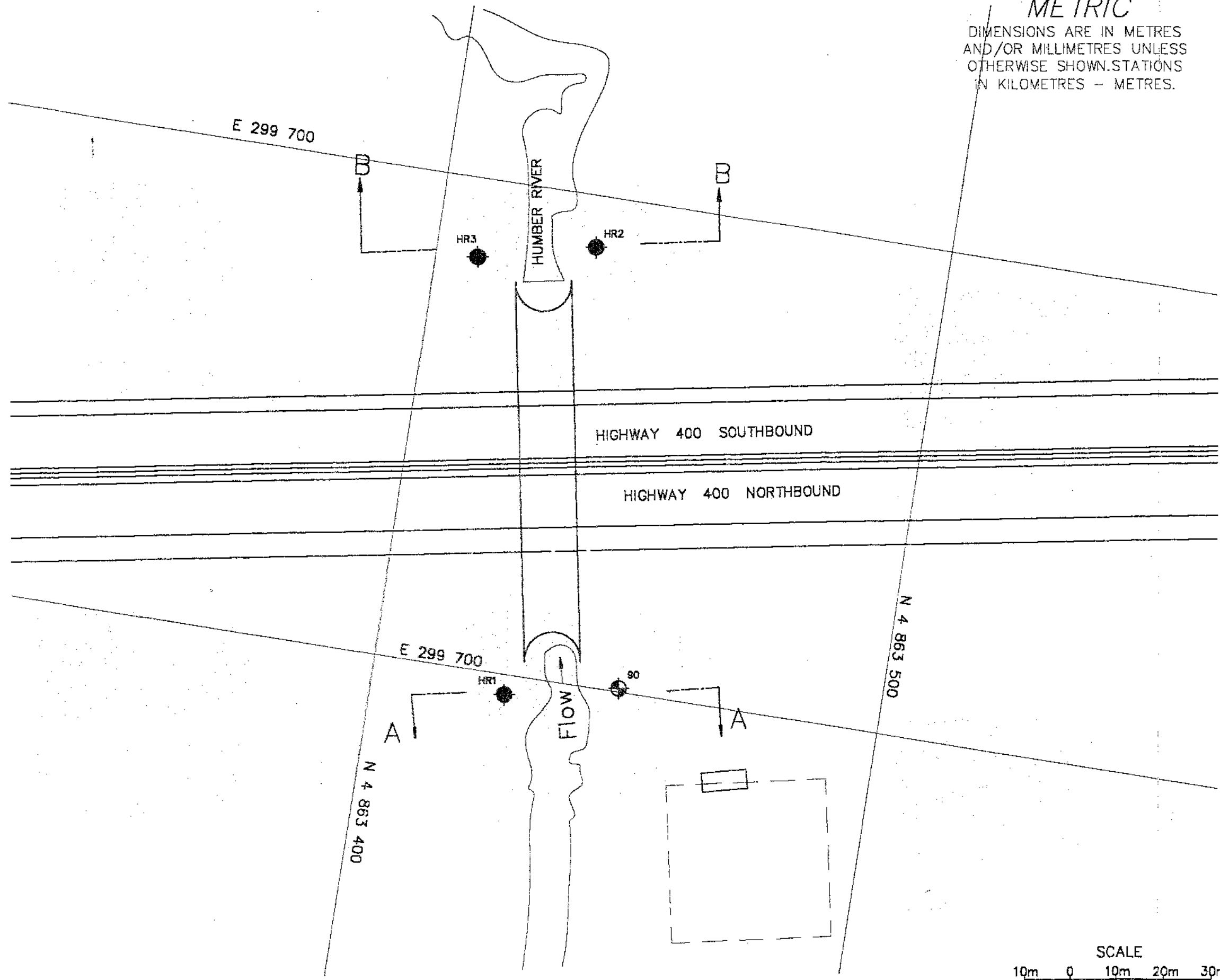
DATE Oct, 2002

CHECKED KSH

SITE

APPROVED

DWG 1



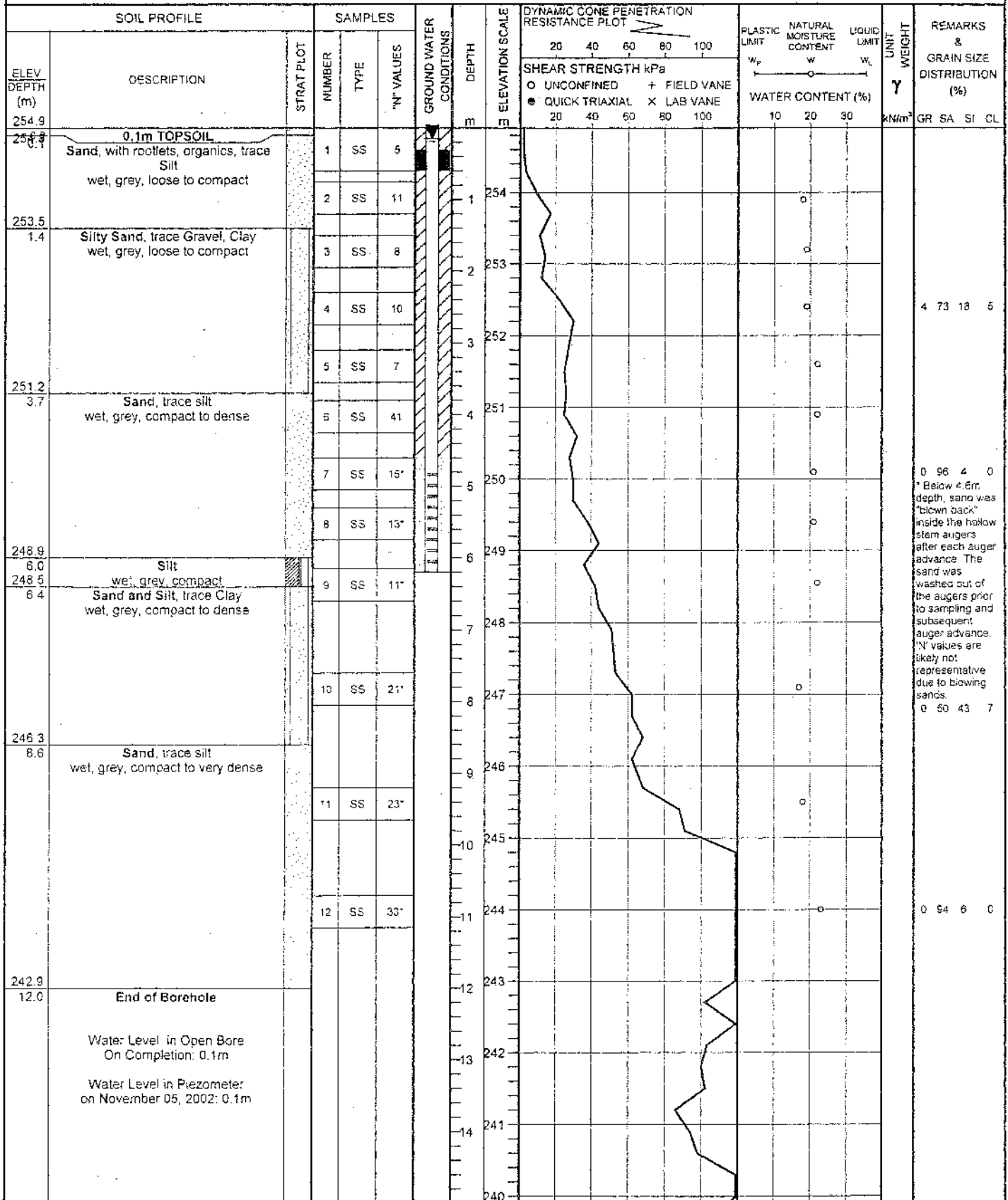


RECORD OF BOREHOLE SHEETS

RECORD OF BOREHOLE No HR1



W.P. 192-00-00 LOCATION 4863532.7N 299710.2E 1 OF 2 ORIGINATED BY FPM
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Augering COMPILED BY IH
 DATUM Geodetic DATE 16 September 2002 - 16 September 2002 CHECKED BY AD
 PROJECT HWY 400 Widening, Vaughan, Ontario JOB NO. TT22852



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No HR1

2 OF 2

W.P. 192-00-00 LOCATION 4863532.7N 299710.2E ORIGINATED BY PPM
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Augering COMPILED BY IH
 DATUM Geodetic DATE 15 September 2002 - 16 September 2002 CHECKED BY AD
 PROJECT HWY 400 Widening, Vaughan, Ontario JOB NO. TT22852

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE				"N" VALUES	20						40
236.6	End of DCPT DCPT carried out about 2m east of borehole					16	239								
18.3						17	238								
						18	237								

RECORD OF BOREHOLE No HR2

1 OF 1

W.P. 192-00-00 LOCATION 4863525.9N 299615.6E ORIGINATED BY IH
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Augering COMPILED BY IH
 DATUM Geodetic DATE 17 September 2002 - 17 September 2002 CHECKED BY AD
 PROJECT HWY 400 Widening, Vaughan, Ontario JOB NO. TT22852

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa						
254.6									20 40 60 80 100						
0.0	Silty Sand, with Gravel, trace clay, rootlets		1	SS	3			254	○ UNCONFINED + FIELD VANE						
254.0	damp, brown, very loose		2	SS	6		1		● QUICK TRIAXIAL x LAB VANE						0 93 7 0
0.6	Sand, trace silt		3	SS	47		2	253							
	wet, brown to grey, loose		4	SS	5*		3	252							
251.6			5	SS	9*		4	251							8 78 (14)
3.0	Silty Sand, trace Gravel, Clay		6	SS	9*		5	250							0 57 27 6
	wet, grey, loose to compact		7	SS	17*		6	249							* Below 2.3m depth, sand was "blown back" inside the hollow stem augers after each auger advance. The sand was washed out of the augers prior to sampling and subsequent auger advance. "N" values are likely not representative due to blowing sands
			8	SS	5*		7	248							
247.6			9	SS	6*		8	247							
7.0	Sand, trace Silt, Clay		10	SS	12*		9	246							
	wet, grey, loose to compact		11	SS	20*		10	245							
243.3							11	244							1 83 10 6
11.3	End of DCPT						12	243							
242.4	DCPT carried out about 2m west of borehole														
12.2	End of Borehole														
	Water Level in Open Bore On Completion : 1.2m September 18, 2002: 0.3m above augers November 05, 2002: 0.2 m above ground														

RECORD OF BOREHOLE No HR3

1 OF 1

W.P. 192-00-00 LOCATION 4863514.9N 299615.6E ORIGINATED BY IH
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Augering COMPILED BY IH
 DATUM Geodetic DATE 17 September 2002 - 18 September 2002 CHECKED BY AD
 PROJECT HWY 400 Widening, Vaughan, Ontario JOB NO. TT22852

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE				SHEAR STRENGTH kPa		w_p	w	w_L		
254.7 0.0	Sand, with Gravel, some Organics, Rootlets		1	SS	3			20 40 60 80 100						
254.0 0.7	damp, brown to grey, very loose		2	SS	8			20 40 60 80 100						
253.3 1.4	Sand, with Gravel moist, grey, loose		3	SS	17			20 40 60 80 100						
	Sand, trace Silt wet, grey, loose to dense		4	SS	50			20 40 60 80 100						
			5	SS	6*			20 40 60 80 100						
			6	SS	7*			20 40 60 80 100						
250.3 4.4	Silty Sand, trace Clay wet, grey, loose to compact		7	SS	5*			20 40 60 80 100						
			8	SS	20*			20 40 60 80 100						
			9	SS	13*			20 40 60 80 100						
			10	SS	19*			20 40 60 80 100						
			11	SS	31*			20 40 60 80 100						
243.1 11.6	CLAYEY SILT, with Sand, trace gravel (TILL)							20 40 60 80 100						
242.5 12.2	moist, gray, hard							20 40 60 80 100						
242.0 12.7	End of DCPT							20 40 60 80 100						
	DCPT carried out about 2m west of borehole							20 40 60 80 100						
	End of Borehole							20 40 60 80 100						
	Water Level in Open Bore On Completion : 0.9 m							20 40 60 80 100						

+ 3 . X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

APPENDIX 'A'

**Borehole Logs extracted from report
prepared by Golder Associates Limited dated May 2001
GEOCRES File No. 30M13-51**

PROJECT 001-1122F		RECORD OF BOREHOLE No 90		1 OF 2		METRIC												
W.P. 222-97-00		LOCATION N 4853549 E 2996599		ORIGINATED BY AZ														
DIST Central HWY 400		BOREHOLE TYPE 108mm I.D. Hollow Stem Augers		COMPILED BY LCC														
DATUM Geodetic		DATE October 16 & 17, 2000		CHECKED BY ASP														
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	T _N VALUES	20						40	60	80	100	10
255.0	GROUND SURFACE																	
254.7	Topsoil (silty clay with organics)																	
254.4	Silty Clay, trace organics Brown																	
254.0	Silty Sand, trace gravel, trace clay, trace organics Compact to dense Brown to grey Wet below 1.5m depth 20mm layer of organics at about 1.8m depth		1	SS	47													
252.8			2	SS	12													
252.2	Sand, trace to some silt Compact to dense Brown becoming grey at 4.5m depth Wet		3	SS	13													
	SPT "N" values are considered to be impacted by blowing sands (See Note 1)		4	SS	3													
			5	SS	7													
			6	SS	8													
			7	SS	15													
			8	SS	12													
	Layer of grey sandy silt, trace clay encountered at 7.6m depth		9	SS	19													
			10	SS	20													
			11	SS	18													
242.5	Probably compact to dense sand																	
241.5																		

Continued Next Page

+ 3 X 3

Numbers refer to
Sensitivity

85%

100% STRAIN AT FAILURE

ON MOT 001-1122F ON MOT 001 19/001

PROJECT <u>001-1122F</u>		RECORD OF BOREHOLE No 90		2 OF 2 METRIC	
W.P. <u>222-97-00</u>		LOCATION <u>N 4803549 # 299699</u>		ORIGINATED BY <u>AZ</u>	
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>105mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>LCC</u>	
DATUM <u>Geodetic</u>		DATE <u>October 16 & 17 2000</u>		CHECKED BY <u>ASP</u>	

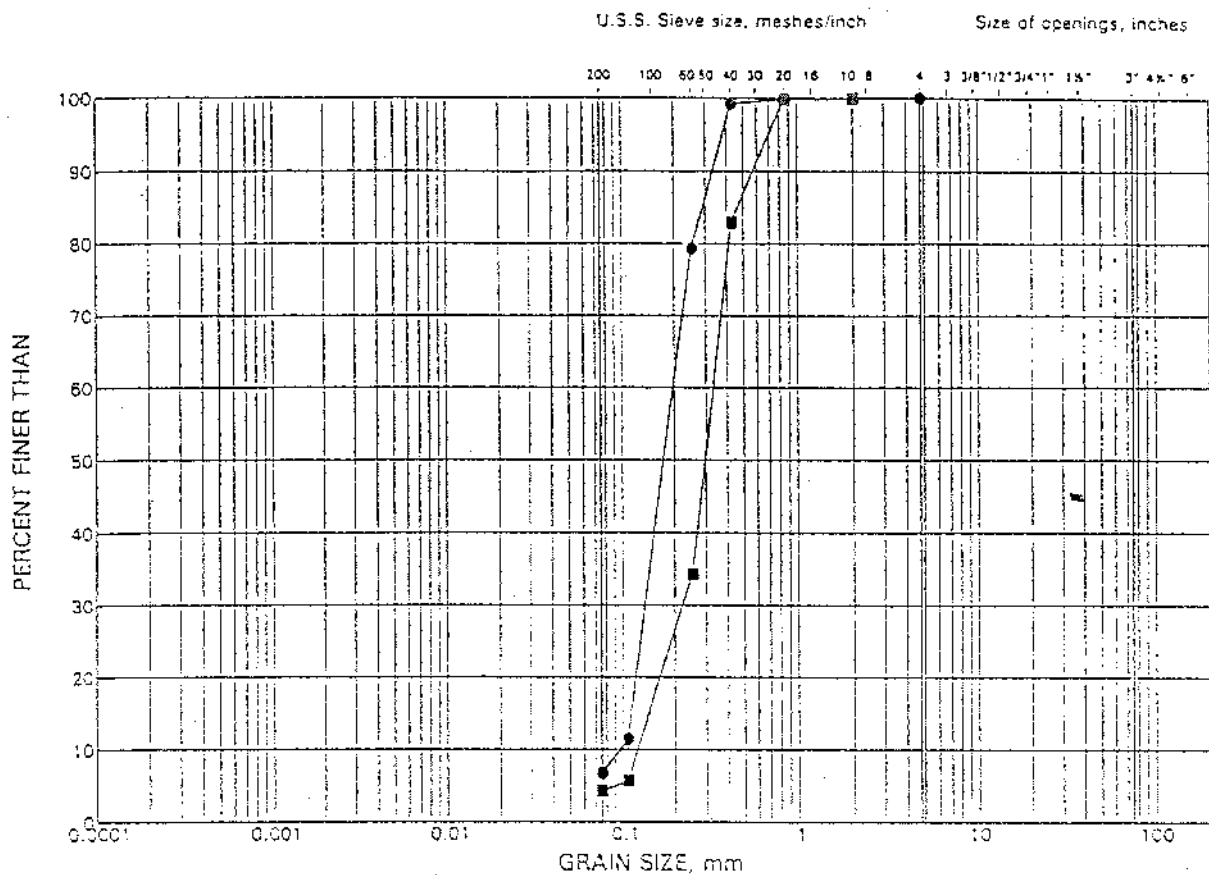
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)		
							20 40 60 80 100	20 40 60 80 100						10 20 30		
— CONTINUED FROM PREVIOUS PAGE —								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED								
	Probably compact to dense sand															
238.2						239										
18.8	END OF BOREHOLE Notes: 1. Below about 2.5m depth, between 1.5m and 3.5m of sand was "blown back" inside the hollow stem augers after each auger advance. This material was washed out of the augers prior to sampling and subsequent auger advance. 2. Water level in open borehole at 1.5m depth (Elev. 253.5m) during drilling and at 1.1m depth (Elev. 253.9m) on completion of drilling. 3. Water level in piezometer at 0.2m depth (Elev. 254.5m) on January 18, 2001.															

ON MOT 001-1122 GPJ ON MOT GOF 19/001

GRAIN SIZE DISTRIBUTION

Sand

FIGURE 1



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	90	5	4.4
■	90	7	6.7

APPENDIX 'B'

Site Photographs



Photo 1: Looking north at west end of Humber River Arch Culvert.



Photo 2: Looking north at east end of Humber River Arch Culvert.