



FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED KIRBY SIDEROAD OVERPASS WIDENING
HIGHWAY 400 INTERIM WIDENING
VAUGHAN, ONTARIO
W.P. 192-00-00, CENTRAL REGION

Submitted to:

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RECORD OF BOREHOLE SHEETS Borehole Numbers: KSR 1 to KSR 5

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GEOCRES File No. 30M13-93

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GEOCRES File No. 30M13-49

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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 replacement and/or widening of the existing Kirby Sideroad Overpass in the City of Vaughan, 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 bridge structure replacement/widening by means of a number of boreholes, in-situ tests and laboratory tests on selected samples. The work carried out for this geotechnical investigation was completed in accordance with AMEC's proposal (ref. P-22280, dated 20 June 2002).

The plan and profile for the proposed bridge replacement/widening and approaches were provided to us by the Ministry of Transportation.

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

- MTO. *Proposed Extension of Kirby Sideroad Overpass*, dated December 15, 1989 – GEOCRE File No. 30M13-93.
- Golder Associates Limited. *Preliminary Foundation Investigation and Design Report, Kirby Sideroad Overpass, 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 June 2001 – GEOCRE File No. 30M13-49.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

2.1 SITE DESCRIPTION

The existing Kirby Sideroad overpass structure is located about 4.5 km north of Major Mackenzie Drive in the City of Vaughan, Regional Municipality of York.

The original ground at the overpass structure appears to be at about Elevation 256 to 258m. The existing Highway 400 grade is at about Elevation 257m, while Kirby Sideroad grade is at Elevation 251 to 252m at the structure location. Kirby Sideroad is in cut with existing side slopes

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at about 2H to 1V and about 5 to 6m in height.

The existing overpass was constructed in the early 1950s under Contract 50-212. It is a single span structure with abutments, wingwalls extending parallel to Highway 400 and retaining walls extending parallel to Kirby Sideroad east of Highway 400. The abutment footings are supported on spread footings founded at about Elevation 249.7m. The retaining walls west of Highway 400 parallel to Kirby Sideroad consist of gabion baskets about 2m in height. In the early 1990s, the overpass was widened on the east side only under Contract 92-95. The widened portion of the structure has also been supported on spread footings at the same elevation as the existing structure. New retaining walls were supported on spread footings on a granular pad which step up from Kirby Sideroad to Highway 400. 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 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 14 and 22 August 2002 and on 16 September 2002, and consisted of drilling and sampling five boreholes (Boreholes KSR 1 to KSR 5, inclusive) to depths of 8.2 m to 22.0 m below the existing ground surface. Boreholes KSR1 and KSR 2 were advanced from the Kirby Sideroad grade, while Boreholes KSR3, KSR 4 and KSR5 were advanced from Highway 400 grade.

Also referenced in this report in Appendix A and B, are boreholes advanced by MTO during a 1989 subsurface investigation (GEOCRE No. 30M13-93, referenced in Section 1.0) and one borehole advanced by Golder Associates in 2001 for a preliminary foundation investigation (GEOCRE No. 30M13-49, referenced in Section 1.0).

Borehole 89, advanced by Golder Associates in 2001, was advanced to a depth of 29.6 m below the existing Kirby Sideroad grade and extended to a depth of about 33.2m by the dynamic cone penetration test method. Boreholes 1, 2 and 3 were advanced by MTO in 1989 to

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depths of 6.9 to 15.7m. Boreholes 1 and 2 were advanced from the Kirby Sideroad grade, while Borehole 3 was advanced from the Highway 400 grade.

The plan locations of the boreholes advanced in the current and previous investigations, and selected stratigraphic sections are shown on Drawing Nos. 1, 2 and 3. 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 solid stem continuous flight augers with track-mounted and truck-mounted power auger drill rigs (CME 55 and 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 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, grain size analyses, Atterberg Limits tests and unit weight determinations, 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, 2, 3, 4 and 5, inclusive.

Groundwater conditions in the current investigation open boreholes were observed throughout and immediately after the drilling operations. A standpipe piezometer was installed in Borehole KSR2, to permit long term monitoring of groundwater levels at the site. All boreholes were adequately backfilled with auger cuttings 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 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.

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As noted by Golder in their report, the borehole northings and eastings of the 1989 investigation by MTO are not consistent with the MTM NAD83 coordinate system currently in use by MTO. The approximate borehole locations have been determined from the borehole location plan in the 1989 MTO report, and their approximate co-ordinates in MTM NAD83 are presented in Drawing No. 1.

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 Appendices A and B.

4.1 Fill

Boreholes KSR1 and KSR2 were advanced on Kirby Sideroad and encountered damp, brown sand and gravel fill to a depth of 0.6m below Kirby Sideroad. Borehole 89 advanced by Golder Associates on Kirby Sideroad encountered 0.6m of sand and gravel fill with trace of clay and organics mixed with the fill. A measured 'N'-value of 30 blows per 0.3m was obtained in the fill indicating a compact to dense relative density. Borehole 2 of the 1989 MTO investigation was also advanced on Kirby Sideroad and encountered sand fill with trace gravel to a depth of 1.8m. Measured 'N'-values within this borehole range from 5 to 6 blows per 0.3m indicating that the fill has a loose relative density, although this fill has likely been removed from the structure widening in the early 1990s.

Underlying the pavement fill in Boreholes KSR1 and KSR2, a clayey silt fill deposit was encountered to a depth of 2.2m. The fill is grey and contains some sand. Measured 'N'-values range from 6 to 14 blows per 0.3m, indicating a firm to stiff consistency. Measured natural moisture contents range from 12 to 13%.

At the ground surface of Borehole KSR4, 0.6m of sandy silt fill was encountered at the top of the cut grade adjacent to Highway 400. This fill contains traces of gravel and rootlets. A measured 'N'-value of 32 blows per 0.3m indicates that this fill has a dense relative density.

It should be noted that the thickness of fill may vary in between and beyond the borehole locations.

4.2 Upper Clayey Silt Glacial Till

Below the surficial fill deposits or at the ground surface in Boreholes KSR1, KSR2, 89, 1 and 2, advanced at the Kirby Sideroad grade, a grey cohesive glacial till deposit was encountered to depths of 7.3 to greater than 9.8m below existing Kirby Sideroad grade (or Elevations 244.3 to at least 240.8m). The glacial till is a heterogeneous mixture of silt and clay with trace to with sands and trace gravel. Occasional sand seams and cobbles are noted within the till. Measured 'N'-values range from 8 to 52 blows per 0.3m indicating a firm to hard consistency. In-situ shear vane tests were carried out in Borehole 1 of the MTO 1989 investigation with shear strengths ranging from 40kPa to greater than 120 kPa, confirming the consistency of the till. Measured natural moisture contents range from 10 to 18%.

Boreholes KSR 3, KSR4, KSR5, and Borehole 3 of the MTO 1989 investigation advanced at the Highway 400 grade, encountered a brown to grey clayey silt glacial till underlying the existing ground surface or surficial fill deposits. The till changes colour from brown to grey at a depth of about 3 to 5m. All four boreholes were advanced at the top of the cut slopes adjacent to Highway 400 and were terminated within the cohesive till at depths of 6.9 to 8.3m (or Elevations 248.0 to 250.9m). Measured 'N'-values range from 10 to 56 blows per 0.3m indicating a stiff to hard consistency. Measured natural moisture contents range from 8 to 17%.

Laboratory tests carried out on the clayey silt till are summarized on the various Record of Boreholes, in Appendices A and B and in Figure Nos. 1 and 2. The laboratory results are summarized below.

Natural Moisture Content	8 to 18%
Plasticity Index	6 to 9%
Plastic Limit	9 to 18%
Liquid Limit	12 to 24%
Unit Weight	20.6 to 23.7 kN/m ³

Grain Size Distribution (5 samples)

Gravel	0 to 8%
Sand	18 to 30%
Silt	48 to 69%
Clay	3 to 17%

The above results are indicative of an overconsolidated low plasticity clay. One grain size has been omitted from the above summary as it was carried out on a sand layer within the till. The grain size distribution curve from the current investigation is presented on Figure No. 1 and the .../...

Atterberg Limits test results for the current investigation are presented on Figure No. 5.

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

4.3 Upper Sands and Silts

Below the surficial fill and cohesive till, a grey, moist to wet, cohesionless interbedded deposit of sands and silts was encountered in Boreholes KSR2, 89, 1 and 2. Boreholes KSR2 and 89 encountered these sands and silts to depths of 10.5 to 11.7m below Kirby Sideroad (or Elevations 240.5 m). Boreholes 1 and 2 advanced in the 1989 MTO investigation were terminated within these deposits at depths of 12.6 to 15.7m (or Elevations 236.2 to 239.1m). The sands and silts contain trace gravel, clay and varying amounts of sand and silt size particles. Measured 'N'-values range from 15 to 75 blows per 0.3 m indicating a compact to very dense relative density, but in general compact to dense. The high 'N'-values measured may be attributed to probable cobbles and / or boulders. It is noted that the cobbles and boulders could not be sampled with the spoon sampler. One low value of 4 blows per 0.3m was obtained in Borehole 2, however based on the characteristics of this deposit this may be attributed to disturbance. Measured natural moisture contents range from 12 to 20%.

An Atterberg Limits test was carried out on a sample of the silt and the results are presented on Figure No. 5. The results indicate a liquid limit of 21% and a plastic limit of 19%.

4.4 Middle Clayey Silt to Sand and Silt Glacial Till

Underlying the upper sands and silts in Boreholes 89 and KSR2, at a depth of about 10.5 to 11.7m below Kirby Sideroad (or Elevations 240.5 m), a second grey glacial till deposit was encountered. The till ranges in composition from clayey silt at Borehole KSR2 to sand and silt at Borehole 89. The glacial till is a heterogeneous mixture of clay and silt with sand and trace gravel size particles. This "middle" till deposit was encountered to a depth of about 11.9 to 13.3m (or Elevation 238.9 to 239.1m). Measured 'N'-values of 20 and 61 blows per 0.3m was obtained within the till indicating a hard consistency or compact relative density. For the clayey silt till, a measured moisture content of 13% was obtained and a unit weight of 22.7 kN/m³ was measured, in the current investigation. A grain size analysis was carried out on a sample of the clayey till and the grain size is presented in Figure No. 2. The results indicate 2% gravel, 30% sand, 48% silt and 20% clay.

It should be noted that Borehole 1, Sample 12 from the 1989 MTO investigation (at Elevation 239.5m or a depth of 12.2m) may be a sandy silt till, as seen from the grain size distribution analysis (15% sand, 76% silt and 9% clay) and slight plasticity indicated on the Record of .../...

Borehole. A measured 'N'-value of 55 blows per 0.3m was obtained at Sample 12 which indicates a very dense relative density.

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

4.5 Middle Sands and Silts

Underlying the middle till deposit and upper silts and sands, a second interbedded grey, wet, silt to silty sand and minor clayey silt deposit was encountered in Boreholes 89 and KSR2. Borehole KSR 2 was terminated within this deposit at a depth of about 22.0m (or Elevation 230.2m). Borehole 89 encountered this deposit to a depth of about 21.3m (or Elevation 229.7m). Measured 'N'-values range from 33 to greater than 100 blows per 0.3m indicating a dense to very dense relative density. Measured natural moisture contents range from 12 to 19%. One grain size analysis was carried out on a sample of the silty sand from Borehole 89 with results indicating 58% sand, 39% silt and 3% clay size particles. A grain size analysis was carried out on a sample of the sandy silt and the results are presented in Figure No. 4. The results indicate 13% sand and 87% fines, with less than 8% clay.

In Borehole KSR2, underlying the middle clayey silt till, a grey, hard clayey silt deposit was encountered to a depth of about 14.8m. An Atterberg Limits test was carried out on this layer and the results are presented on Figure No. 5. The results indicate a liquid limit of 26% and a plastic limit of 18%. A unit weight of a split spoon sample obtained with a result of 21.0 kN/m³. A grain size analysis was also carried out on the sample, with results presented on Figure No. 3, indicating 1% sand, 68% silt and 31% clay.

4.6 Lower Silty Clay Glacial Till

Underlying the second interbedded sand and silt deposit in Borehole 89 is a grey silty clay glacial till that was encountered at a depth of 21.3m (Elevation 229.7m) and was contacted to a depth of about 24.1m (Elevation 226.9m). Measured 'N'-values of 34 and 109 blows per 0.3m were obtained indicating a hard consistency, however the consistency appears to decrease with depth. A measured natural moisture content of 14% was obtained within this deposit.

4.7 Lower Sands and Silts

Below the lower (third) glacial till deposit in Borehole 89 is a grey, wet, silt to sand and silt deposit. The borehole was terminated within this deposit at a depth of about 29.6m, or Elevation 221.4m. The deposit contains occasional silty clay seams. Measured 'N'-values range from 1 to 27 blows per 0.3m, however in our opinion, these values are likely the result of disturbance and

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the relative density of the deposit is probably much higher. Measured natural moisture contents range from 20 to 25%.

4.8 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 Borehole 89 within the lower sands and silts and in Borehole KSR2 within the middle sands and silts. At Borehole 89 the water level in the piezometer was 0.5m above ground surface 3 months after installation. At Borehole KSR 2 the water level in the piezometer was 0.1m below existing ground surface one month after installation.

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 approaches lies within the upper clayey silt till, about 1 to 2m above the Kirby Sideroad grade (at about Elevation 252 to 253m), dropping to the ground surface level at Kirby Sideroad (Elevation 251 to 252m). Excessive hydrostatic pressure (artesian to sub-artesian) was observed from the piezometer installations within the middle and lower sands and silts.

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. 6-2) prepared by McCormick Rankin Corporation dated June 2002, the existing Kirby Sideroad Overpass that carries the NBL and SBL of Highway 400 over Kirby Sideroad consists of a single span bridge, 34.45m wide with a span of 11.325m. The proposed design consists of widening the existing structure 8.47m to the east and 12.34 m to the west. We also understand that the existing road profile of Kirby Sideroad will be lowered by up to 400 mm at the structure location.

In general, the subsoils along the proposed bridge widenings consists of an upper stiff to very stiff clayey silt till underlain by water-bearing interbedded deposits of compact to very dense sands and silts with minor interlayers of hard to compact/very dense glacial till. The water bearing sands and silts extend to depths of at least 30 m below Kirby Sideroad grade. The water bearing sands and silts are under artesian pressure at depth.

The approaches are composed of about 5 to 6m high cuts within the clayey silt till and have side slopes of about 2H to 1V. The till within the cuts has a consistency of very stiff to hard.

5.1 Bridge Foundations

Based on the proposed design and subsurface conditions encountered in the boreholes, two options are available for the foundations of the proposed bridge structure:

- i) Shallow foundations – spread footings on native subgrade
- ii) Deep foundations – pile foundations.

5.1.1 Spread Footings

It is recommended that after dewatering the site, all the organic and otherwise unsuitable soils be removed to the surface of the inorganic stratum. The excavation should then be extended within a sufficient horizontal distance (i.e. 2m) beyond the proposed footing perimeter, to the surface of the competent soil.

Based on the data obtained from the boreholes, the proposed structure widening can be supported on spread footing foundations placed on the undisturbed stiff to very stiff clayey silt till. The existing bridge foundations are founded at Elevation 249.7m and will therefore control the founding level of the proposed widenings.

The following highest founding depths (Table I) and soil resistances are recommended at the borehole locations for footings placed in undisturbed, competent natural soils.

TABLE I

BOREHOLE / LOCATION	EXISTING GROUND SURFACE ELEVATION (m)	RECOMMENDED FOUNDING LEVEL ELEVATION (m)	DEPTH TO FOUNDING LEVEL (m)	BEARING RESISTANCE AT S.L.S. (kPa)	FACTORED BEARING RESISTANCE AT U.L.S. (kPa)	SUBGRADE MATERIAL
89 North Abutment - West Side	251.0	249.7	1.3	200	350	Stiff to very stiff Clayey Silt Till
KSR1 South Abutment - West Side	250.6	248.4	2.2	200	350	Very stiff Clayey Silt Till
1 North Abutment - East Side	251.7	249.7	2.0	200	350	Stiff Clayey Silt Till
2, KSR2 South Abutment - East Side	252.2	250.0	2.3	200	350	Stiff to very stiff Clayey Silt Till

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.

Prior to the excavation it should be ensured that the site was dewatered below the proposed subgrade excavation level. After the excavation reaches the required elevation, the subgrade should be evaluated and approved by the Quality Verification Engineer. If necessary, the excavation may need to be deepened to the surface of sufficiently competent soil. The exposed subgrade may need to be compacted from the surface to achieve a density of not less than 98% of the material's Standard Proctor Maximum Dry Density (SPMDD).

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For frost protection, the footings should have a permanent earth cover of at least 1.2m. However, this may not be possible due to the grade lowering of Kirby Sideroad. If less than 1.2m of earth cover will be placed over the proposed and existing foundations, then the proposed and existing foundations could be protected against frost by placing artificial insulation with a thermal equivalent to 1.2m of earth cover and/or a suitable combination of artificial insulation and earth cover.

In the above quoted values for spread footing foundations the serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively, provided that the founding subgrade is undisturbed during the construction.

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 approved till surface can be calculated using a friction angle of 22 degrees.

Care should be taken to avoid damaging the existing foundations. Excavation below the founding level of the existing foundation is not recommended as this will require underpinning of the existing footings.

Low soil bearing pressures are available within the till at shallow depth, with higher bearing pressures available at depths of 4 to 6m below Kirby Sideroad grade. Replacement of the weaker till with compacted granular would require sub-excavation to depths of 4 to 6m below existing ground surface, which would require shoring or replacement of the existing bridge structure.

5.1.2 Pile Foundations

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

If the existing bridge is to be replaced, consideration should be given to replacing the existing bridge with a structure supported by deep foundations in the form of steel H-piles. The piles should be driven into the very dense middle sand and silt deposits at about Elevation 238.5m \pm . The piles should not be driven deeper as the relative density of the sands and silts decreases

with depth and the lower clayey till should not be penetrated as the lower sands and silts appear to be under artesian pressure, as reported from Borehole 89.

In order to adequately penetrate into the overburden, a heavier section such as HP 310 x 110 equipped with reinforced tips (driving shoes as per Ontario Provincial Standards Drawing Number: 3301.00) would be suitable for use.

Based on the results of the boreholes, the following Table 2 summarizes the estimated average pile tip elevations that may be assumed for design purposes.

TABLE 2
ESTIMATED APPROXIMATE PILE TIP LEVEL

SUPPORT LOCATION	REFERENCE BOREHOLE	ESTIMATED APPROXIMATE PILE TIP LEVEL (m)		FOUNDING STRATUM
		ELEVATION	DEPTH	
South Abutment	KSR2	238.2 ±	14 ±	Hard Clayey Silt
North Abutment	89	237 ±	14 ±	Very dense Sand and Silt

The above estimated pile tip elevations are based on the assumption that the piles would penetrate the hard clayey till and the compact to very dense upper sands and silts into the very dense middle silts to sands and silts to hard clayey silt. The above estimated pile depths are also based on the elevations of the existing ground surface during the time of investigation.

5.1.3 Resistance to Axial Loads

For HP 310 x 110 steel H-piles driven to Elevation 237 to 238 m± within the very dense cohesionless deposits at or below the elevations shown in Table 2 above, the following axial resistances may be assumed for design.

- Factored Axial Resistance at Ultimate Limit States = 1,100 kN, with an applied resistance factor of 0.5.
- Geotechnical Resistance at Serviceability Limit States = 800 kN.

If Hiley Formula is used, the estimated ultimate resistance of the piles driven to about the elevations quoted in Table 2, is approximately 2,200 kN. The piles should be driven with a

suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ per blow. The energy should, however, be restricted to not more than 60 kJ per blow.

Cobbles and / or boulders may be encountered within the subsoils, therefore, as mentioned before, the piles should be equipped with reinforced tips as per Ministry of Transportation of Ontario's Standards (Ontario Provincial Standards Drawing Number: 3301.00). Consideration may be given to pre-augering along the pile vertical alignment to an elevation of about 2.0 m above the pile tip elevations (in Table 2) prior to driving the piles but pre-augering should only be used as a last resort measure.

In accordance with Ministry of Transportation of Ontario's standard practice, the piles should be driven to about 2.0 m to 3.0 m above the design elevations as given in Table 2 and the driving should then be monitored and controlled by the Hiley Formula. The Contractor should be aware that a Quality Verification Engineer should be retained to provide piling inspections and a certificate of conformance, as per SP902S01.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 10% of the piles and at least two piles in each foundation support element be re-struck no sooner than 24 hours after initial installation, as a precaution against relaxation, as per SP903S04. If relaxation occurs, then all piles in that foundation element should be re-tapped.

The geotechnical resistance at Serviceability Limit States is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. The pile group configuration is currently not available to us. Provided that the piles are designed and installed as recommended above, it is considered that the quoted Serviceability Limit States value corresponds to no more than 25 mm of settlement for the pile group. We can confirm the estimated settlement once information on the pile group configuration is known.

5.1.4 Resistance to Lateral Loads

Laterally applied loads on piles can be resisted geotechnically by the driven piles through passive earth pressure developed in the soil in which the piles are embedded. The pile tip elevations recommended above indicate that the piles will be in the order of 14 m in length. Lateral pile resistance may be considered in accordance with Section 6.8.7 of the C.H.B.D.C. (CAN/CSA-S6-00).

The recommended horizontal resistances for a HP 310 x 110 pile at this site are as follows:

.../...

Factored Horizontal Resistance at Ultimate Limit States = 120 kN
Horizontal Resistance at Serviceability Limit States = 50 kN

In accordance with Ministry of Transportation of Ontario's requirements (Ministry of Transportation of Ontario Structural Office Standard), piles for integral abutments require a 3.0m long flexible zone. In essence the current Ministry of Transportation of Ontario's standards for the flexible zone consists of an annular space in between two concentric corrugated steel pipe culverts. One of the corrugated steel pipe culvert surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile), while the second corrugated steel pipe culvert has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile. The annular space in between the two corrugated steel pipe culverts is the 3.0 m long flexible zone. After the pile is driven, the space between the H-pile and the inner corrugated steel pipe culvert is filled with coarse sand. A non-standard special provision should be included in the contract document detailing the gradation of the sand backfill as follows:

SIEVE SIZE	PERCENTAGE PASSING
2.0 mm	100%
0.6 mm	80% to 100%
0.425 mm	40% to 80%
0.25 mm	4% to 25%
0.15 mm	0% to 6%

If conventional abutments on pile groups are to be built instead of integral abutments, then the unbalanced horizontal forces could be partially resisted by battered piles. Conventional pile groups at the piers should also have battered piles to resist horizontal forces.

For lateral soil-pile interaction analysis, in cohesionless soils, the horizontal subgrade reaction to the pile can be calculated from the expression:

... where $k_s = n_h \times z/d$
 k_s = coefficient of horizontal subgrade reaction
 n_h = coefficient related to soil density as given in Table 3
 d = pile width
 z = depth

In cohesive soils the coefficient of horizontal subgrade reaction may be estimated from:

... where $k_s = 67 c_u / d$
 k_s = coefficient of horizontal subgrade reaction
 c_u = undrained shear strength of the soil as given in Table 3

d = pile width

Also presented in Table 3 are the estimated values for angle of internal friction and bulk unit weights.

TABLE 3

REFERENCE BOREHOLE	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (m)	APPROX. ELEVATION (m)	SIMPLIFIED SOIL TYPE	ESTIMATED BULK UNIT WEIGHT (kN/m ³)	ESTIMATED ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	ESTIMATED UNDRAINED SHEAR STRENGTH (kPa)	ESTIMATED n_h VALUE (MN/m ³)
South Abutment							
KSR2	2 to 8	250 to 244	Very stiff to hard clayey silt till	22.5	--	100	--
	8 to 12	244 to 240	Compact to very dense silt	21.0	32	--	5
	12 to 13	240 to 239	Hard clayey silt till	23	--	200	--
	13 to 14	239 to 238	Hard clayey silt	21.0	--	200	--
North Abutment							
89	2 to 7	250 to 245	Very stiff clayey silt till	22.5	--	100	--
	7 to 10	245 to 242	Compact to dense sand and silt	21.0	32	--	5
	10 to 12	242 to 240	Compact sand and silt till	22.5	35	--	8
	12 to 13	240 to 239	Very dense sand and silt	21.0	35	--	11

It is recommended that all surficial organics and other deleterious materials within the footprints of the abutments and approaches be sub-excavated.

Where conventional piled foundations are used, there is likely more than one row of piles. In this instance, group action for lateral loading should be considered when the pile spacing in the direction of loading is less than six to eight times pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor R as summarized in Table 4 below.

.../...

TABLE 4

PILE SPACING IN DIRECTION OF LOADING d = PILE DIAMETER	SUBGRADE REACTION REDUCTION FACTOR, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.2 Retaining Wall Foundations

Retaining walls are proposed parallel to Kirby Sideroad and parallel to Highway 400. The subsurface conditions along the slope consist of stiff to hard clayey silt till and no groundwater seepage was observed along the slope. Based on the results of the investigation, a number of design options in constructing the proposed retaining wall were considered at this site. 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 5.

Table 5: 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 5. 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 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. The recommended base widths for the gabion wall range from about 1.0 to 2.0 m for wall heights of 2 to 5m.

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.

.../...

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

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

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

Due to the limited space caused by the existing cut slope and variable wall height (even less than 2 m), this option may not be considered feasible.

5.2.4 Retaining Wall Foundations

From the findings of the boreholes, the footings for the retaining wall can be founded on the competent native stiff to hard clayey silt till.

For design purposes the following C.H.B.D.C. bearing resistances may be used for the following subgrades:

BOREHOLE / LOCATION	EXISTING GROUND SURFACE ELEVATION (m)	RECOMMENDED FOUNDING LEVEL ELEVATION (m)	DEPTH TO FOUNDING LEVEL (m)	BEARING RESISTANCE AT S.L.S. (kPa)	FACTORED BEARING RESISTANCE AT U.L.S. (kPa)	SUBGRADE MATERIAL
3 North East Slope	258	257	1.2	350	500	Hard Clayey Silt Till
		255	3.0	200	350	Stiff to Very Stiff Clayey Silt Till
KSR3 North West Slope	256.4	255	1.2	350	500	Hard Clayey Silt Till
KSR4 South West Slope	256.6	255	1.5	250	400	Very Stiff Clayey Silt Till
		253	3.0	350	500	Hard Clayey Silt Till
KSR5 South East Slope	256.1	255	1.2	350	500	Hard Clayey Silt Till

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

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 till is 22 degrees. For gabions, the sliding resistance over the till is 20 degrees.

Footings 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. It should be noted that the existing retaining wall footings from the previous contract may still be buried at the site. The contractor should be aware that their removal may be required.

5.2.5 Recommended Options

From the options presented above, Option 3 (RSS Wall) is considered not feasible within the present restricted space.

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

For the retaining wall:

- 1. Option 1: Reinforced Concrete Retaining Wall**

This option involves construction of concrete footings. This option is recommended for high retaining walls, which could be about 5 m high for this project.

- 2. Option 2: Gabion Retaining Wall**

This option involves construction of a flexible structure, and is considered cost effective for walls up to about 5 m in height.

Considering the height of the proposed wall (maximum 5 m high), **Option 1**, reinforced concrete retaining wall, is considered the most suitable option at this site. Geotechnical recommendations for retaining wall foundations are presented in Section 5.2.4.

5.3 Lateral Earth Pressures

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.

5.4 Approach Cuts

Kirby Sideroad is constructed in cut with slopes are at an inclination of about 2H to 1V and about 6m in height.

Boreholes 3, KSR3, KSR4, and KSR 5 show that the subsurface conditions within the cut consist of stiff to hard clayey silt till. Cut slopes of 2H to 1V, would be stable against deep-seated (i.e. foundation) failures, provided that the slope is not disturbed during construction.

Site development and construction activities should be conducted in a manner which does not result in surface erosion on the slope. Final site grading and drainage (including surface

.../...

drainage) should be designed to prevent direct concentrated or channelized surface from flowing directly over the slopes.

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 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 fills and firm to hard clayey silt tills. Open cut excavations can be expected to stand temporarily at 1H to 1V side slopes. Pumping from properly filtered sumps may be required to control water seepage due to perched water and surface runoff. The base of the excavation should be graded towards a sump pump in order to drain any surface water inflow into the excavation and from severe weather events in order to avoid excessive mucking of the base.

No major excavation difficulties are foreseen but allowance should be made for boulders and cobbles which occur randomly in glacial deposits.

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. 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 silty subgrade of the existing foundations during excavation.

It should also be noted that pile driving should not be carried out below Elevation 227m due to artesian groundwater conditions below this elevation.

5.6 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.

5.7 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 satisfied. 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

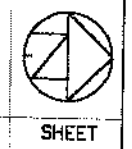


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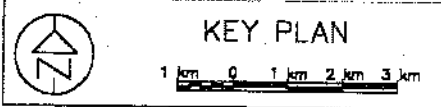
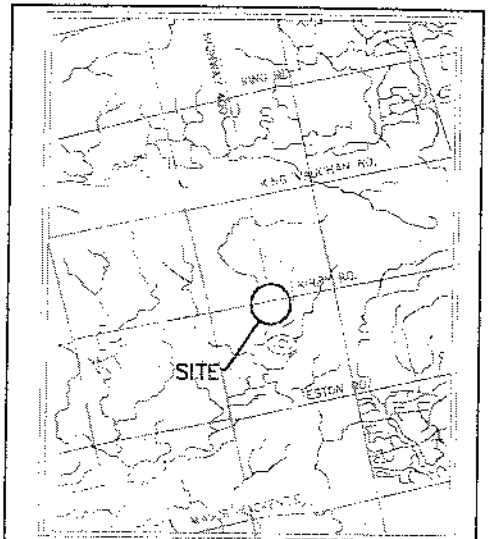
DRAWINGS

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

W.P. No. 192-00-00
 KIRBY SIDEROAD OVERPASS
 HIGHWAY 400
 BORE HOLE LOCATIONS & SOIL STRATA



AMEC Earth & Environmental Limited



LEGEND

- Bore Hole - this investigation
- ⊕ Bore Hole & Cone - previous investigations done by others
- "N" Blows/0.3m (Std Pen Test, 475 J/blow)
- "CONE" Blows/0.3m (80° Cone, 475 J/blow)
- ≡ WL at time of investigation
- ≡ WL in Piezometer
- ⊕ Piezometer
- ⊥ End of Borehole

No	ELEV	CO-ORDINATES NORTH EAST
1	251.7	4 860 443 300 207
2	251.9	4 860 434 300 268
3	257.8	4 860 459 300 209
89	251.0	4 860 436 300 167
KSR1	250.6	4 860 423 300 165
KSR2	252.2	4 860 438 300 217
KSR3	258.4	4 860 451 300 163
KSR4	258.6	4 860 402 300 172
KSR5	258.1	4 860 413 300 217

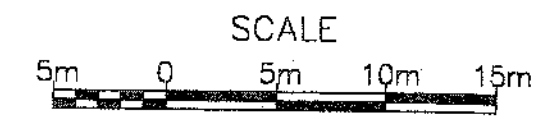
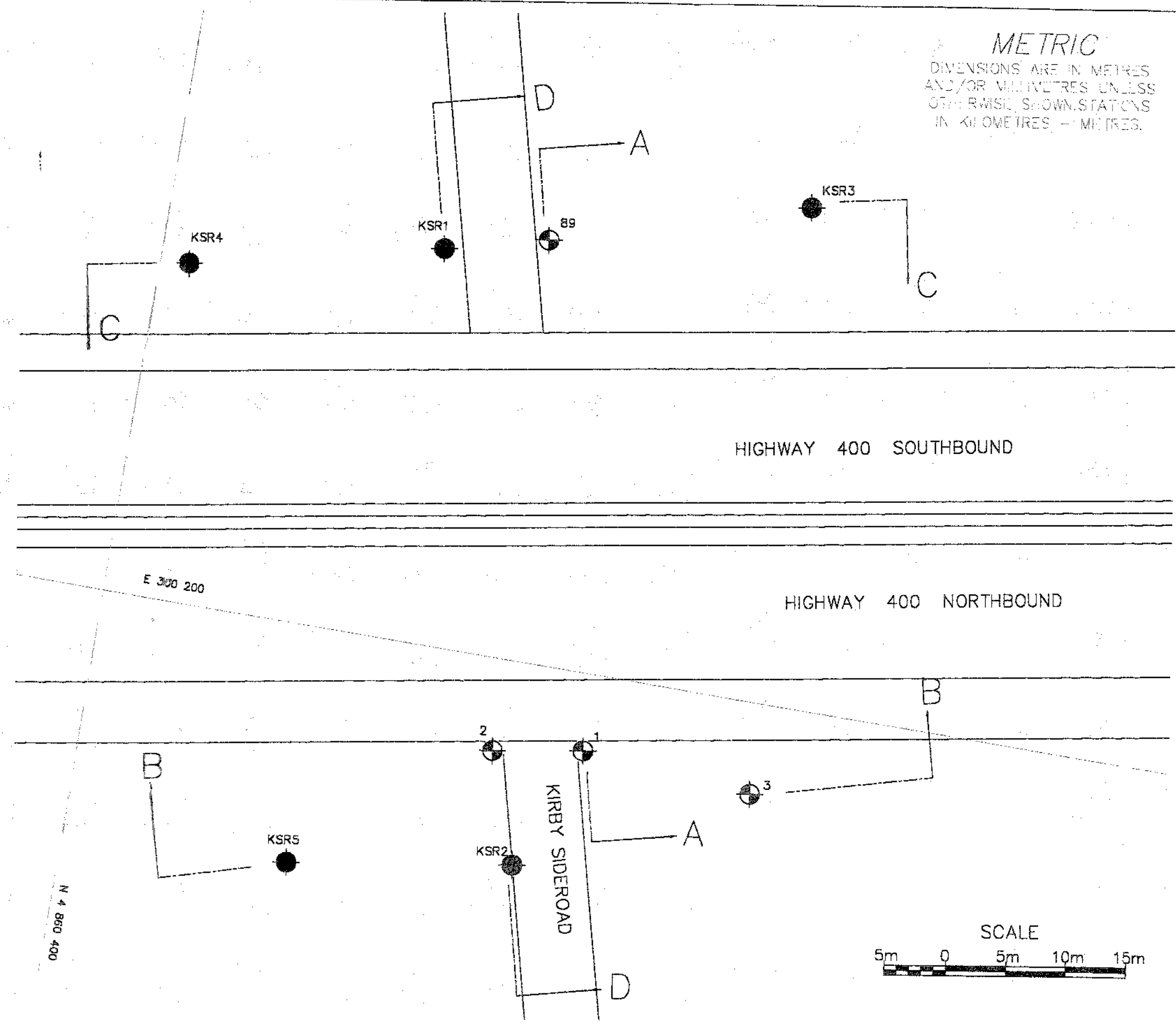
-NOTE-

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
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HWY No 400	DIST
SUBM'D FOR CHECKED AD DATE Sept. 2002	SITE
DRAWN VK CHECKED KSM APPROVED	DWG 1

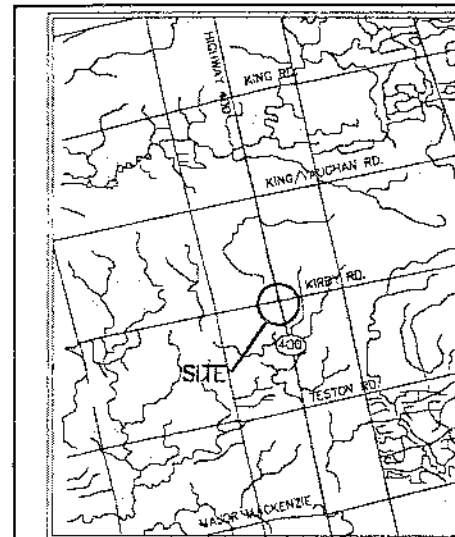


W.P. No. 192-00-00

KIRBY SIDEROAD OVERPASS
HIGHWAY 400
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

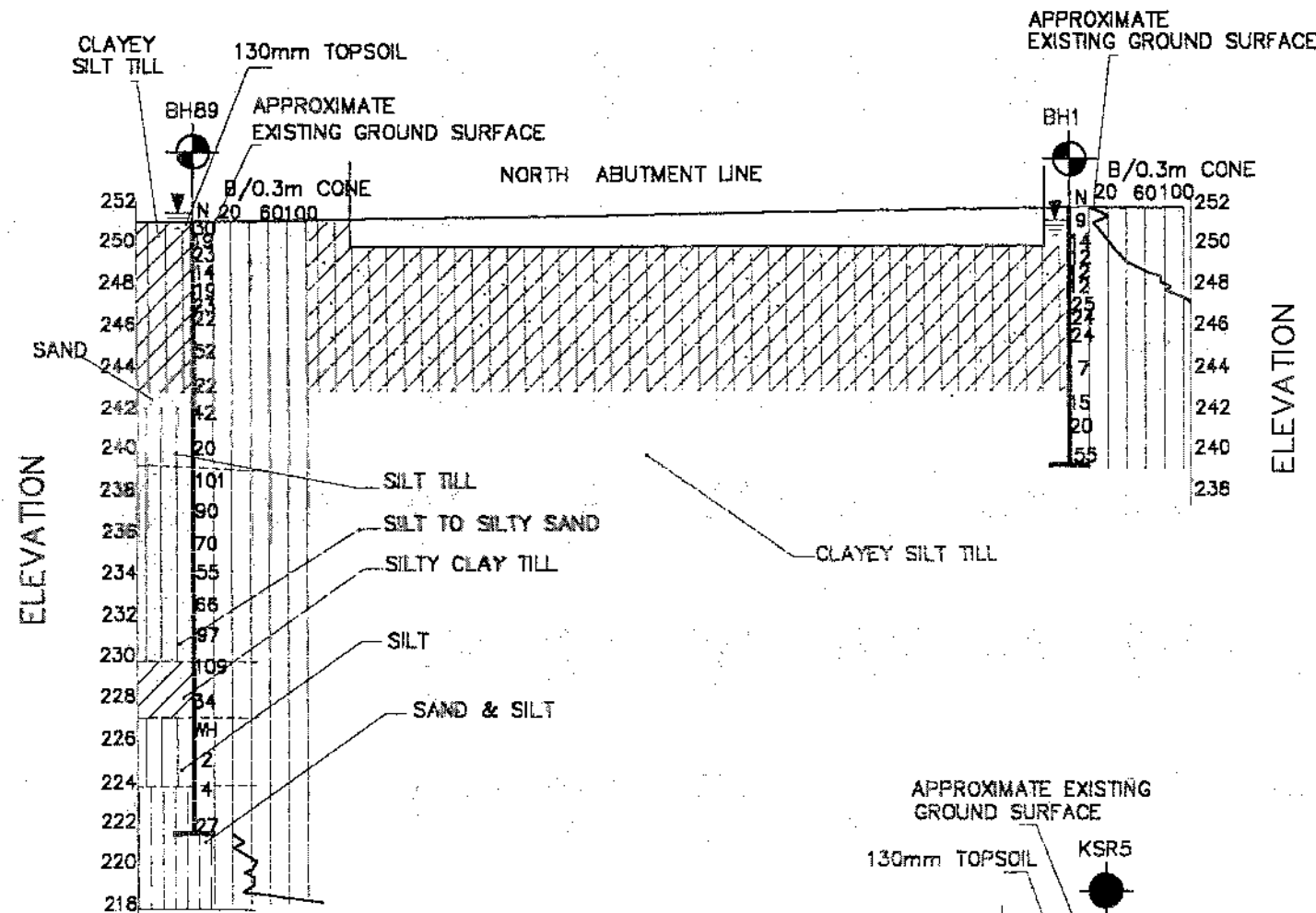
AMEC Earth & Environmental Limited



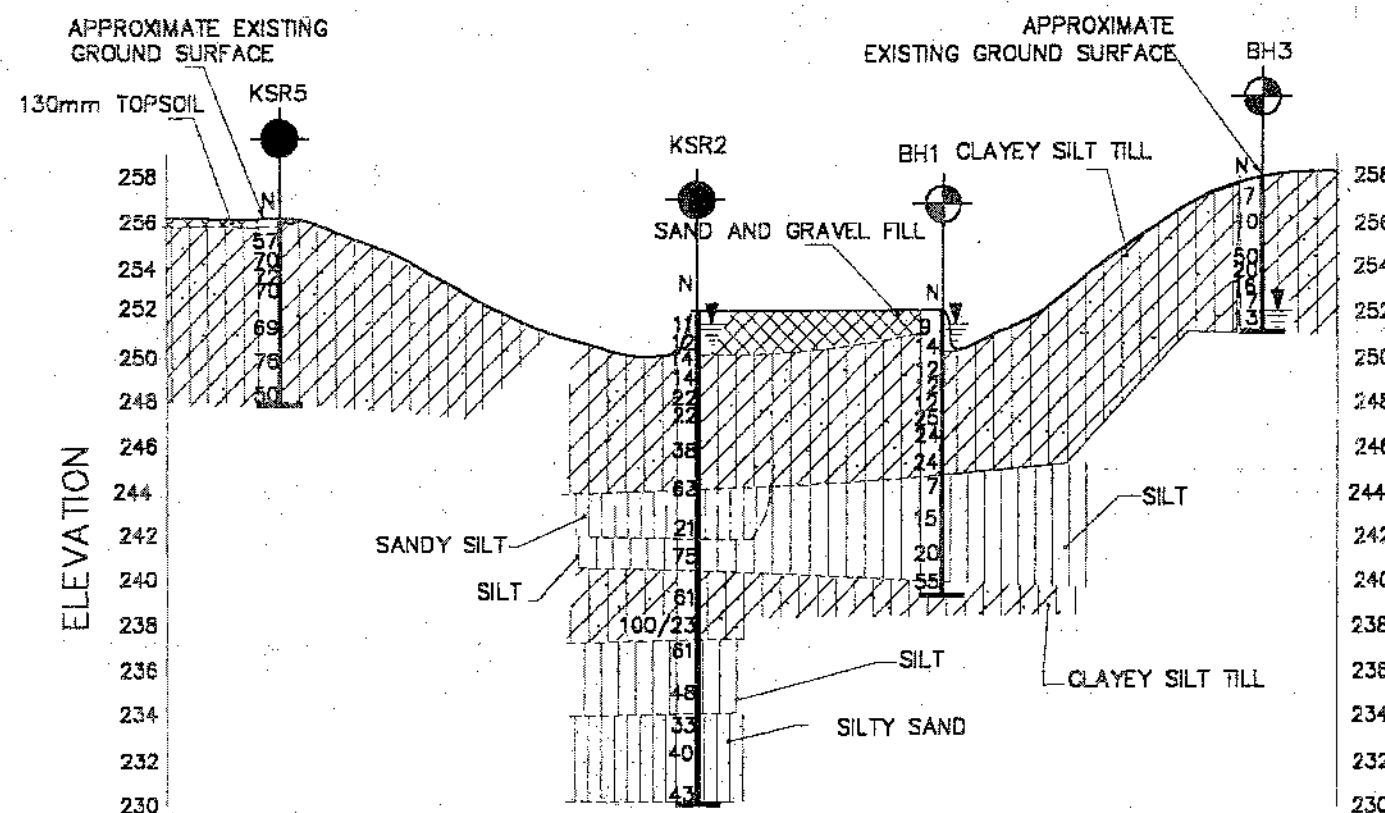
KEY PLAN
1 km 0 1 km 2 km 3 km

METRIC

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



SECTION A-A



SECTION B-B

SOIL STRATIGRAPHY LEGEND

- | | |
|--|---|
| SAND and GRAVEL FILL | SILTY SAND WITH CLAY AND SAND LENSES
Compact to very Dense |
| SAND SOME GRAVEL, TRACE SILT, OCCASIONAL COBBLES
Very Loose to Very Dense | SANDY SILT, TRACE CLAY, TRACE GRAVEL
Loose to Compact |
| CLAYEY SILT TILL
Stiff to Hard | SILTY SAND, TRACE GRAVEL, OCCASIONAL COBBLES
Compact to Very Dense |

LEGEND

- Bore Hole - AMEC Investigation
- ⊕ Bore Hole & Cone - previous investigations done by others
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- CONC Blows/0.3m (30' Cone, 475 J/blow)
- W. at time of investigation -
- W. in Piezometer
- Piezometer
- End of Borehole

No	BEV	CO-ORDINATES NORTH	EAST
1	251.7	4 860 443	300 207
2	251.8	4 860 434	300 208
3	257.8	4 860 458	300 209
89	251.0	4 860 438	300 187
KSR1	250.6	4 860 423	300 185
KSR2	252.2	4 860 438	300 217
KSR3	258.4	4 860 451	300 183
KSR4	256.6	4 860 402	300 172
KSR5	258.1	4 860 413	300 217

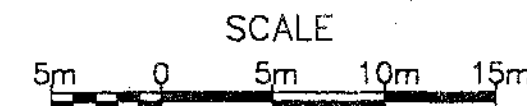
-NOTE-

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REV	DATE	BY	DESCRIPTION
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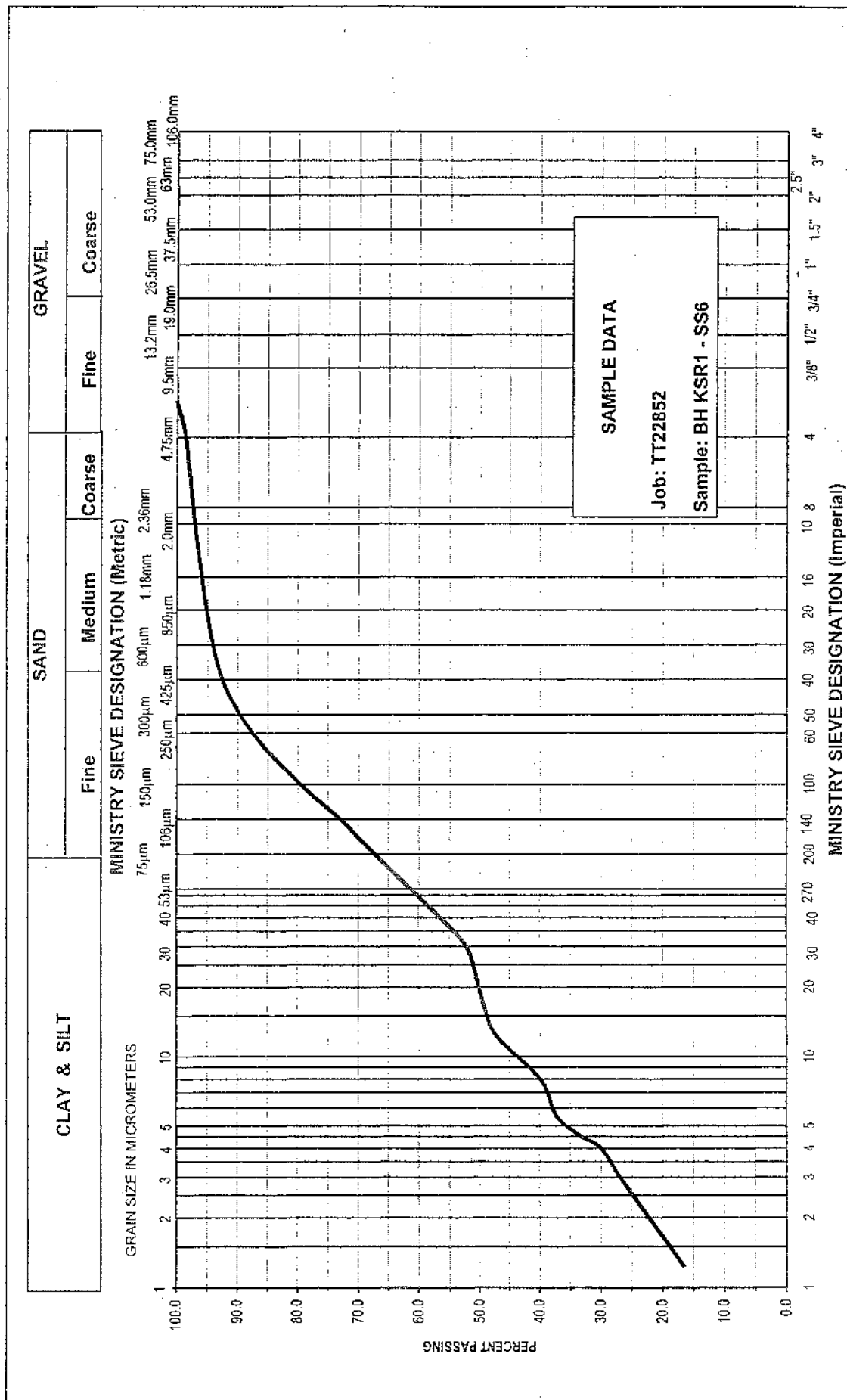
HWY No 400	DIST	
SUBMIT FOR CHECKED AD	DATE Sept, 2002	SITE
DRAWN VK	CHECKED KSH	APPROVED DWG 2



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FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM



AMEC Earth & Environmental Limited 104 Crookford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6505, Fax +1 (416) 751 7592 www.amec.com	GRAIN SIZE DISTRIBUTION	Client :- Ministry of Transportation Ontario
		Project:- Highway 400 Interim Widening
	CLAYEY SILT TILL	Location:- Vaughan, Ontario
	with Sand trace Gravel	Depth :- 4.6 m
		Date :- 10-Sep-2002

FIGURE NO. 1

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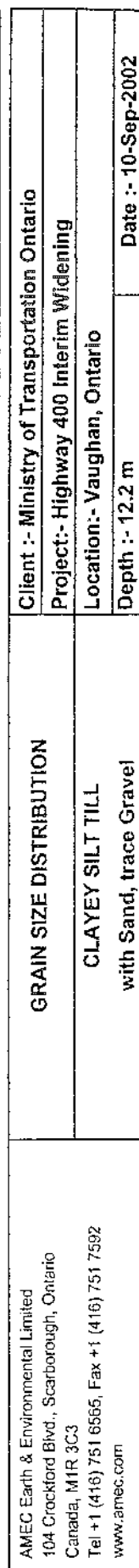
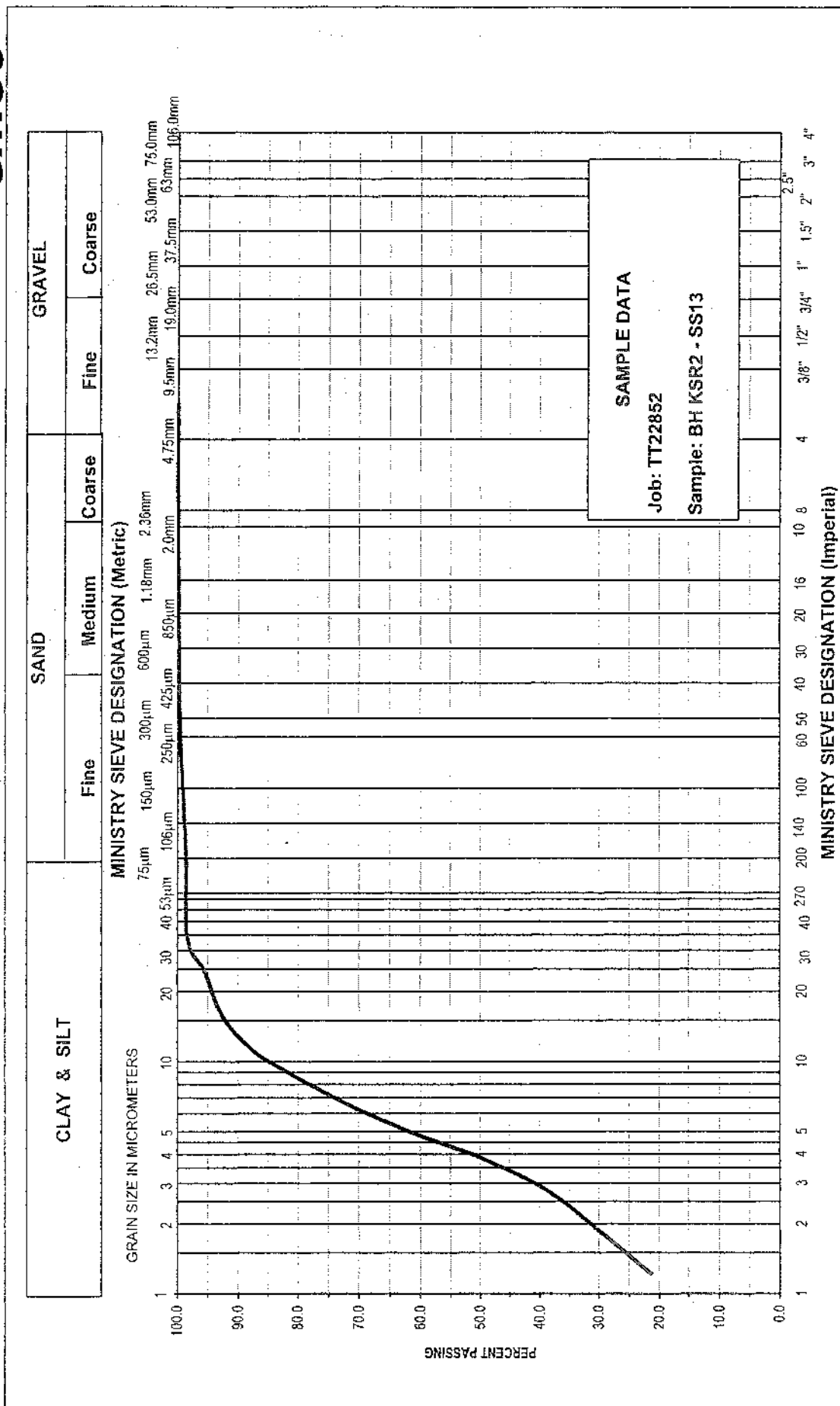


FIGURE NO 2

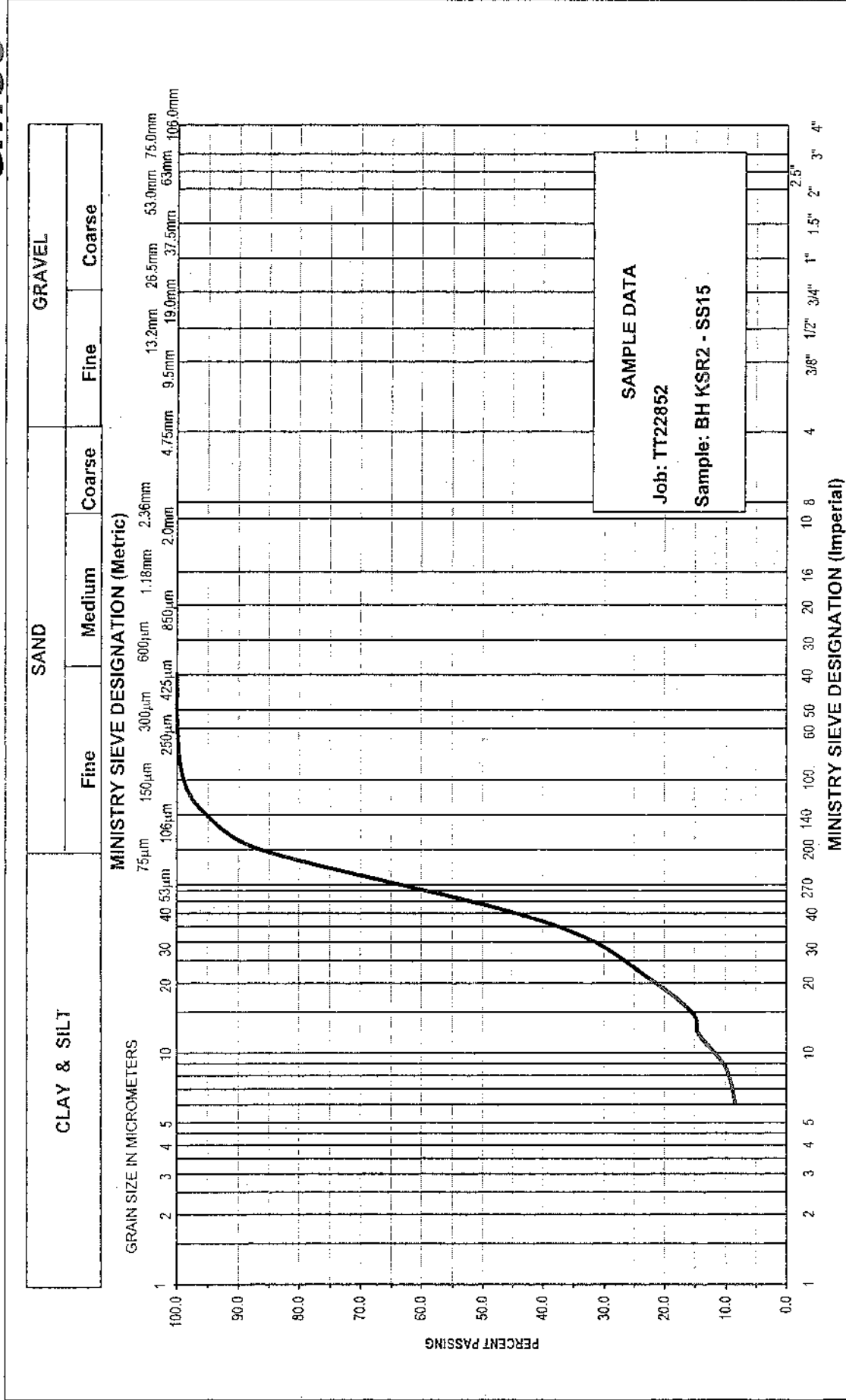
UNIFIED SOIL CLASSIFICATION SYSTEM



AMEC Earth & Environmental Limited 104 Crockford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 8565, Fax +1 (416) 751 7592 www.amec.com	GRAIN SIZE DISTRIBUTION		Client :- Ministry of Transportation Ontario	
	CLAYEY SILT		Project:- Highway 400 Interim Widening	
	trace sand		Location:- Vaughan, Ontario	
			Depth :- 13.7 m Date :- 10-Sep-2002	

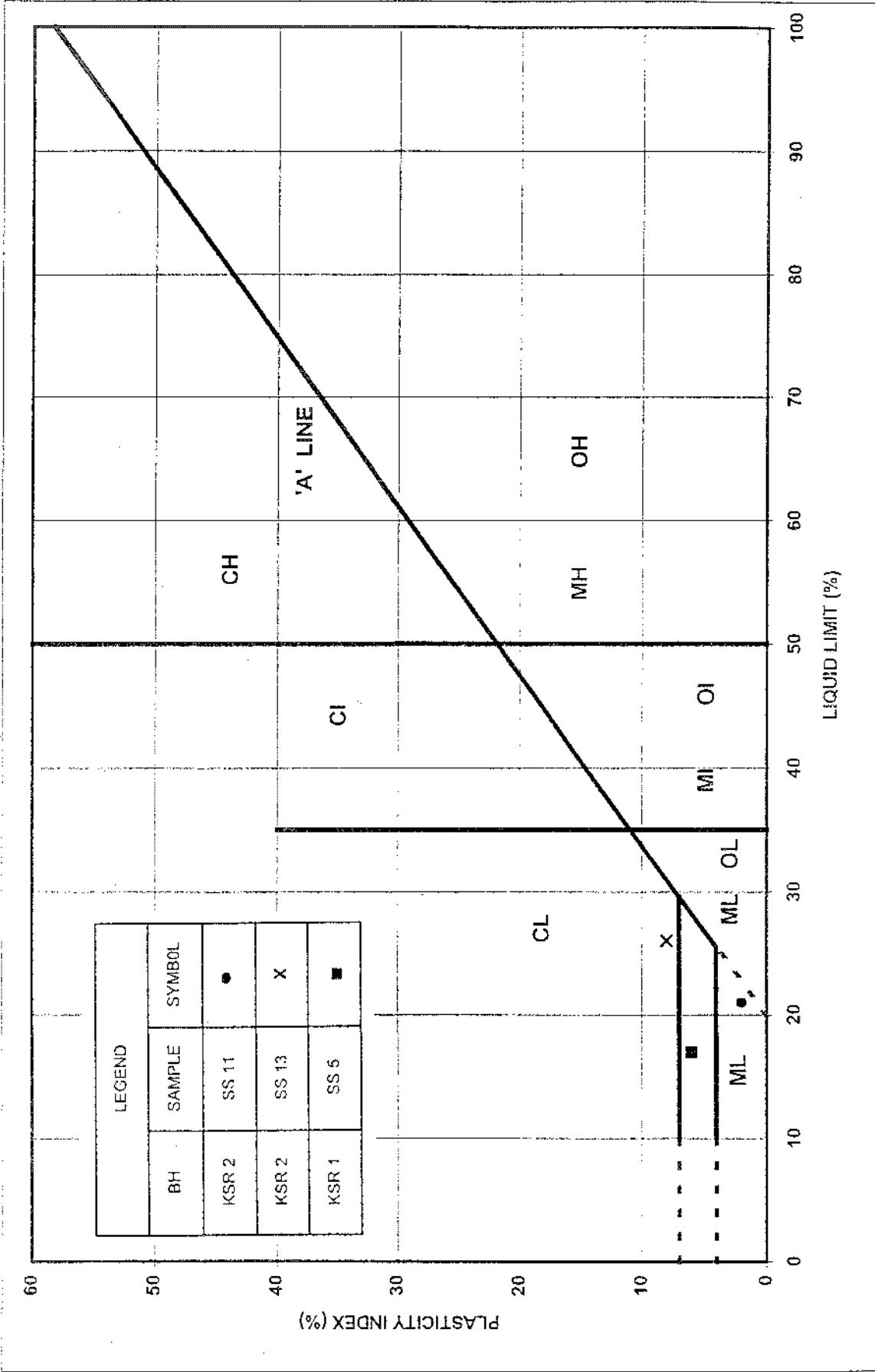
FIGURE NO. 3

UNIFIED SOIL CLASSIFICATION SYSTEM



<p>AMEC Earth & Environmental Limited 104 Crookford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6565, Fax +1 (416) 751 7592 www.amec.com</p>	GRAIN SIZE DISTRIBUTION		Client :- Ministry of Transportation Ontario	
	SILT		Project:- Highway 400 Interim Widening	
	some sand, trace clay		Location:- Vaughan, Ontario	
			Depth :- 13.7 m	Date :- 10-Sep-2002

FIGURE NO. 4



RECORD OF BOREHOLE SHEETS

AMEC EARTH AND ENVIRONMENTAL LIMITED

NOTES TO BOREHOLE LOGS

DRILLING DATA

Method:
 SolSt Augering - Solid Stem Augering
 HolSt Augering - Hollow Stem Augering
 WB - Washed Boring

LABORATORY DATA

WP - Plastic Limit (%)
 W - Water Content (%)
 WL - Liquid Limit (%)
 γ - Natural Unit Weight (kN/m³)
 UNDR STRNG or C_u - Undrained Shear Strength (kPa)
 Field Vane: St-sensitivity
 pp - Pocket Penetrometer
 UC - Unconfined Compression
 UU - Unconsolidated Undrained at Overburden Pressure
 CU - Consolidated Undrained
 CD - Consolidated Drained
 TOV - Total Organic Vapours

SAMPLES

TYPE:
 SS - Split Spoon
 AS - Auger Sample
 TW - Thinwall Open
 TP - Thinwall Piston
 WS - Washed Sample
 BS - Block Sample
 RC - Rock Core
 PH - Sample Advanced Hydraulically
 PM - Sample Advanced Manually

Standard Penetration Test, 'N'-values
 The Standard Penetration Test (SPT) 'N'-values are the number of blows required to cause a standard 51 millimetre o.d. split barrel sample to penetrate 0.3 metres into undisturbed ground in a borehole when driven by a hammer with a mass of 63.5 kilograms falling freely a distance of 0.76 metres. For penetrations of less than 0.3 metres, N-values are indicated as the number of blows for the penetration achieved (e.g. 50/25: 50 blows for 25 centimetre penetration).

Dynamic Cone Penetration Test:
 Continuous penetration of a conical steel point (51 millimetre o.d. 60° cone angle) driven by 475 J impact energy on a size drill rods. The resistance to cone penetration is measured as the number of blows for each 0.3 metres advance of the conical point into the undisturbed ground.

Soils are described by their composition and consistency or compactness.

CONSISTENCY: Cohesive soils are described on the basis of their undrained shear strength (C_u) or 'N'-values as follows:

C_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD
N (blows/0.3 metres)	0 - 2	2 - 4	4 - 8	8 - 15	15 - 30	> 30

COMPACTNESS: Cohesionless soils are described on the basis of compactness as indicated by 'N'-values as follows:

N (blows/0.3 metres)	0 - 4	4 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

Rocks are described by their composition and structural features and/or strength.

RECOVERY: Sum of all recovered rock core pieces from a coring run expressed as a percent of the total length of the coring run.

ROCK QUALITY

DESIGNATION (RQD): Sum of those intact core pieces, 100 millimetres in length expressed as a percent of the length of the coring run. Classification of a rock based on the RQD value as follows:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50 millimetres	50 - 300 millimetres	0.3 - 1.0 millimetres	1.0 - 3.0 millimetres	> 3.0 millimetres
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

RECORD OF BOREHOLE No KSR1

1 OF 1

W.O. 192-00-00 LOCATION 4860422.5N 300165.1E ORIGINATED BY NNK
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY NNK
 DATUM Geodetic DATE 14 August 2002 - 14 August 2002 CHECKED BY AD
 PROJECT Kirby Side Road Over Pass, 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	60	80	100					
250.6																		
0.0	Sand and gravel (FILL) moist, brown																	
250.0																		
0.6	Clayey Silt, some sand, (FILL) firm to stiff, damp, grey		1	SS	14		1	250										
			2	SS	6		2	249										
248.4																		
2.2	Clayey Silt, some sand, trace gravel (TILL) very stiff to hard, damp to moist, grey		3	SS	16		3	248										
			4	SS	16		4	247										
			5	SS	16		5	246										
			6	SS	25		6	245										
			7	SS	30		7	244										
			8	SS	28		8	243										
			9	SS	27		9	242										
			10	SS	25		10	241										
240.8																		
9.6	End of Borehole Groundwater in open bore on completion: 3.6m Cave on completion: 7.6m																	

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

RECORD OF BOREHOLE No KSR2

amec

1 OF 2

W.O. 192-00-00 LOCATION 4860437.7N 300216.9E ORIGINATED BY NNK
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY NNK
 DATUM Geodetic DATE 14 August 2002 - 14 August 2002 CHECKED BY AD
 PROJECT Kirby Side Road Over Pass, 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE						
252.2									20 40 60 80 100						
0.0	100mm ASPHALT							252							
251.5	Sand and Gravel (FILL), moist, brown		1	AS											
0.7	Clayey Silt, some sand, trace gravel, topsoil, (FILL) stiff, damp, grey		2	SS	11		1	251							
			3	SS	12		2	250							
250.0	Clayey Silt, some sand, trace gravel (TILL) occasional sand seams, stiff to hard, damp to moist, grey		4	SS	14		3	249						22.8	
2.2			5	SS	14		4	248						23.5	
			6	SS	22		5	247							
			7	SS	22		6	246							
			8	SS	38		7	245							
244.3	Sandy Silt, occasional sand seams compact, wet, grey		9	SS	63		8	244							
7.9			10	SS	21		9	243							
242.0	Silt, occasional sand and clay seams very dense, damp, grey		11	SS	75		10	242							
10.2			12	SS	51		11	241							
240.5	Clayey Silt, some sand, trace gravel (TILL) hard, damp, grey		13	SS	100/23		12	240						22.7	2 30 48 20
11.7							13	239							
238.9	Clayey Silt, hard damp, grey						14	238						21.0	0 1 68 31
13.3															
237.4															

Continued Next Page

+3, X3, Numbers refer to Sensitivity □ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No KSR2



2 OF 2

W.O. 192-00-00	LOCATION 4860437.7N 300216.9E	ORIGINATED BY NNK
DIST HWY 400	BOREHOLE TYPE Solid Stem Augering	COMPILED BY NNK
DATUM Geodetic	DATE 14 August 2002 - 14 August 2002	CHECKED BY AD
PROJECT Kirby Side Road Over Pass, 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									WATER CONTENT (%)
									○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE						
									20	40	60	80	100					
14.8	Silt, some Sand, trace Clay, dense to very dense wet, grey		14	SS	61		16	237									SS14: 1.2m backup inside borehole	
							17	236									0 14 (86)	
234.2			15	SS	48		18	235										
18.0	Silty Sand, dense wet, grey		16	SS	33		19	234									SS16: 3.0m backup inside borehole	
							20	233									SS17: 3.0m backup inside borehole	
			17	SS	40		21	232										
							22	231									SS18: 4.8m backup inside borehole	
233.2			18	SS	43													
22.0	End of Borehole																	
	Groundwater in open bore on completion: 4.3m																	
	Cave on completion: 14.6m																	
	Water Level in Piezometer:																	
	30 Aug /02: 0.1 m																	
	16 Sept /02: 0.1 m																	

RECORD OF BOREHOLE No KSR3



1 OF 1

W.O. 192-00-00 LOCATION 4860451.0N 300163.4E ORIGINATED BY NNK
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY IH
 DATUM Geodetic DATE 22 August 2002 - 22 August 2002 CHECKED BY AD
 PROJECT Kirby Side Road Over Pass, HWY 400 Widening, Vaughan, Ontario JOB NO. TT22652

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa							
256.4									○ UNCONFINED	+ FIELD VANE						
0.0									● QUICK TRIAXIAL	x LAB VANE						
									20	40	60	80	100	10	20	30
			1	SS	33		256									
			2	SS	41		255									
			3	SS	31		254									
			4	SS	30		253									
			5	SS	56		252									
			6	SS	35		251									
			7	SS	32		250									
			8	SS	33		249									
248.2																
8.3	End of Borehole															
	Groundwater in open bore on completion: none															

+ 3 X 3

Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No KSR4

1 OF 1

W.O. 192-00-00 LOCATION 4860403.9N 300175.0E ORIGINATED BY NNK
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY JH
 DATUM Geodetic DATE 22 August 2002 - 22 August 2002 CHECKED BY AD
 PROJECT Kirby Side Road Over Pass, HWY 400 Widening, Vaughan, Ontario JOB NO. TT22952

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										
256.8	Sandy Silt: trace rootlets, gravel (FILL) damp, dark brown to brown		1	SS	32														
255.9			2	SS	14														
0.7	Clayey Silt: some sand, trace gravel (TILL) stiff to hard, damp to moist, brown to grey		3	SS	23														
			4	SS	23														
			5	SS	33														
			6	SS	100/18														
			7	SS	4C														
			8	SS	32														
			248.4	End of Borehole															
			8.3	Groundwater in open bore on completion, none															

+ 3 x 3

Numbers refer to Sensitivity

○ 3%

STRAIN AT FAILURE

RECORD OF BOREHOLE No KSR5

1 OF 1

W.O. 192-00-00 LOCATION 4860412 5N 300216 6E ORIGINATED BY PPM
 DIST HWY 406 BOREHOLE TYPE Solid Stem Augering COMPILED BY IH
 DATUM Geodetic DATE 16 September 2002 - 16 September 2002 CHECKED BY AD
 PROJECT Kirby Side Road Over Pass, 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						
256.1									20 40 60 80 100	20 40 60 80 100					
256.0	130mm TOPSOIL		1	SS	7		1	256							
			2	SS	57			255							
			3	SS	70		2	254							
			4	SS	72		3	253						21.9	
			5	SS	70		4	252						21.4	
			6	SS	89		5	251							
			7	SS	76		6	250							
			8	SS	50		7	249							
248.0	End of Borehole						8							23.4	
8.1	Groundwater in open bore on completion: none														

APPENDIX 'A'

**Borehole Logs extracted from MTO report dated 15 December 1989
GEOCRES File No. 30M13-93**

RECORD OF BOREHOLE No 1

METRIC

W.P. 95-85-01 LOCATION Co-ords: N 4 860 219.7; E 300 191.5
 DIST 6 HWY 400 BOREHOLE TYPE H.S. Auger & Cone Test
 DATUM Geodetic DATE 89 08 27

ORIGINATED BY AL
 COMPILED BY AL
 CHECKED BY TS

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMA & GRAIN DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			20	40	60	80	100	W _p	W	W _L		
251.7	Ground Surface															
0.0	Clayey Silt Some Sand, Trace Gravel (Glacial Till)		1	SS	9											
			2	SS	14											
			3	SS	12											
	Grey		4	SS	12											
	Self V. Silty		5	SS	12											
			6	SS	15											
			7	SS	14											
			8	SS	14											
243.9			9	SS	7											
7.8	Silt Some Sand, Trace Gravel		10	SS	15											
	Grey, Compact to Very Dense		11	SS	20											
239.1			12	SS	15											
12.0	End of Borehole															

RECORD OF BOREHOLE No 2

METRIC

W.P. 95-85-01 LOCATION Co-ords: N 4 860 311.4; E 300 193.0
 DIST 6 HWY 400 BOREHOLE TYPE H.S. Auger & Cone Test
 DATUM Geodetic DATE 89 08 26
 ORIGINATED BY AL
 COMPILED BY AL
 CHECKED BY JS

OFFICE REPORT ON SOIL EXPLORATION

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 Y	REMARKS & GRAIN DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100				
251.9	Ground Surface														
0.0	Sand Trace Gravel (Fill)		1	SS	5										
250.1	Brown, Loose		2	SS	6										
1.8	Clayey Silt Some Sand, Trace Gravel (Glacial Till)		3	SS	8										
			4	SS	12										
			5	SS	13										
	Grey Firm to V. Stiff		6	SS	22										
			7	SS	16										
	Occ. Sand Seams		8	SS	12										
244.3			9	SS	24										
7.8	Silt Trace Sand, Trace Gravel Grey Compact to Dense		10	SS	24										
			11	SS	28										
			12	SS	18										
			13	SS	18										
236.2			14	SS	1										
13.7	End of Borehole														

RECORD OF BOREHOLE No 3

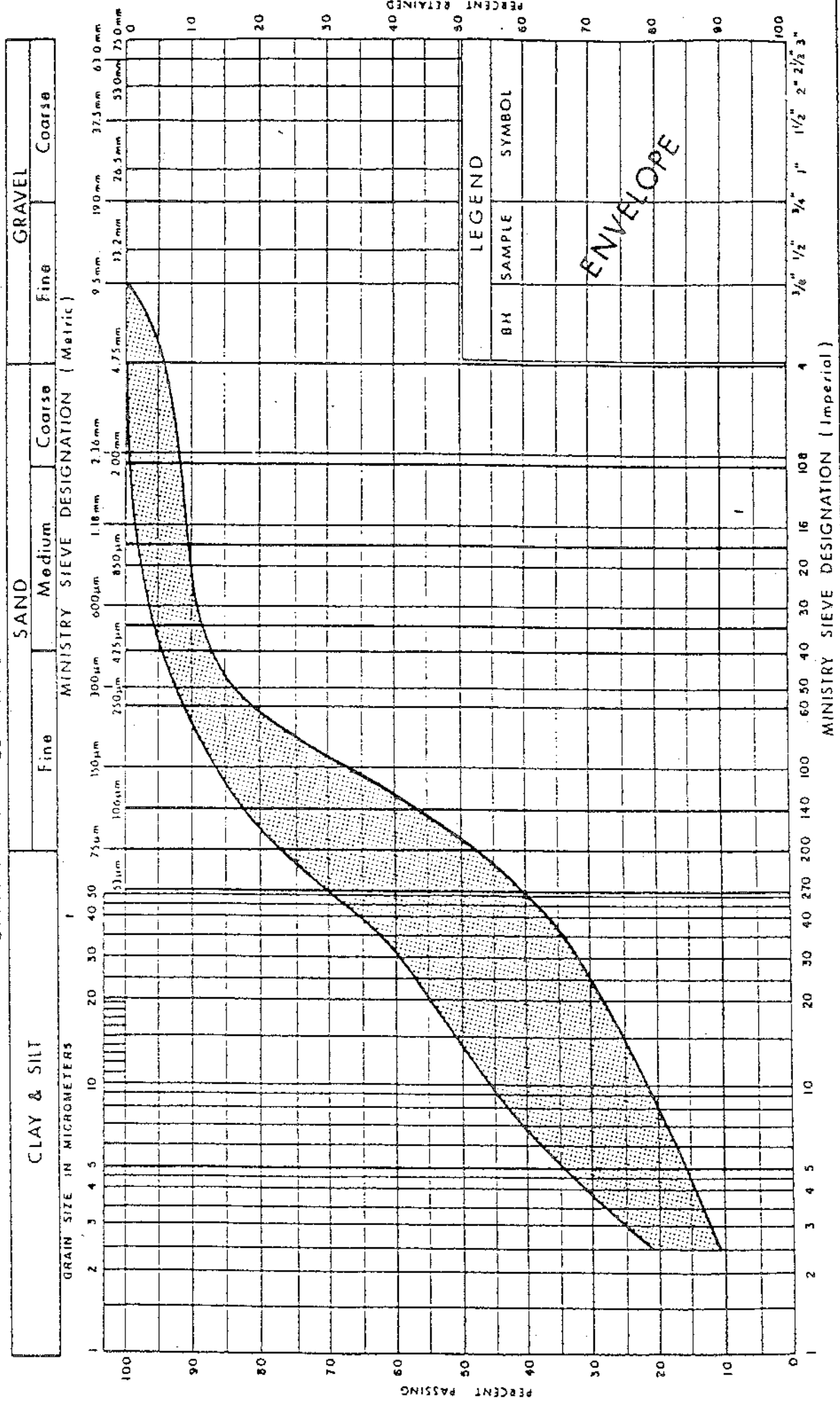
METRIC

W P 95-85-01 LOCATION Co-ords: N 4 860 236.0; E 300 191.8
 DIST 6 HWY 400 BOREHOLE TYPE W. S. Auger
 DATUM Geodetic DATE 89 08 27
 ORIGINATED BY AL
 COMPILED BY AL
 CHECKED BY TS

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE (MPa)					PLASTIC NATURAL MOISTURE LIMIT CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100	W _p	W	W _L		
257.8	Ground Surface															
0.0	Clayey Silt Some Sand, Trace Gravel (Glacial Till)		1	SS	20	256									22.5	6 28 36 10
			2	SS	50											
	Brown Grey		3	SS	20	234										
	Stiff to Hard		4	SS	16										21.9	
	Occ. Sand Seams		5	SS	17	252										0 52 38 10
			6	SS	13											
250.9	End of Borehole															
0.9																

OFFICE REPORT ON SOIL EXPLORATION

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL

Ocl 75, FF-S-21

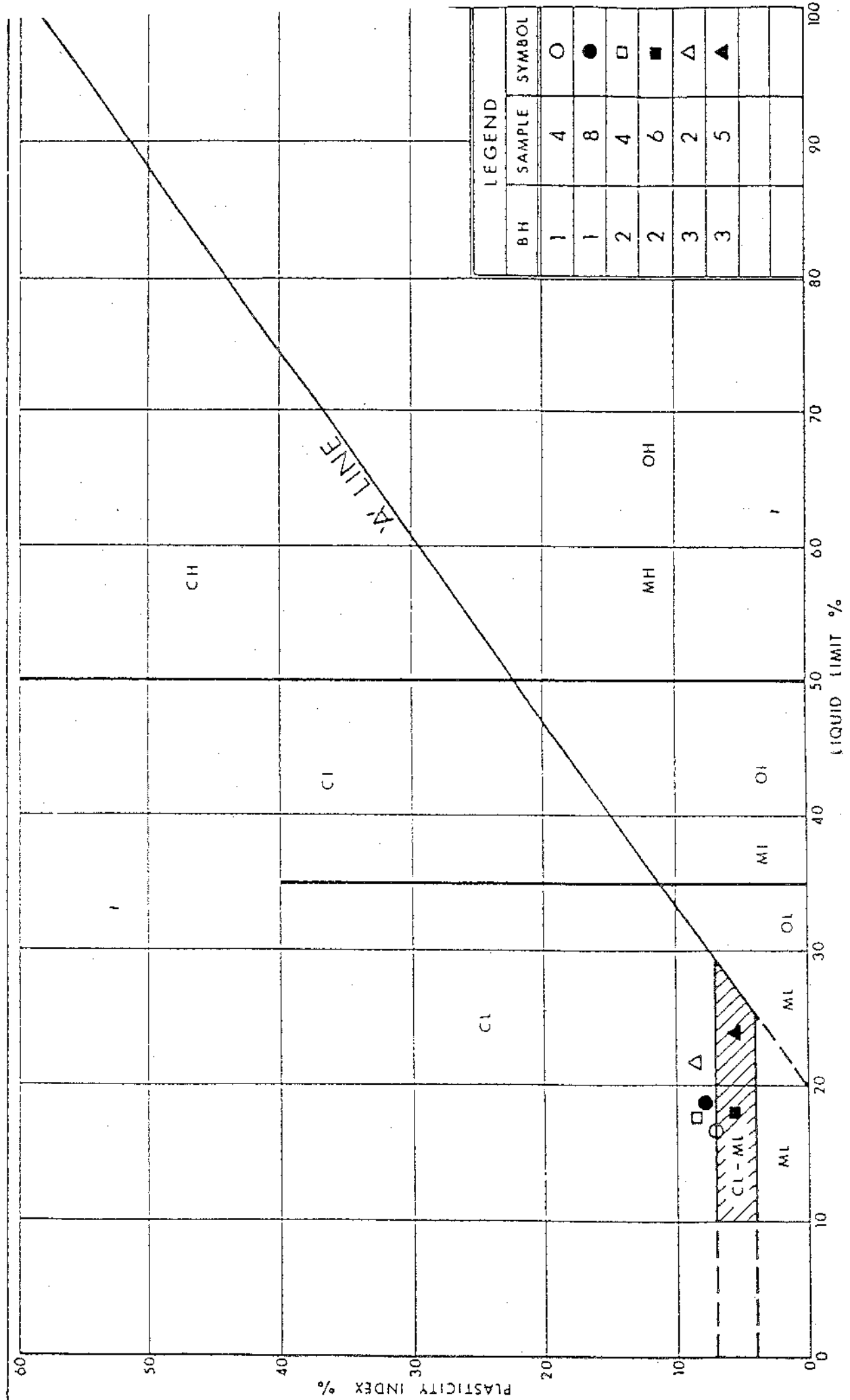


FIG No 2

WP 95-85-01

PLASTICITY CHART
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL
(Glacial Till)

APPENDIX 'B'

**Borehole Logs extracted from report prepared by
Golder Associates Limited dated June 2001
GEOCRES File No. 30M13-49**

PROJECT 001-1122F		RECORD OF BOREHOLE No 89		1 OF 3	METRIC
W.P. 222-97-00	LOCATION N 4850436 E 300167	ORIGINATED BY SB			
DIST Central HWY 400	BOREHOLE TYPE 108mm I.D. Hollow Stem Augers	COMPILED BY LCC			
DATUM Geocentric	DATE October 10 & 11 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 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						
251.0	GROUND SURFACE																	
0.0	Sand and Gravel, trace clay and organics (FILL)		1	SS	30													
250.4	Compact to dense Moist																	
0.6	Brown to grey Clayey Silt with sand, some gravel, occ. cobbles (Till)		2	SS	19													
	Stiff to hard Moist Grey		3	SS	23													
			4	SS	14													
			5	SS	19													
			6	SS	23													
			7	SS	22													
			8	SS	52													
243.7																		
7.3	Sand, trace silt Compact Wet Grey		9	SS	22													
241.9																		
9.1	Silt, trace to some Sand to Sand and silt Dense Moist Grey		10	SS	42													
240.5	Sand and Silt, trace clay (Till) Compact Grey Moist		11	SS	20													
239.1																		
11.9	Interlayered Silt, trace to some sand, trace clay Sand and Silt, trace clay and Silty Sand, trace clay Very dense Wet Grey		12	SS	101													
			13	SS	90													

ON MOT 001-1122 GPJ ON MOT GDI 12/2/01

Continued Next Page

+3, X3, Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

PROJECT 001-1122F				RECORD OF BOREHOLE No 89				2 OF 3		METRIC		
W.P. 222-97-00				LOCATION N 4850436 E 300167				ORIGINATED BY SS				
DIST Central HWY 400				BOREHOLE TYPE 108mm I.D. Hollow Stem Augers				COMPILED BY LCC				
DATUM Geodetic				DATE October 10 & 11, 2000				CHECKED BY ASP				
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT WEIGHT REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W _n W _L	WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	GR SA SI CL
— CONTINUED FROM PREVIOUS PAGE —												
	Interlayered Silt, trace to some sand, trace clay Sand and Silt, trace clay and Silty Sand, trace clay Very dense Wet Grey		14	SS	70		235					
			15	SS	55		234					
							233					
			16	SS	65		232					
							231					
			17	SS	97		230					
229.7												
21.3	Silty Clay trace sand, trace gravel to silt, trace clay, sand and gravel (Till) Hard/dense Moist Grey		18	SS	109		229					
	Hard augering between 22.5m and 24m depth		19	SS	34		228					
228.9							227					
24.1	Silt, trace sand, occasional silty clay seams Very loose Wet Grey		20	SS	WH		226					
							225					
			21	SS	2		224					
223.6							223					
27.4	Sand and Silt, trace clay Loose to compact Wet Grey		22	SS	4		222					
			23	SS	27							
221.4												
29.8	END OF BOREHOLE											

ON MOT 001-1122 GPJ ON MOT GDT 12/2001

Continued Next Page

+ 3 X 3 Numbers refer to
Sensitivity

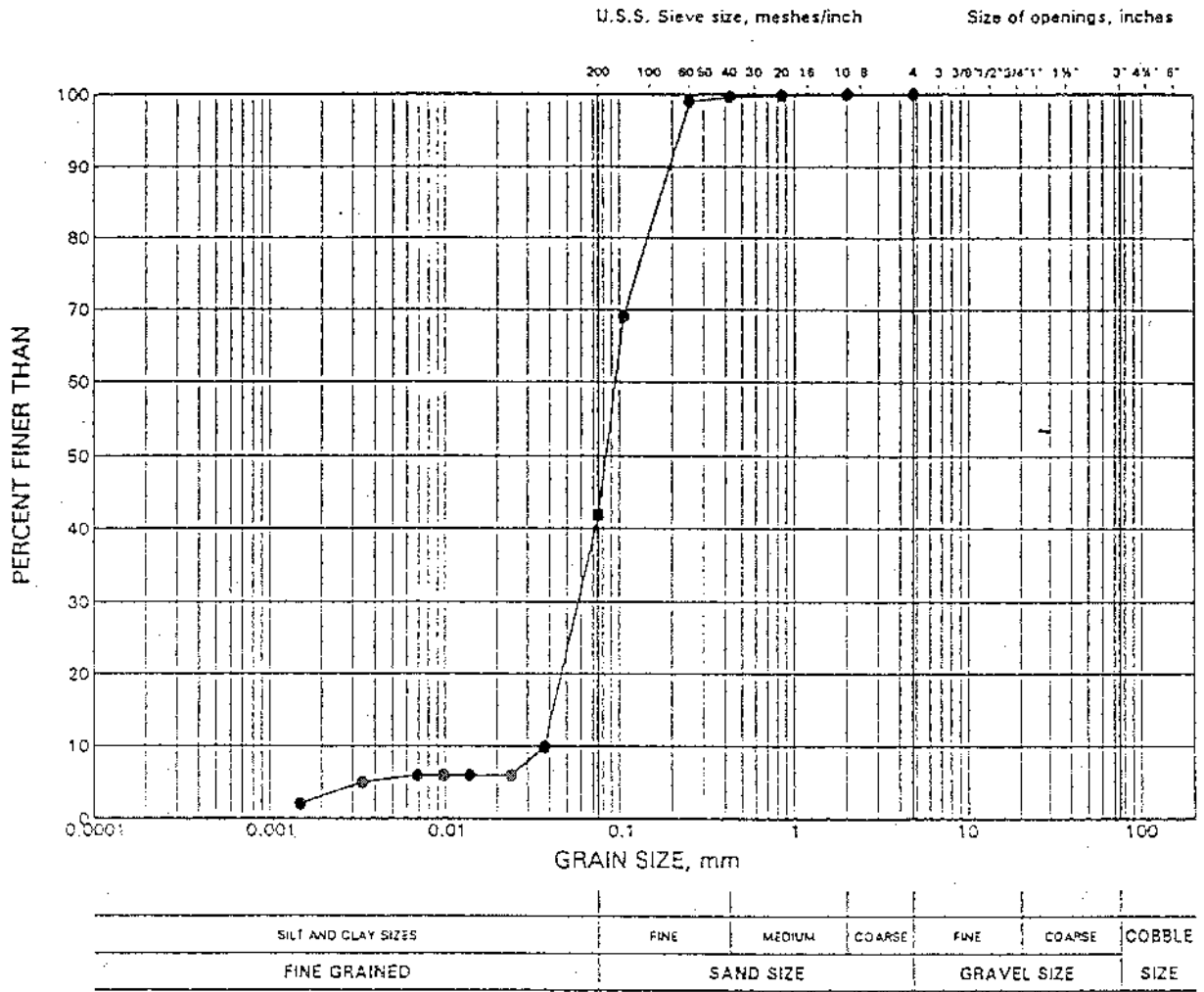
○ 3% STRAIN AT FAILURE

PROJECT 001-1122F										RECORD OF BOREHOLE No 89										3 OF 3										METRIC									
W.P. 222-97-00										LOCATION N 4860436 E 300167										ORIGINATED BY SB																			
DIST Central HWY 400										BOREHOLE TYPE 108mm I.D. Hollow Stem Augers										COMPILED BY LCC																			
DATUM Geodetic										DATE October 10 & 11, 2000										CHECKED BY ASP																			
SOIL PROFILE					SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT					UNIT WEIGHT					REMARKS & GRAIN SIZE DISTRIBUTION (%)														
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					W _p — W — W _L					WATER CONTENT (%)					GR SA SI CL																
— CONTINUED FROM PREVIOUS PAGE —									20	40	60	80	100	O UNCONFINED + FIELD VANE					10 20 30																				
								20	40	60	80	100	● QUICK TRIAXIAL X REMOULDED																										
217.8	Driving resistance greater than 100 blows/0.3m over final 0.6m.						220																																
33.2							219																																
							218																																
	END OF DYNAMIC CONE PENETRATION TEST																																						
	Note: The groundwater level was measured in the piezometer at 0.5m above ground surface (Elevation 251.5m) on January 19, 2001.																																						

ON MOT 001-1122 GPJ ON MOT GDT 12/2/01

GRAIN SIZE DISTRIBUTION SAND AND SILT

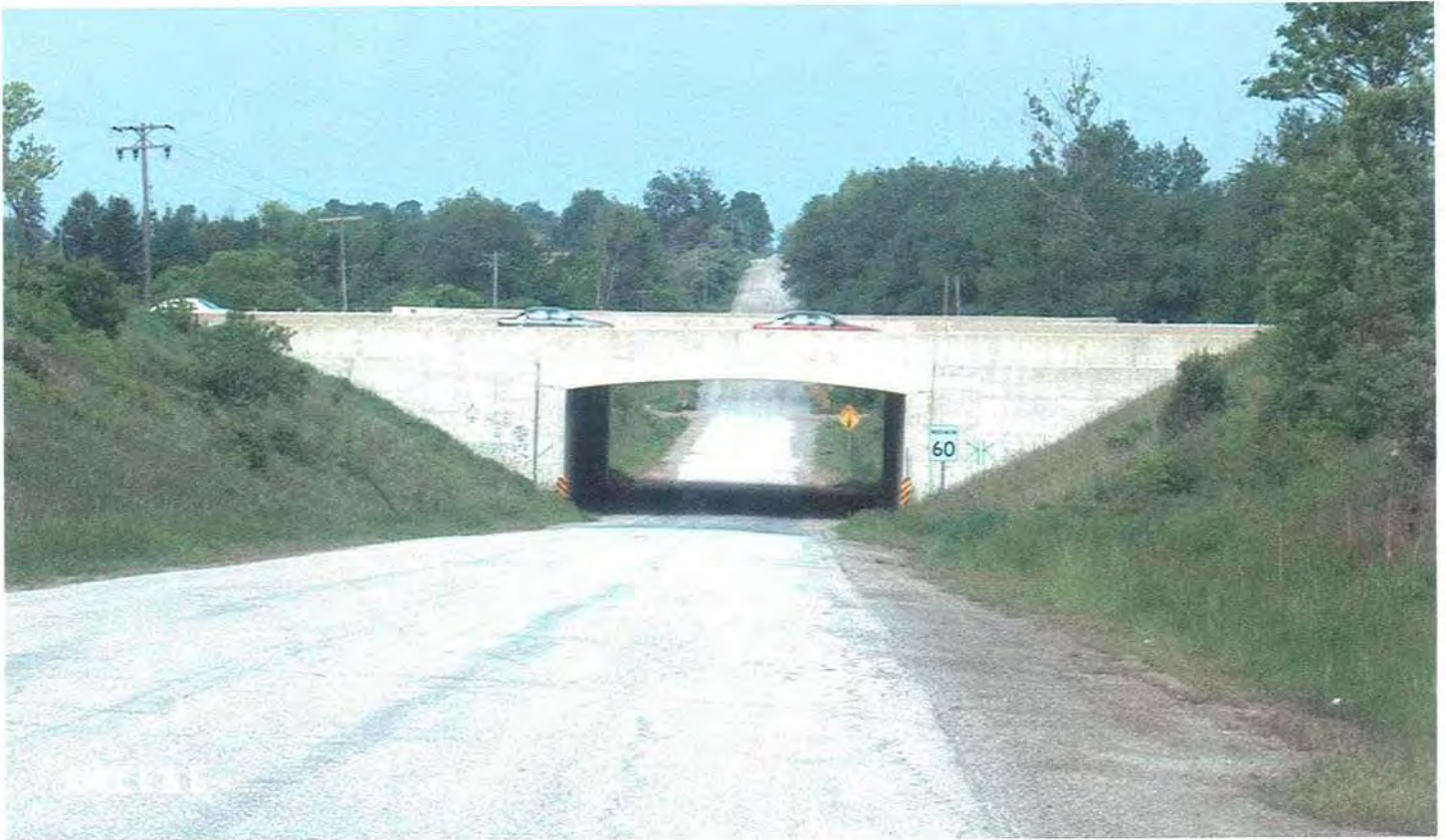
FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	89	16	18.7

APPENDIX 'C'
Site Photographs



Photograph No. 1: Looking west at Kirby Sideroad Overpass Structure at Highway 400.



Photograph No. 2: Looking east at Kirby Sideroad Overpass at Highway 400.