



FINAL  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
PROPOSED VAUGHAN-KING TOWNSHIP ROAD UNDERPASS  
HIGHWAY 400 INTERIM WIDENING  
VAUGHAN, ONTARIO  
W.P. 192-00-00, CENTRAL REGION

Submitted to:

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## TABLE OF CONTENTS

	Page
1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION AND PHYSIOGRAPHY .....	1
2.1 Site Description .....	1
2.2 Physiography.....	2
3.0 INVESTIGATION PROCEDURES.....	2
4.0 SUB-SURFACE CONDITIONS .....	3
4.1 Fill and Topsoil .....	4
4.2 Upper Clayey Silt Glacial Till.....	4
4.3 Silts and Sands.....	5
4.4 Lower Clayey Silt Glacial Till.....	6
4.5 Groundwater Conditions .....	7
5.0 DISCUSSION AND RECOMMENDATIONS.....	8
5.1 Bridge Foundations.....	8
5.1.1 Spread Footings on Native Subgrade .....	8
5.1.2 Pile Foundations .....	10
5.1.2.1 Resistance to Axial Loads .....	11
5.1.2.2 Resistance to Lateral Loads .....	12
5.2 Lateral Earth Pressures .....	15
5.3 Approach Embankments.....	16
5.4 Construction Comments .....	18
5.5 Frost Protection .....	19
5.6 Construction Inspection .....	19
6.0 CLOSURE.....	19

## DRAWINGS

DRAWING NO. 1	Borehole Locations
DRAWING NOS. 2 to 3	Soil Strata Cross-sections

## FIGURES

FIGURE NO. 1	Atterberg Limits – Clayey Silt Glacial Till
FIGURE NOS. 2 TO 5	Grain Size Distribution – Clayey Silt Glacial Till
FIGURE NO. 6 TO 8	Grain Size Distribution – Silt
FIGURE NO. 9	Grain Size Distribution – Sand
FIGURE NO. 10	Grain Size Distribution – Lower Clayey Silt Glacial Till

## RECORD OF BOREHOLE SHEETS

### NOTES OF BOREHOLE LOGS

RECORD OF BOREHOLE SHEETS ..... Borehole Numbers: KV1 to KV8

## 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 of the existing Vaughan-King Township Road Underpass on the border of the Township of King and 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 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 and approaches 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

- Department of Highways Ontario. *Foundation Investigation Report, Subsurface Exploration at Highway 400 and Gravel Road, King City*. dated 1959 - GEOCRE File No. 30M13-2.

## 2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

### 2.1 SITE DESCRIPTION

The existing Vaughan-King Road underpass structure is located about 1.5 km south of the King Road interchange on the border of the Township of King and the City of Vaughan, Regional Municipality of York, Ontario.

The original ground at the underpass structure appears to be at about Elevation 276 to 277m. The existing Highway 400 grade is at about Elevation 278.5m, while Vaughan-King Township Road grade is at Elevation 285m at the structure location. Vaughan-King Township Road has been constructed in fill with approach embankments about 7m high.

The existing underpass structure was constructed in the early 1960s under Contract 60-276. It

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is a single span structure supported on spread footings at the abutments.

## **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 tills, 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 150 to 200 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 at greater depth.

## **3.0 INVESTIGATION PROCEDURES**

The fieldwork for the current investigation was carried out on 9 January 2003, 8 and 22 February 2003, and 15 March 2003, and consisted of drilling and sampling eight boreholes (Boreholes KV1 to KV8, inclusive) to depths of 9.6 m to 22.0 m below the existing ground surface. Boreholes KV7 and KV8 were advanced from the Highway 400 grade, while Boreholes KV1, KV2, K4 and K6 were advanced from Vaughan-King Township Road, and Boreholes KV 3 and KV5 were advanced beyond the toe of the existing road embankment.

Also referenced in this report in Appendix A, are boreholes (Boreholes 1 to 4) advanced by the Department of Highways Ontario (DHO) in 1959 for a foundation investigation for the existing bridge (GEOCRES No. 30M13-2, referenced in Section 1.0).

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 hollow and solid stem continuous flight augers with track and truck-mounted power auger drill rigs (CME 75) owned and operated by Atcost Soil Drilling Inc. and Walker Drilling Inc., 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

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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 to 10, inclusive.

Groundwater conditions in the current investigation open boreholes were observed throughout and immediately after the drilling operations. Standpipe piezometers were installed in Boreholes KV3 and KV5 beyond the toe of the Vaughan-King Township Road embankment, 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 AMEC engineering staff. These elevations are referenced to the geodetic datum. The locations and co-ordinates of the boreholes are shown on Drawing No. 1, 2 and 3; the co-ordinates and elevations are indicated on the Record of Borehole Sheets.

The borehole northings and eastings of the 1959 investigation by DHO were estimated based on existing features from the plans obtained from MTO.

#### **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 Fill and Topsoil**

Boreholes KV7 and KV8 were advanced from the Highway 400 grade, while Boreholes KV1, KV2, K4 and K6 were advanced from Vaughan-King Township Road, and Boreholes KV 3 and KV5 were advanced beyond the toe of the existing road embankment. Boreholes advanced on Highway 400 encountered 250 to 280 mm of asphaltic concrete underlain by a pavement fill to depths of 0.9 m below existing grade. The fill consisted of a damp, brown sand to gravelly sand, with varying amounts of gravel, and trace silt.

Boreholes KV2, KV4 and KV6 advanced on Vaughan-King Township Road encountered 110 mm to 150 mm of asphaltic concrete. Underlying the asphaltic concrete in these boreholes, and at the surface of Borehole KV1, the boreholes encountered a pavement fill to depths of about 0.5 to 2.8m below existing grade. The fill consisted of a damp, brown sand to gravelly sand, with varying amounts of gravel, and trace silt.

Boreholes KV3 and KV5 were advanced beyond the toe of the existing road embankment and encountered about 330 mm to 450 mm of topsoil.

Underlying the pavement fill, Boreholes KV1, KV2, KV4, KV6, KV7 and KV8 encountered a brown clayey silt fill to depths of 2.9 m to 3.1m (Elevation 274.6m to 275.4m) in Boreholes KV7 and KV8 on Highway 400, and to depths of 5.5m to 7.8 m (Elevation 276.7m to 279.0m) below Vaughan-King Township Road in Boreholes KV1, KV2, KV4 and KV6. Underlying the topsoil in Boreholes KV3 and KV5, a brown to grey silty clay to clayey silt fill deposit was encountered to depths of about 2.1m to 2.4m (Elevation 275.0m to 275.1m). This cohesive fill contained varying amounts of sand, trace gravel, organics and rootlets. Excessive organics were encountered in Boreholes KV3 and KV5. Interbedded layers of sand/silty sand or sand and gravel were encountered within the fill in Boreholes KV2, KV4 and KV8. Measured 'N'-values range from 6 to 20 blows per 0.3m indicating a firm to very stiff consistency, but generally stiff. Measured moisture contents within the cohesive fill range from 11 to 20%.

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

#### **4.3 Upper Clayey Silt Glacial Till**

Below the fill deposits, all boreholes encountered a brown to grey clayey silt glacial till deposit. Boreholes KV1, KV2, KV4, KV6 and KV8 were terminated within the cohesive till at depths of 9.6m to 12.7m (Elevations 268.7m to 274.4m). The remaining boreholes encountered the base of the till at depths of 11.6 to 13.1 m below King-Vaughan Township Road (Elevations 264.3 to 265.6m) or 13.3m below Highway 400 (Elevation 264.4m ). The glacial till is a heterogeneous .../...

mixture of silt and clay with trace to some sand and gravel. Measured 'N'-values range from 9 to greater than 50 blows per 0.3m, indicating a stiff to hard consistency, but generally hard.

Boreholes 1 to 4 advanced by DHO in 1959 encountered the clayey silt till to a depth of 11.3m to 13.7m (or Elevation 262.9m to 265.3m) below the existing ground surface in 1959. Measured 'N'-values ranged from 13 to greater than 50 blows per 0.3m indicating a stiff to hard consistency, but generally very stiff to hard consistency below a depth of about 2m.

Laboratory tests carried out on the clayey silt till are summarized on the various Record of Boreholes, in Appendix A and in Figure Nos. 1 to 5. The laboratory results are summarized below.

	<u>AMEC 2003 Investigation</u>	<u>DHO 1959 Investigation</u>
Natural Moisture Content	11 to 23%	8 to 27%
Liquid Limit	32 to 36%	23 to 30%
Plastic Limit	16 to 17%	15 to 18%
Unit Weight (kN/m <sup>3</sup> )	20.4 to 21.3 (4 samples)	21.0(average of 18 samples)

#### Grain Size Distribution (4 samples)

Gravel	0 to 5%
Sand	12 to 20%
Silt	45 to 54%
Clay	26 to 38%

The above results are indicative of an overconsolidated low plasticity clay. The grain size distribution curves from the current investigation are presented on Figure Nos. 1 to 4 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 Silts and Sands

Below the clayey silt till in Boreholes KV3, KV5 and KV7 a grey interlayered silt and sand deposit was encountered to depths of 17.7m to 20.4m (Elevation 256.8m to 259.7m). The deposits contain trace to some clay. Measured 'N'-values ranged from 27 to greater than 50 blows per 0.3m indicating a compact dense to very dense relative density, but generally dense. Measured moisture contents ranged from 11 to 22%.

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Grain size analyses were carried out on two samples of the silt in the current investigation. The results are presented on Figure Nos. 6, 7, 8 and 9. The results are summarized below:

#### Grain Size Distribution

##### Silt (3 samples)

Gravel	0 to 1%
Sand	0 to 18%
Silt	67 to 95%
Clay	5 to 14%

##### Sand (1 sample)

Gravel	0%
Sand	75%
Silt	21%
Clay	4%

Below the clayey silt till in Boreholes 1 to 4 advanced in 1959 by DHO, all boreholes encountered a silty sand to sand deposit. Boreholes 2 and 4 were terminated within the cohesionless deposit at a depth of about 18.7m, or Elevation 257.0m to 257.9m. Boreholes 1 and 3 encountered the base of the stratum at a depth of about 16.8m to 17.7m (or Elevations 258.9 to 260.3m). Measured 'N'-values within this deposit ranged from 44 to greater than 100 blows per 0.3m indicating a dense to generally very dense relative density. Measured moisture contents ranged from 9 to 21%, but generally above 14%.

#### 4.4 Lower Clayey Silt Glacial Till

Underlying the lower silts and clayey silts, a lower grey clayey silt glacial till deposit was encountered in Boreholes KV3, KV5 and KV7. This grey lower till was encountered at depths of 17.7m to 20.4m (Elevations 256.8m to 259.7 m). The boreholes were terminated within the till at a depth of 20.3m to 22.6m (Elevation 255.1m to 257.1m). The glacial till is a heterogeneous mixture of silt and clay with trace to some sand and gravel. Measured 'N'-values range from 92 to greater than 100 blows per 0.3m were obtained indicating a hard consistency. Measured natural moisture contents range from 11 to 15%.

A grain size analysis was carried out on a sample of the lower clayey silt till and the results are summarized below and in Figure No. 10.

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#### Grain Size Distribution (1 sample)

Gravel	5%
Sand	27%
Silt	45%
Clay	23%

Boreholes 1 and 3 advanced by DHO in 1959 encountered the lower clayey silt till at a depth of about 16.8m to 17.7m (or Elevations 258.9 to 260.3m). The two boreholes were terminated within the till at a depth of 18.7m to 18.8m (or Elevations 257.9 to 258.3m). Measured 'N'-values range of 115 and 140 blows per 0.3m were obtained indicating a hard consistency. Measured natural moisture contents of 9 and 10% were obtained.

#### **4.5 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 KV3 and KV5 within the sands and silts.

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 site lies within the upper clayey silt till at about Elevation 271 to 272m, or about 6 to 7m below Highway 400 grade.

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 Bridge Site Plan for the Crossing at Vaughan-King Township Road and Highway 400 (Drawing No. 2 prepared by McCormick Rankin Corporation dated October 2002) the existing Vaughan-King Township Road Underpass consists of a single span bridge, 10m wide with a span of about 37m. The proposed design consists of a two span structure, about 19m wide, with a span of about 68 m. The proposed approaches will be raised by 1.5m to 3.0m.

In general, the subsoils along the proposed bridge consists of a clayey fill embankment underlain by a very stiff to hard upper clayey silt till, dense to very dense sands and silts and a lower hard clayey silt till. The groundwater table lies about 6 to 7m below Highway 400 grade.

### **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 on Native Subgrade**

Based on the data obtained from the boreholes, the proposed structure can be supported on spread footing foundations placed on the undisturbed very stiff to hard clayey silt till.

All the fill, organic and otherwise unsuitable soils should be removed to the surface of the inorganic stratum. The excavation should then be extended within a sufficient horizontal distance (ie. 2 m) beyond the proposed footing perimeter, to the surface of the competent soil.

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
Borehole KV2 East Abutment North Side	284.5	276.5	8.0	450	800	Hard Clayey Silt Till
Borehole KV3 East Abutment South Side	277.4	274.4	3.0	450	800	Hard Clayey Silt Till
Borehole KV8 Central Pier North Side	278.3	275.0	3.3	300	500	Hard Clayey Silt Till
Borehole KV7 Central Pier South Side	277.7	273.0	4.7	300	500	Hard Clayey Silt Till
Borehole KV4 West Abutment North Side	284.5	276.5	8.0	200	300	Very Stiff Clayey Silt Till
		274.5	10.0	450	800	Hard Clayey Silt Till
Borehole KV5 West Abutment Centre	277.2	274.2	3.0	300	500	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.

The following notes are inferred from the above table:

a) East Abutment

Spread footings can be founded on the hard clayey silt till at a depth of about 2 to 3 m (Elevation 274.4m to 276.5m) below Highway 400 grade. There should be sufficient space for an excavation of this size. However, dewatering may be required due to perched water.

b) Central Pier

Spread footings can be founded on the hard clayey silt till at a depth of about 3.3 to 4.7 m

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(Elevation 273.0m to 275.0m) below Highway 400 grade. Due to restricted space adjacent to the traveling lanes, shoring may be required. Temporary shoring system may include the use of shotcrete and mesh strengthened by soil anchors. Dewatering may be required due to perched water. Steel sheetpile shoring may not be feasible due to the possible hard driving through the hard clayey silt till layer ( $N > 50$ ).

#### c) West Abutment

Spread footings can be founded on the very stiff to hard clayey silt till at a depth of about 1.5m to 3.0m (Elevation 274.2m to 276.5m) below Highway 400 grade. There should be sufficient space for an excavation of this size. However, dewatering may be required due to perched water.

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.

For frost protection, the footings should have a permanent earth cover of at least 1.2m.

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.

#### 5.1.2 Pile Foundations

For a new bridge structure, consideration may 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 hard lower clayey silt glacial till at about Elevation 257m to 259m  $\pm$ .

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

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Number: 3301.00) would be suitable for use. Due to the hard upper clayey silt till and very dense sands, and the possible presence of cobbles or boulders within the till, difficult driving conditions will occur unless each hole is pre-augered.

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	
East Abutment	KV3	259 ±	18.5 ±	Hard Clayey Silt Till
Central Pier	KV7	256.5 ±	21 ±	Hard Clayey Silt Till
West Abutment	KV5	256.5 ±	20 ±	Hard Clayey Silt Till

The above estimated pile tip elevations are based on the assumption that the piles would penetrate the upper hard clayey till with N values over 50 blows per 0.3m and the compact to very dense upper sands and into the hard lower clayey silt till. The above estimated pile depths are also based on the elevations of the existing ground surface during the time of investigation.

#### 5.1.2.1 Resistance to Axial Loads

For HP 310 x 110 steel H-piles driven to Elevation 256.5m to 259m± within the hard cohesive 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 = 1,600 kN, with an applied resistance factor of 0.5.
- Geotechnical Resistance at Serviceability Limit States = 1,100 kN.

If Hiley Formula is used, the estimated ultimate resistance of the piles driven to about the elevations quoted in Table 2, is approximately 3,200 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). All pile locations will likely require 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.

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 3 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.

#### **5.1.2.2 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 18.5 to 21 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	=	160 kN
Horizontal Resistance at Serviceability Limit States	=	80 kN

If integral abutments are to be used, 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 <sup>3</sup> )	ESTIMATED ANGLE OF INTERNAL FRICTION $\phi$ (DEGREES)	ESTIMATED UNDRAINED SHEAR STENGTH (kPa)	ESTIMATED $\gamma_h$ VALUE (MN/m <sup>3</sup> )
<b>East Abutment</b>							
KV3	2 to 3	275 to 274	Very Stiff Clayey Silt till	21	--	100	--
	3 to 13	274 to 264	Hard clayey silt till	22	--	200	--
	13 to 18	264 to 259	Dense to Very Dense Silt	21	35	--	9
<b>Central Pier</b>							
KV7	3 to 4	275 to 274	Stiff Clayey Silt till	21	--	100	--
	4 to 7	274 to 271	Hard Clayey Silt till	22	--	200	--
	7 to 13	271 to 265	Very Stiff to Hard Clayey Silt till	21	--	150	--
	13 to 21	265 to 257	Very dense Sand	22	35	--	9
<b>West Abutment</b>							
KV5	2 to 3	275 to 274	Stiff Clayey Silt till	20	--	75	--
	3 to 12	274 to 265	Very Stiff to Hard Clayey Silt Till	21	--	150	--
	12 to 15	265 to 262	Very dense sand	22	35	--	9
	15 to 19	262 to 258	Compact to Dense sand	21	34	--	7
	19 to 20	258 to 257	Very Dense Sand/Silt	22	35	--	9

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 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<sup>3</sup>
- Coefficient of Lateral Earth Pressures:  
 $K_a = 0.27$  (active condition)  
 $K_o = 0.43$  (at-rest condition)

#### Compacted Granular 'B'

- Unit Weight =  $21 \text{ kN/m}^3$
- 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.

### **5.3 Approach Embankments**

The highway embankment within this investigation area is about 6 to 7m in height with side slope inclinations of about 2H:1V. It is proposed to increase the embankment height to a maximum of 9 m and widen the existing embankment to the south by about 10 m.

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. 202.030.

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.

For earth fill embankment, assuming properly compacted and acceptable inorganic earth fill material, 2 horizontal to 1 vertical (2H:1V) side slopes can be used throughout. Proper erosion .../...

control measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (OPSS 572). For embankment slopes higher than 5 m, erosion control blanket is recommended.

The fill materials used for construction of the earth fill embankment, or for the purposes of backfilling, should consist of approved, clean earth fill (preferably clayey fill to be consistent with the existing embankment material). The majority of the fill will have to be imported for this purpose. 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 and, although stripping depths will vary, the design depth used should be 150 mm. 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. If clayey fill is unavailable, the embankment fill can comprise acceptable earth borrow such as select subgrade material (SSM) as per OPSS1010. This fill will be installed to the bottom of the road sub-base.

For whichever type of embankment fill is selected, the granular base and sub-base thickness should match those within the existing roadway. Any transition treatment for fill between two different materials (earth fill and granular materials) should be carried out in accordance with appropriate MTO standard.

Based on the results of investigation and provided the embankment widening is constructed as detailed above and as per the applicable MTO standards, the settlement of the embankment should not exceed 15 mm. The expected settlements are anticipated to occur along the new slopes and shoulders and the grading can be corrected, if necessary, by the placement of additional granular materials.

Based on the borehole results, provided that all organics, weak and otherwise unsuitable materials are removed before placing the embankment widening fill, no instability problems are anticipated for this section.

.../...

Site development and construction activities should be conducted in a manner which does not result in surface erosion on any slopes. 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).

#### **5.4 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 stiff to very stiff clayey silt tills. Open cut excavations can be expected to stand temporarily at 1H to 1V side slopes, however the existing embankment fill which is about 6 to 7 m in height should be cut back so that the toe of the embankment is at a sufficient distance (i.e. 2m) beyond the top of the proposed excavation. 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.

## 5.5 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.

## 5.6 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

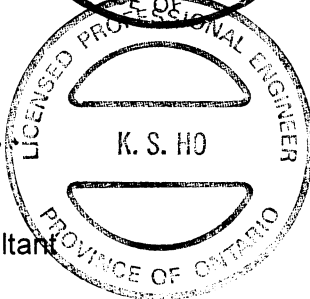
We recommend that once the details of the structures are finalized, our recommendations should be reviewed for their specific applicability.

Sincerely,

  
Andrew Drevininkas, P. Eng.  
Assistant Manager  
Geotechnical Services



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



## **DRAWINGS**

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

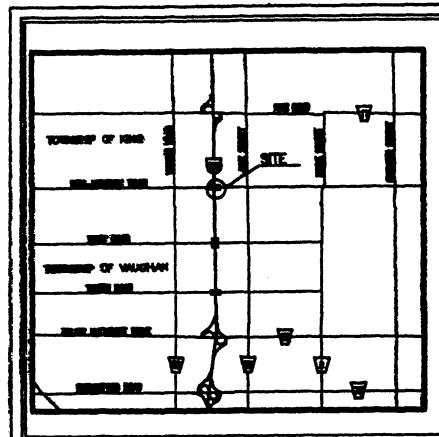
W.P. No. 192-00-00



BOREHOLE LOCATIONS  
CROSSING AT KING - VAUGHAN ROAD

SHEET

AMEC Earth & Environmental Limited



KEY PLAN  
N.T.S.

#### LEGEND

- Borehole (AMEC 2003)
- Borehole (DHO 1959)
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60' Cone, 475 J/blow)
- WL at time of investigation -
- WL in Piezometer
- Piezometer
- └ End of Borehole

No	ELEVATION	CO-ORDINATES NORTHING	EASTING
BH 1	277.1	4 862 503	299 555
BH 2	278.5	4 862 485	299 858
BH 3	278.6	4 862 474	299 820
BH 4	275.7	4 862 492	299 817
BH KV1	284.0	4 862 507	299 888
BH KV2	284.5	4 862 501	299 868
BH KV3	277.4	4 862 478	299 880
BH KV4	284.5	4 862 485	299 808
BH KV5	277.2	4 862 454	299 803
BH KV6	283.6	4 862 472	299 787
BH KV7	277.7	4 862 479	299 836
BH KV8	278.3	4 862 499	299 833

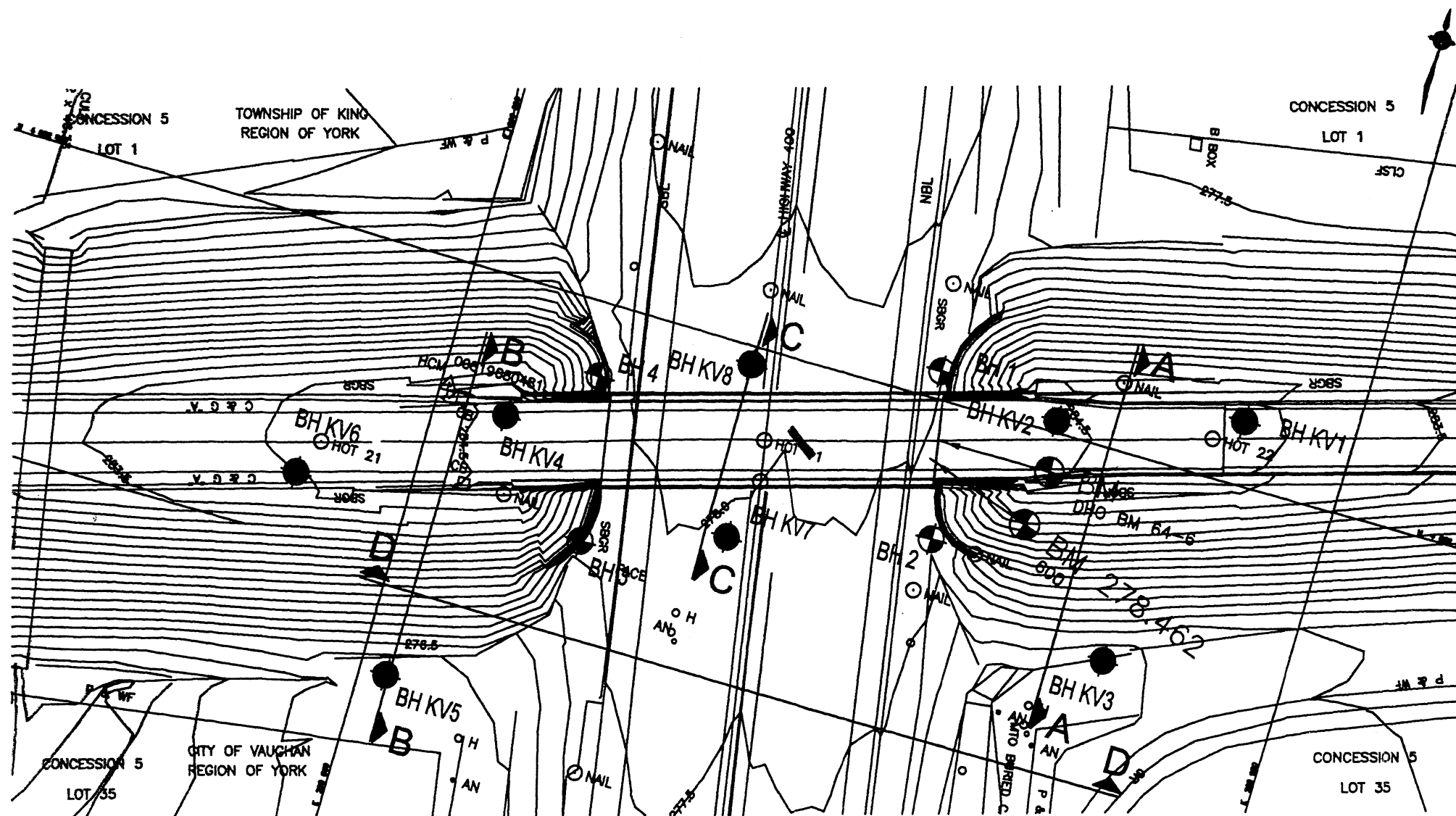
-NOTE-

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

NOTED: The complete foundation investigation and design report for this project and other related documents may be obtained at the Engineering Materials Office, Department of Infrastructure Services, City of Vaughan, 2000 Vaughan Road, Unit 100, Vaughan, Ontario L4N 4C1.

DATE	BY	DESCRIPTION

VAUGHAN AND KING ROAD UNDERPASS	REF. 6-TORONTO
SUBMIT NO.	CHECKED KSH DATE APRIL 2003 SITE
DRAWN NO.	CHECKED APPROVED DWG 1

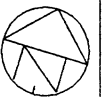


BOREHOLE LOCATION PLAN



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

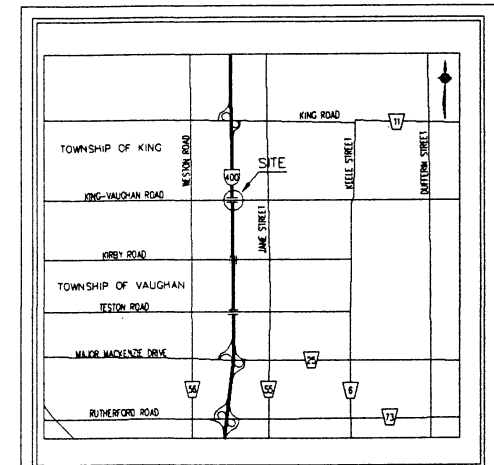
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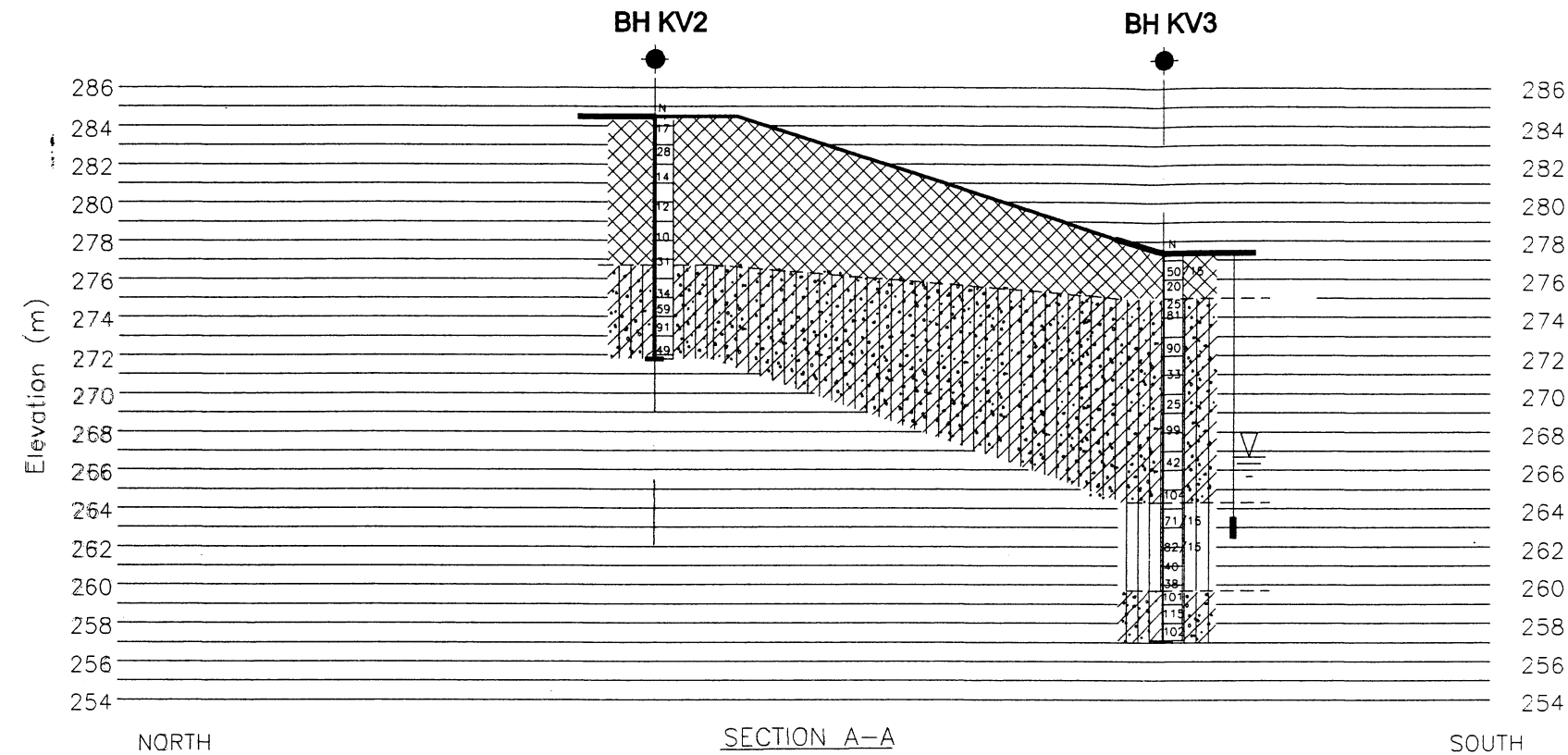
SECTION A-A & SECTION B-B  
CROSSING AT KING-VAUGHAN ROAD

SHEET

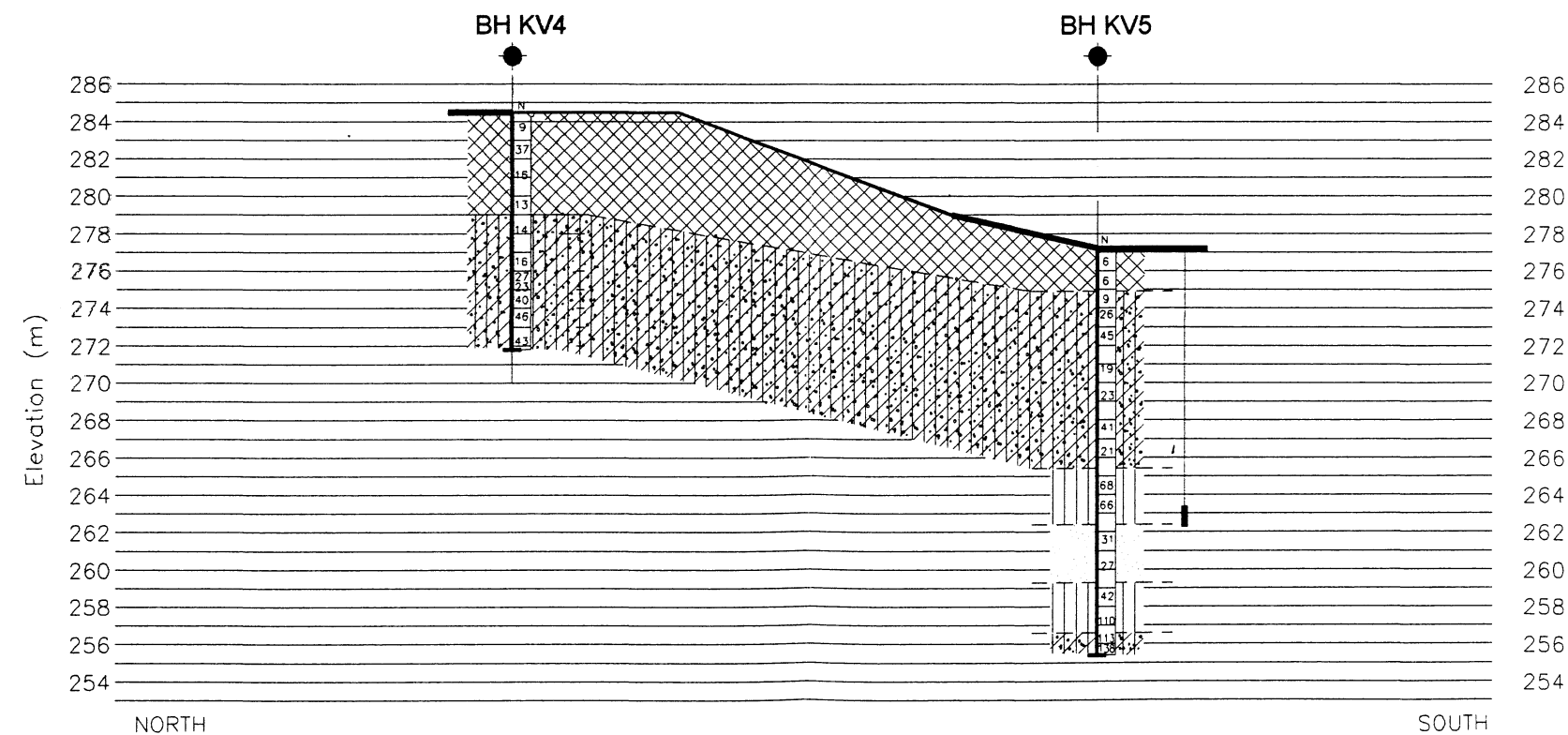
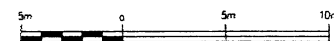
AMEC Earth & Environmental Limited



KEY PLAN  
N.T.S.



SECTION A-A



SECTION B-B



**LEGEND:**

- FILL Sand to Clayey Silt compact to dense/stiff to very stiff
- SILT very dense
- SAND to SILTY SAND dense to very dense
- SILTY CLAY to CLAYEY SILT TILL stiff to hard

LEGEND			
	Borehole (AMEC 2003)		
	Borehole (DHO 1959)		
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60° Cone, 475 J/blow)		
	WL at time of investigation		
	WL in Piezometer		
	Piezometer		
	End of Borehole		

No	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BH 1	277.1	4 862 503	299 555
BH 2	276.5	4 862 485	299 858
BH 3	276.6	4 862 474	299 820
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BH KV5	277.2	4 862 454	299 803
BH KV6	283.8	4 862 472	299 787
BH KV7	277.7	4 862 479	299 836
BH KV8	278.3	4 862 499	299 833

—NOTE—  
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes 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.Cand.

REV	DATE	BY	DESCRIPTION
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VAUGHAN - KING ROAD UNDERPASS	DIST 6-TORONTO
SUBM'D AD CHECKED KSH DATE FEB. 2003	SITE
DRAWN NS CHECKED APPROVED	DWG 2

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

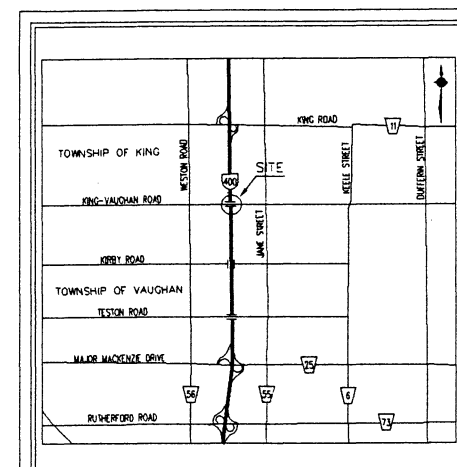
W.P. No. 192-00-00



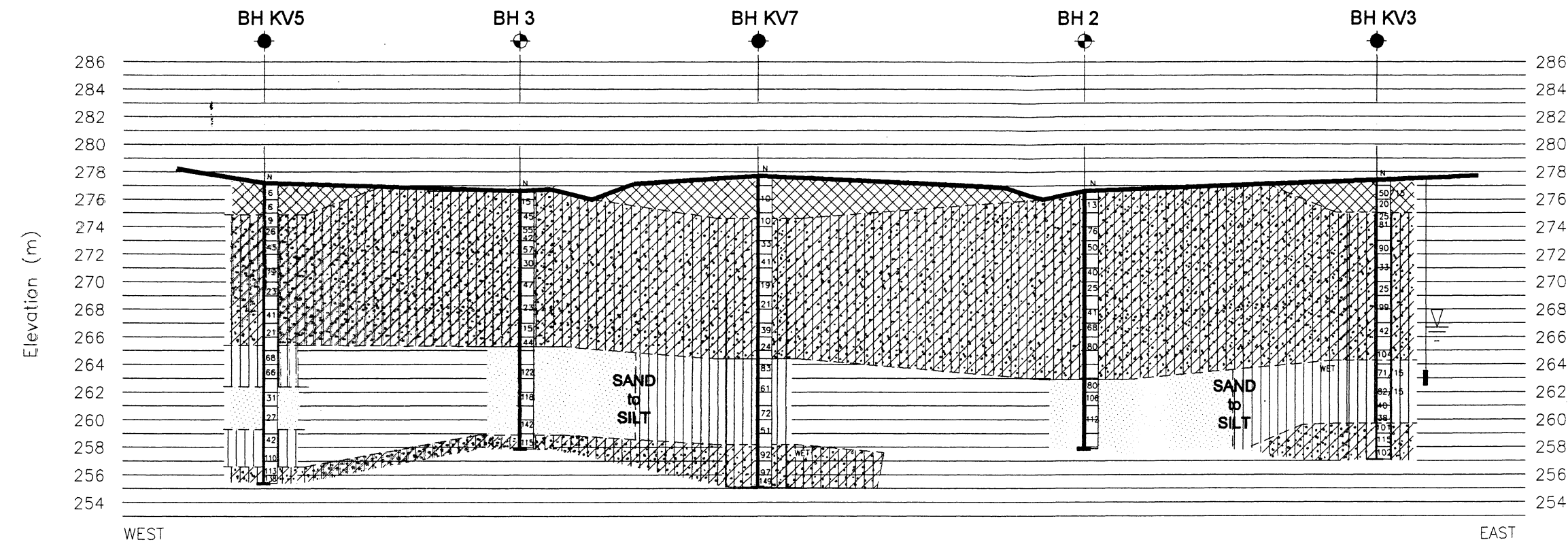
SHEET

SECTION C-C & PROFILE D-D  
CROSSING AT KING-VAUGHAN ROAD

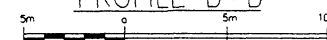
AMEC Earth & Environmental Limited



KEY PLAN  
N.T.S.

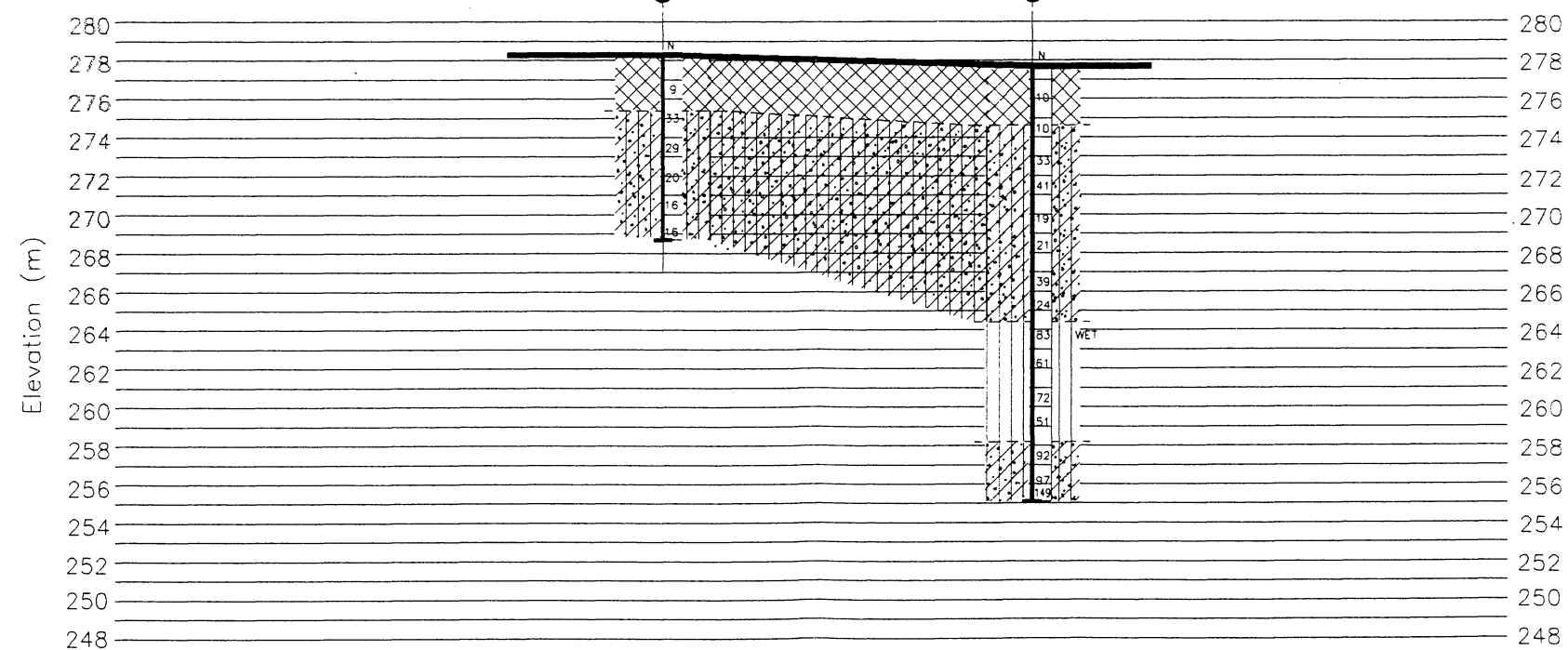


PROFILE D-D



BH KV8

BH KV7



SECTION C-C at PIER



## LEGEND :

- FILL Sand to Clayey Silt  
compact to dense/stiff to  
very stiff
- SILT  
very dense
- SAND to SILTY SAND  
dense to very dense
- SILTY CLAY to  
CLAYEY SILT TILL  
stiff to hard

## LEGEND

- Borehole (AMEC 2003)
- Borehole (DHO 1959)
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation--
- WL in Piezometer
- Piezometer
- End of Borehole

No	ELEVATION	CC-ORDINATES	
		NORTHING	EASTING
BH 1	277.1	4 862 503	299 555
BH 2	276.5	4 862 485	299 858
BH 3	276.6	4 862 474	299 820
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BH KV7	277.7	4 862 479	299 836
BH KV8	278.3	4 862 499	299 833

## NOTE

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

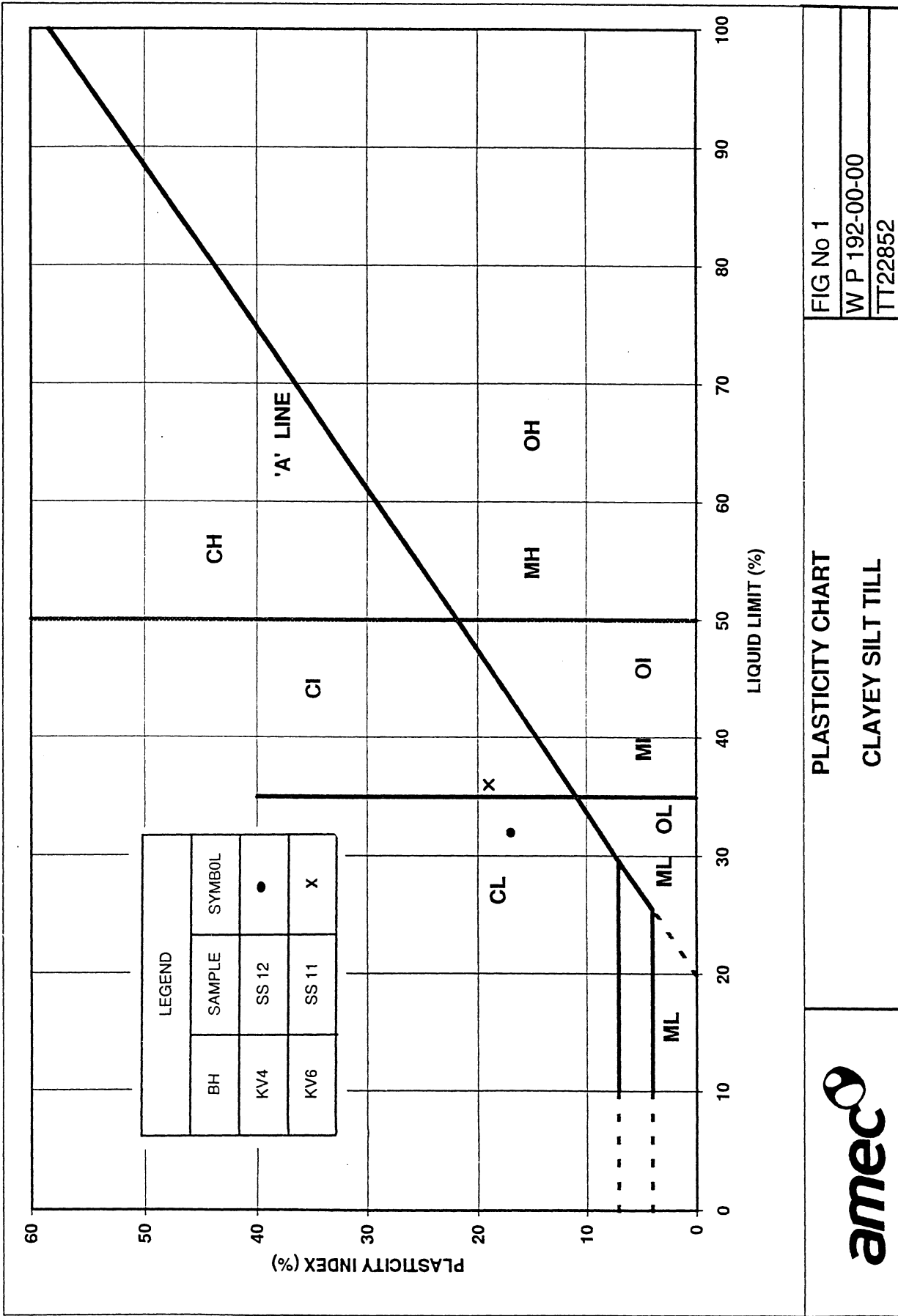
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC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

REV.	DATE	BY	DESCRIPTION
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VAUGHAN AND KING ROAD UNDERPASS	DIST 6-TORONTO
SUBM'D AD CHECKED KSH DATE FEB. 2003	SITE
DRAWN NS CHECKED	DWG 3

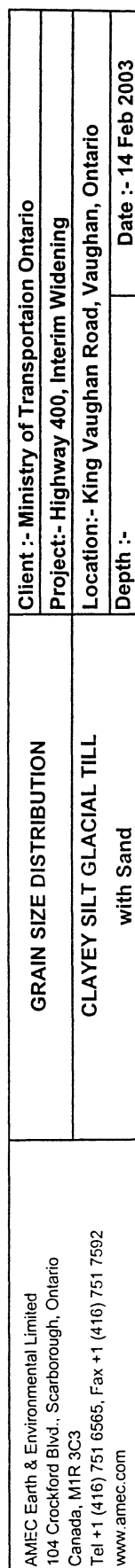
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## FIGURES



**PLASTICITY CHART**  
**CLAYEY SILT TILL**

FIG No 1  
W P 192-00-00  
TT22852

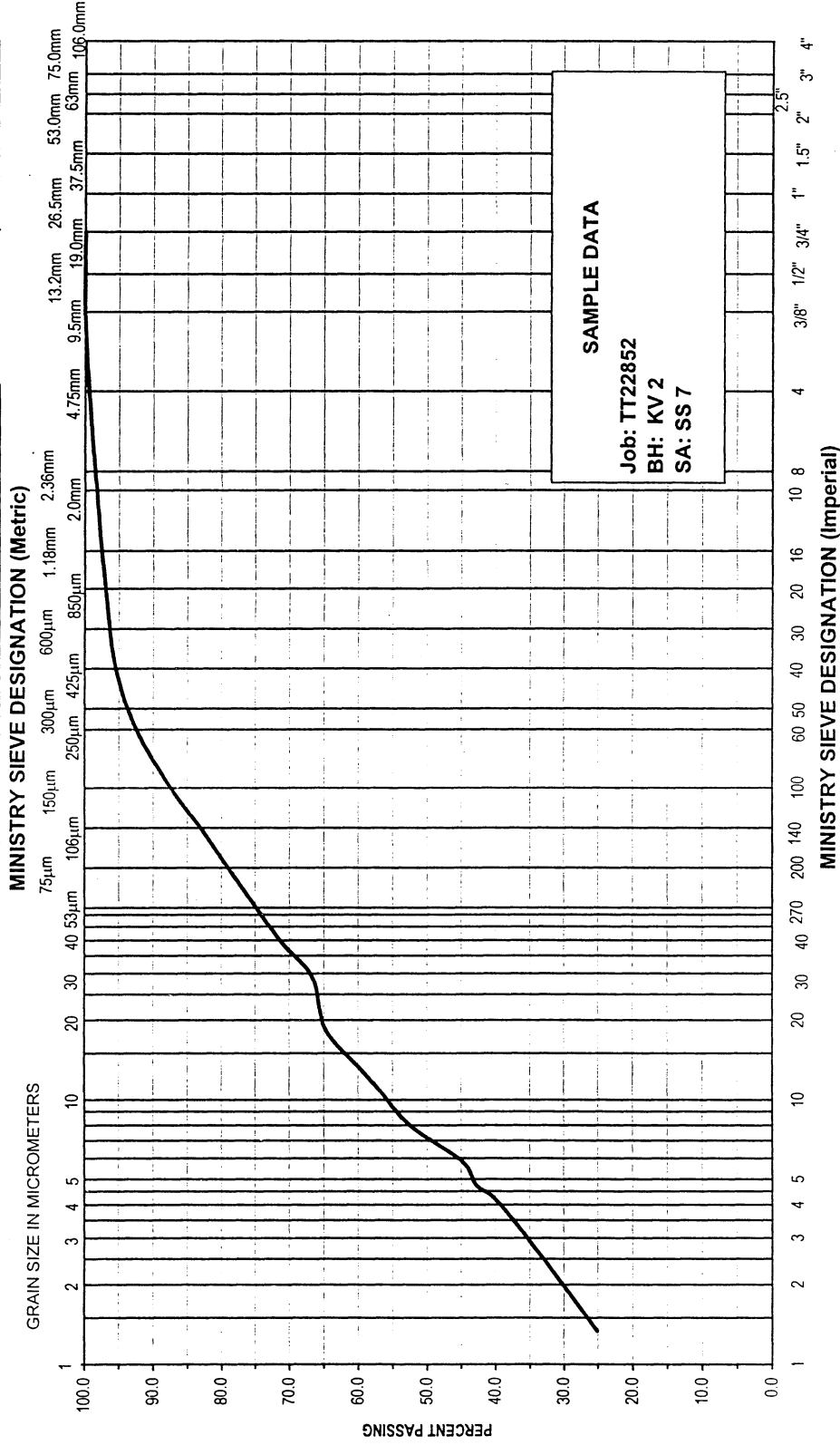


**Figure No. 2**

# UNIFIED SOIL CLASSIFICATION SYSTEM



CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse



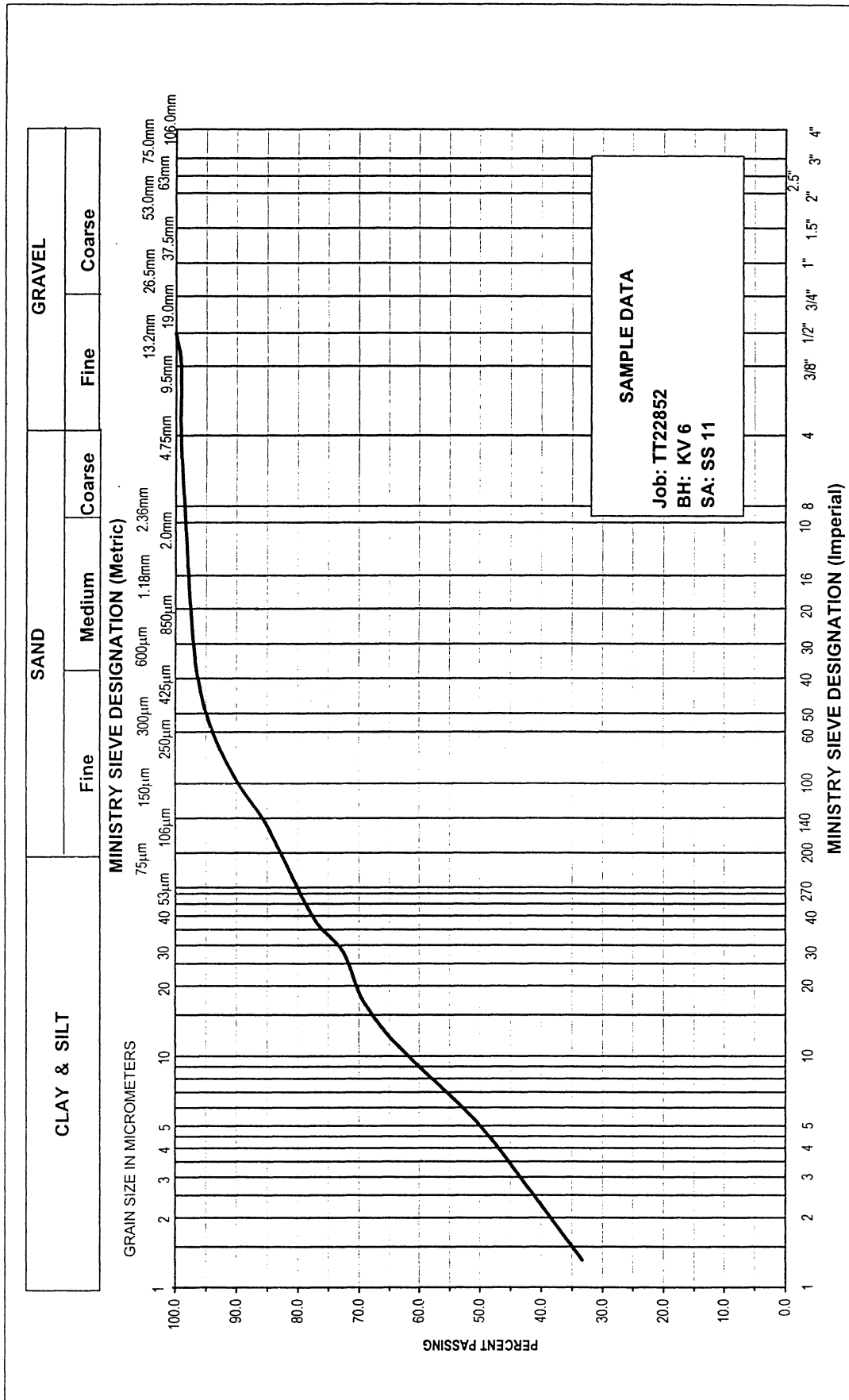
<b>AMEC Earth &amp; Environmental Limited</b> 104 Crockford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6585, Fax +1 (416) 751 7592 www.amec.com	<b>GRAIN SIZE DISTRIBUTION</b>		Client :- Ministry of Transportation Ontario
	CLAYEY SILT GLACIAL TILL		Project:- Highway 400, Interim Widening
		Location:- King Vaughan Road, Vaughan, Ontario	
		Depth :-	Date :- 28 Jan 2003

Figure No. 3



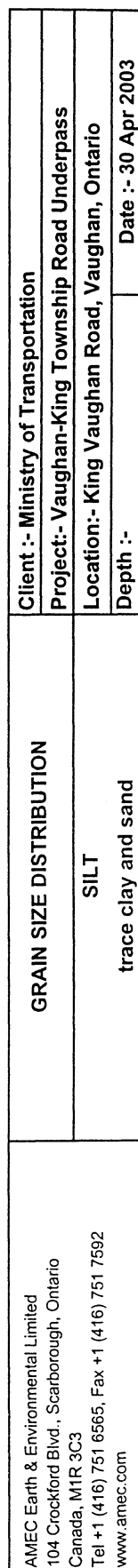


# UNIFIED SOIL CLASSIFICATION SYSTEM



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	CLAYEY SILT GLACIAL TILL		Project:- Highway 400, Interim Widening
	CLAYEY SILT GLACIAL TILL		Location:- King Vaughan Road, Vaughan, Ontario
Depth :-		Date :- 28 Jan 2003	

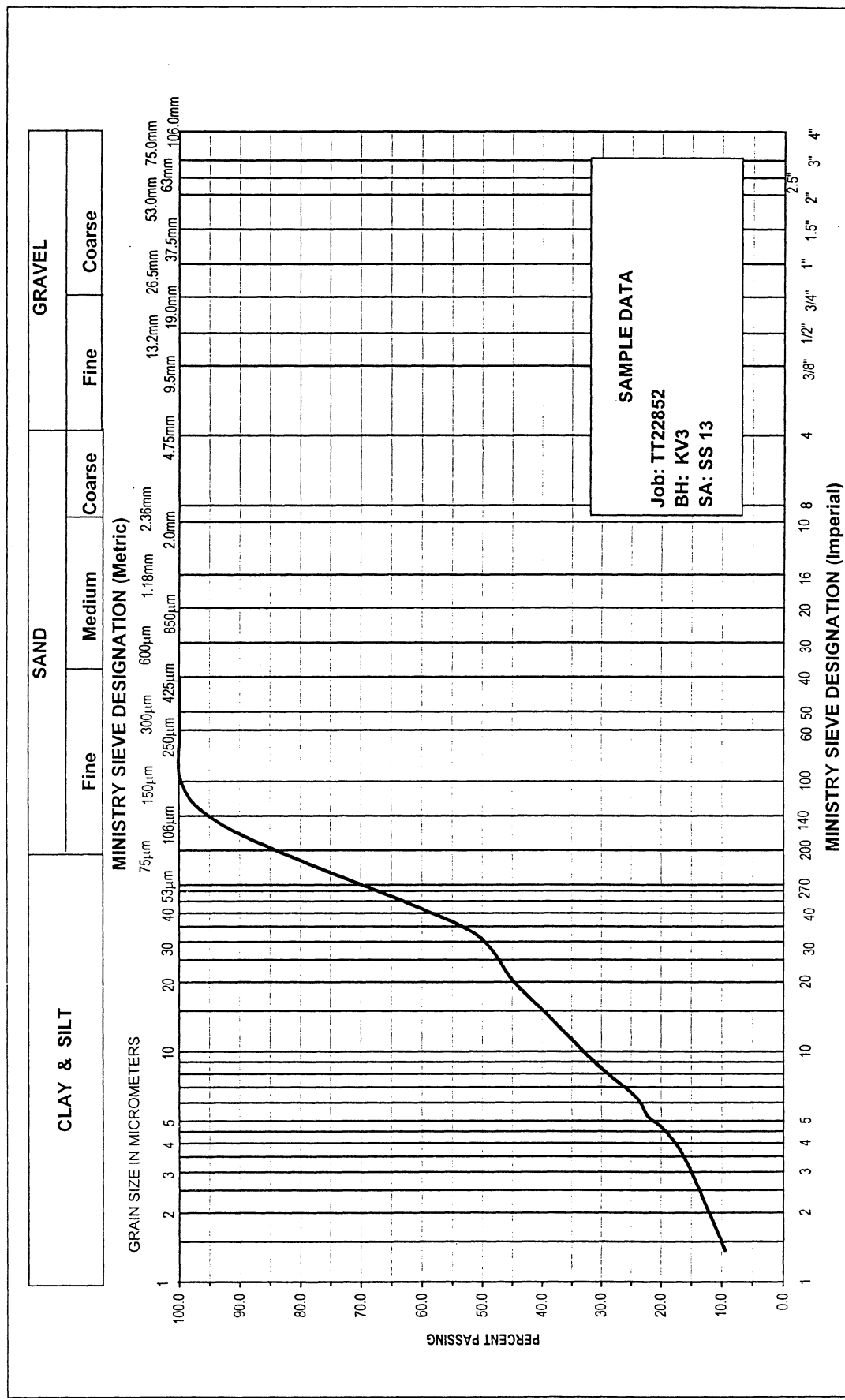
Figure No. 5



**Figure No 6**

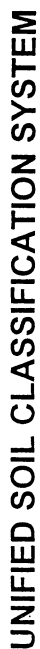


UNIFIED SOIL CLASSIFICATION SYSTEM

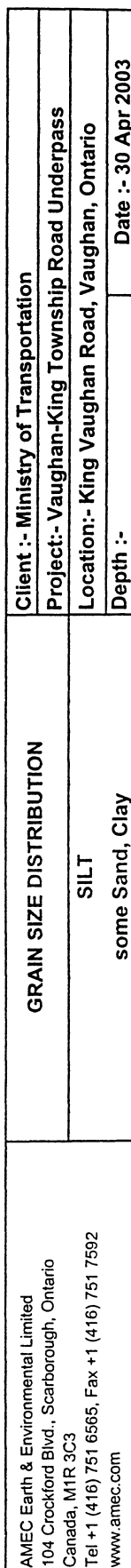


AMEC Earth & Environmental Limited 104 Crockford Blvd., Scarborough, Ontario Canada, M1R 3C3 Tel +1 (416) 751 6585, Fax +1 (416) 751 7592 www.amec.com	GRAIN SIZE DISTRIBUTION		Client :- Ministry of Transportaion Ontario	
			Project:- Highway 400, Interim Widening	
	SILT		Location:- King Vaughan Road, Vaughan, Ontario	
	some Sand, Clay		Depth :-	
			Date :- 14 Feb 2003	

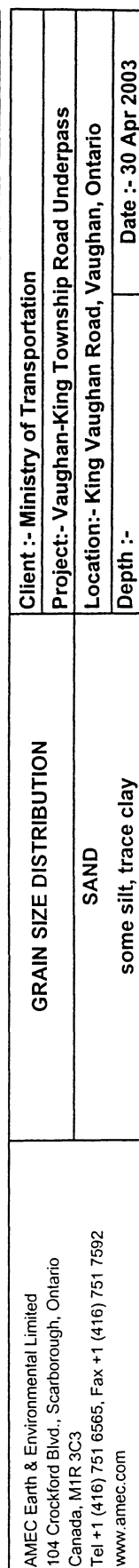
Figure No. 7



# UNIFIED SOIL CLASSIFICATION SYSTEM

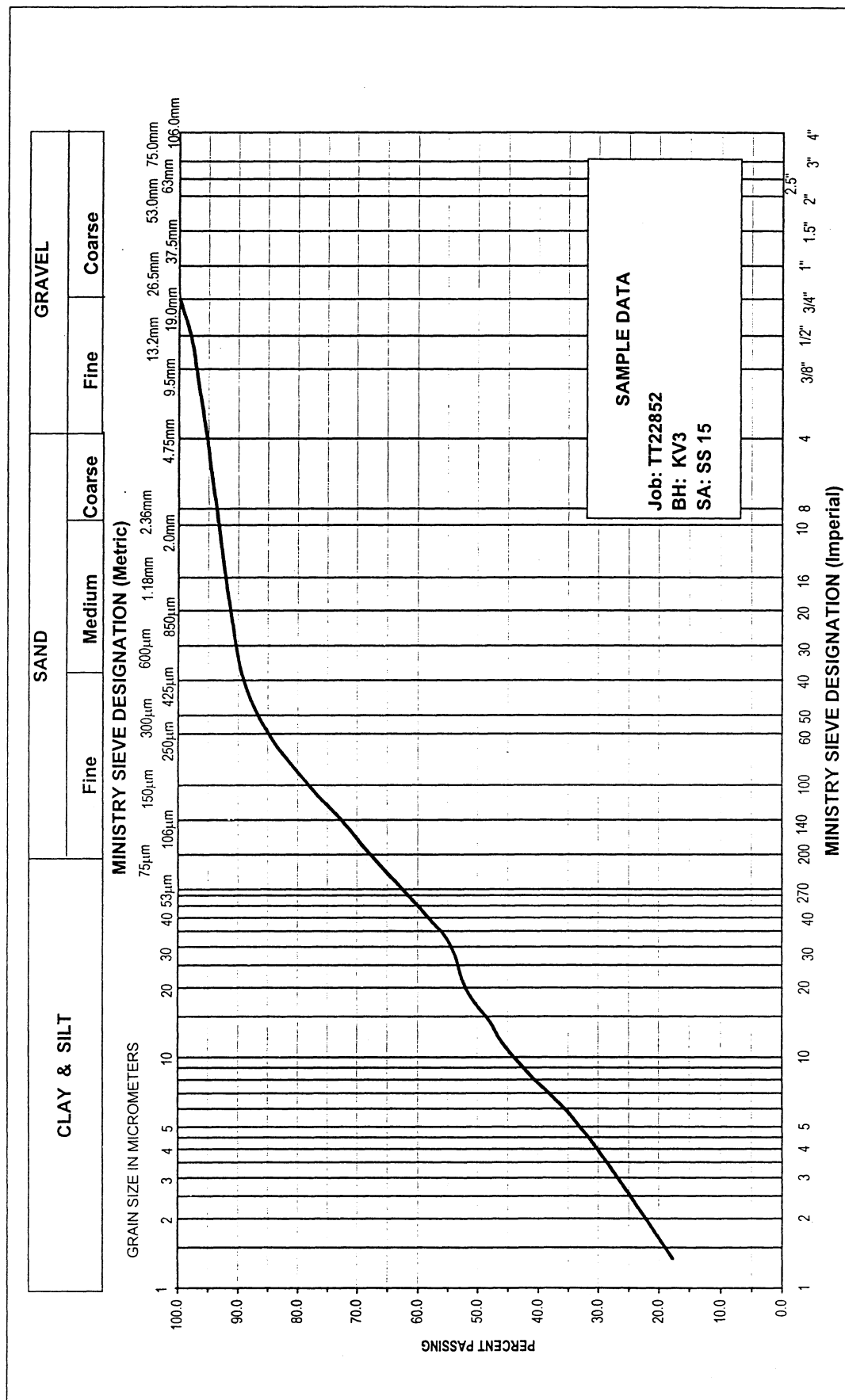


**Figure No. 8**



**Figure No 9**

# UNIFIED SOIL CLASSIFICATION SYSTEM



<p>AMEC Earth &amp; Environmental Limited 104 Crockford 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 Transportaion Ontario
	LOWER CLAYEY SILT GLACIAL TILL		Project:- Highway 400, Interim Widening
			Location:- King Vaughan Road, Vaughan, Ontario
	Depth :-		Date :- 14 Feb 2003

Figure No. 10

**RECORD OF BOREHOLE SHEETS**

## NOTES TO BOREHOLE LOGS

### DRILLING DATA

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

### 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

### LABORATORY DATA

WP - Plastic Limit (%)  
 W - Water Content (%)  
 WL - Liquid Limit (%)  
 $\gamma$  - Natural Unit Weight (kN/m<sup>3</sup>)  
 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

**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 a 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.

**TOTAL CORE RECOVERY (TCR):** Sum of all recovered rock core pieces from a coring run expressed as a percent of the total length of the coring run.

**SOLID CORE RECOVERY (SCR):** Total length of recovered solid, full diameter rock core expressed as a percent of the length of the coring run.

**FRACTURE INDEX:** Number of cracks per 1.5m.

### 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 KV1

**amec**

1 OF 1

W.P. 192-00-00	LOCATION 4862507N 299888E	ORIGINATED BY JF
DIST HWY 400	BOREHOLE TYPE Solid Stem Augering	COMPILED BY PPM
DATUM Geodetic	DATE 9 January 2003 - 9 January 2003	CHECKED BY AD
PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400		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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	GR SA SI CL		
284.0 0.0	Gravelly Sand (FILL) damp, brown, compact		1	SS	18										
283.1 0.9	Clayey Silt (FILL), trace sand moist, brown, very stiff		2	SS	19										
	----- trace rootlets, grey		3	SS	17									20.4	
	----- trace topsoil seams		4	SS	15										
277.3 6.7	CLAYEY SILT (TILL), trace gravel, trace sand lenses, trace rootlets damp, grey, very stiff		5	SS	10										
	----- moist		6	SS	27										
			7	SS	21										
			8	SS	31									21.2	
			9	SS	40										
274.4 9.6	End of Borehole														
	Water Level in Open Bore On Completion : none														

+<sup>3</sup>.X<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3</sup>% STRAIN AT FAILURE

# RECORD OF BOREHOLE No KV2

amec

W.P. 192-00-00 LOCATION 4862501N 299868E 1 OF 1 ORIGINATED BY JF  
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY PPM  
 DATUM Geodetic DATE 9 January 2003 - 9 January 2003 CHECKED BY AD  
 PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400 JOB NO. TT22852

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa					
284.5	110mm ASPHALT								20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
284.4	Sand (FILL), some silt, trace gravel damp, brown, compact		1	SS	17				20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
							1	284						
			2	SS	28		2	283						
281.7	Clayey Silt (FILL), trace sand moist, brown, stiff		3	SS	14		3	282						
2.8	trace topsoil seams						4	281						
			4	SS	12		5	280						
							6	279						
			5	SS	10									
277.5	Sand (FILL), trace gravel wet, brown, dense						7	277						
7.0														
276.7	CLAYEY SILT (TILL), trace gravel, occasional sand lenses moist, brown, hard		6	SS	31		8	276						
7.8							9	275						
			7	SS	34		10	274						
			8	SS	59		11	273						
			9	SS	91		12	272						
	grey													
271.8	End of Borehole		10	SS	49									
12.7	Water Level in Open Bore On Completion : none													

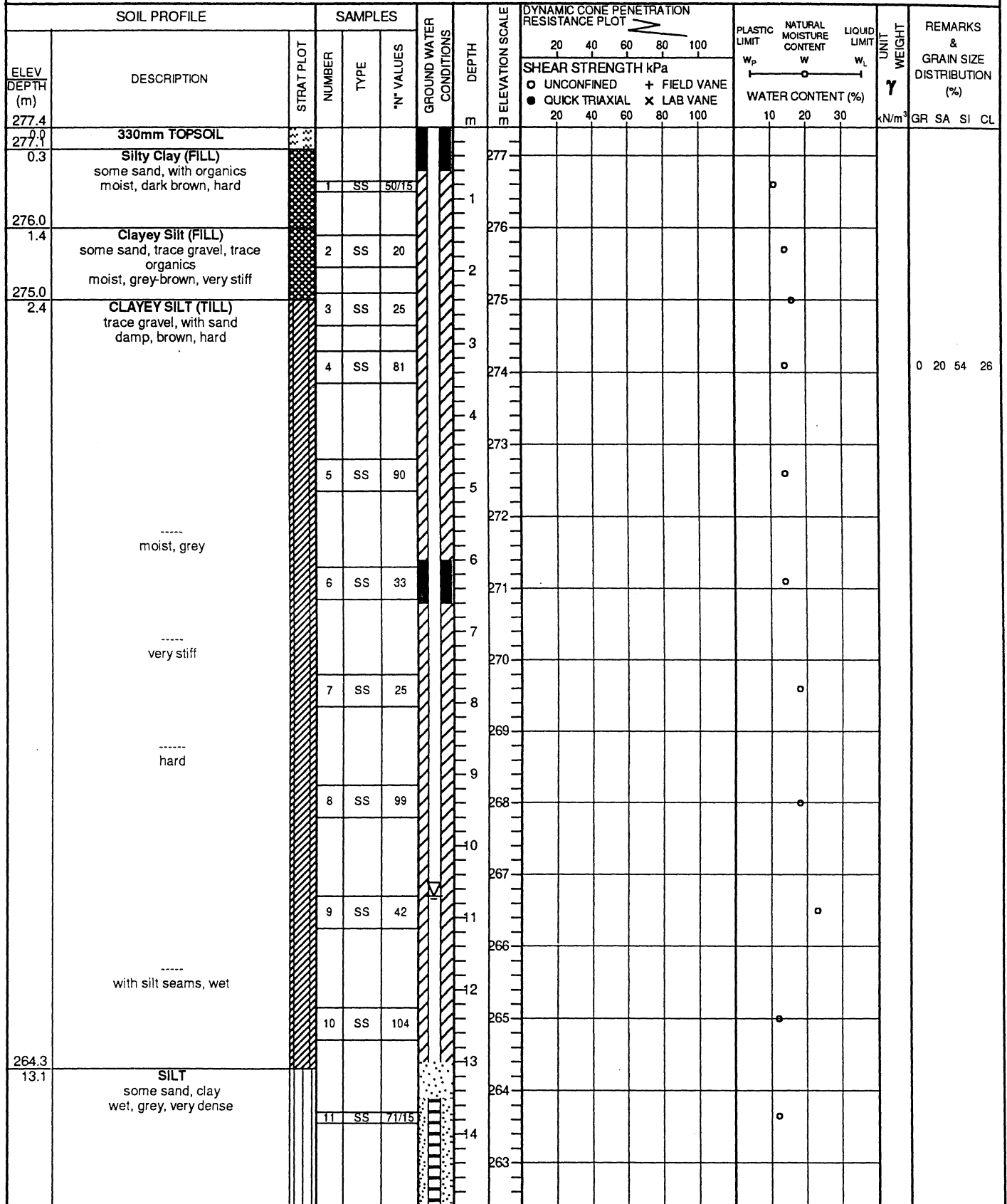
+3, x3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No KV3

amec

1 OF 2

W.P. 192-00-00 LOCATION 4862478N 299880E ORIGINATED BY JF  
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY PPM  
 DATUM Geodetic DATE 8 February 2003 - 8 February 2003 CHECKED BY AD  
 PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400 JOB NO. TT22852



Continued Next Page

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

**amec**®

2 OF 2

**+<sup>3</sup>, X<sup>3</sup>:** Numbers refer to Sensitivity      **O<sup>3</sup>%** STRAIN AT FAILURE

# RECORD OF BOREHOLE No KV4

amec

1 OF 1

W.P. 192-00-00 LOCATION 4862485N 299808E ORIGINATED BY JF  
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY PPM  
 DATUM Geodetic DATE 9 January 2003 - 9 January 2003 CHECKED BY AD  
 PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400 JOB NO. TT22852

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa						
284.5	120mm ASPHALT								20 40 60 80 100						
283.4	Sand (FILL), with gravel, trace silt damp, brown, loose		1	AS	-		1	284	○ UNCONFINED + FIELD VANE						
283.1	Clayey Silt (FILL) moist, brown, stiff		2	SS	9				● QUICK TRIAXIAL x LAB VANE						
283.1	Sand and Gravel (FILL), trace silt damp, brown, dense		3	SS	37		2	283							
281.1	some silt, loose		4	SS	16		3	282							
279.0	Clayey Silt (FILL), trace sand, moist, brown, very stiff		5	SS	13		4	281							
279.0	stiff		6	SS	14		5	280							
271.8	CLAYEY SILT (TILL), trace gravel, some sand moist, brown, very stiff		7	AS	-		6	279							
271.8	hard		8	SS	16		7	278							
			9	SS	27		8	277							
			10	SS	23		9	276							
			11	SS	40		10	275							
			12	SS	46		11	274							
			13	AS	-		12	273							
			14	SS	43			272							
271.8	End of Borehole														
12.7	Water Level in Open Bore On Completion : none														

+3, x3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No KV5

amec

1 OF 2

W.P. 192-00-00 LOCATION 4862454N 299803E ORIGINATED BY JF  
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY PPM  
 DATUM Geodetic DATE 22 February 2003 - 22 February 2003 CHECKED BY AD  
 PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400 JOB NO. TT22852

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa						
277.2 0.0	450mm TOPSOIL								20 40 60 80 100						
276.8 0.5	Silty Clay (FILL) some sand, trace organics moist, grey firm		1	AS	-		1	277	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
	----- trace sand, some organics dark brown		2	SS	6		2	276							
275.1 2.1	CLAYEY SILT to SILTY CLAY (TILL) trace gravel, trace sand moist, grey, stiff		3	SS	6		3	275							
	----- damp, brown, very stiff		4	SS	9		4	274							
	----- hard		5	SS	26		5	273							
	----- grey, very stiff		6	SS	45		6	272							
	----- trace sand lenses, hard		7	SS	19		7	271							
	----- moist to wet, very stiff		8	SS	23		8	270							
			9	SS	41		9	269							
			10	SS	21		10	268							
265.6 11.6	SILT occasional gravel, some sand, clay wet, grey, very dense		11	SS	68		11	267							
			12	SS	66		12	266							
262.6 14.6	SAND some silt, trace clay						13	265							1 18 67 14
							14	264							
								263							

Continued Next Page

+<sup>3</sup>, x<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

**amec**

2 OF 2

W.P. 192-00-00 LOCATION 4862454N 299803E 2 OF 2  
 DIST HWY 400 BOREHOLE TYPE Soild Stem Augering ORIGINATED BY JF  
 DATUM Geodetic DATE 22 February 2003 - 22 February 2003 COMPILED BY PPM  
 PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400 CHECKED BY AD  
 JOB NO. TT22852

[illegible]

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      O<sup>3</sup>% STRAIN AT FAILURE

**amec**

ORIGINATED BY JF

LOCATION 4862472N 299787E

**BOREHOLE TYPE** Soild Stem Augering

COMPILED BY PPM

DATE 9 January 2003 - 9 January 2003

— CHECKED BY AD

JOB NO. TT22852

**+<sup>3</sup>, ×<sup>3</sup>:** Numbers refer to Sensitivity      **○<sup>3%</sup>** STRAIN AT FAILURE

# RECORD OF BOREHOLE No KV7



1 OF 2

W.P. 192-00-00 LOCATION 4862479N 299836E ORIGINATED BY JF  
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Augering COMPILED BY IH  
 DATUM Geodetic DATE 15 March 2003 - 15 March 2003 CHECKED BY AD  
 PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400 JOB NO. TT22852

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE ELEVATION (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES				20 40 60 80 100	20 40 60 80 100					
277.7	280mm ASPHALT														
277.4	Sand and Gravel (FILL), trace silt damp, brown		1	AS	-			277							
0.3	Clayey Silt (FILL) some sand, organics, rootlets brown, stiff, damp		2	SS	10		1	276							
276.8			3	AS	-		2	275							
0.9							3	274							
274.6	stiff -----		4	SS	10		4	273							
3.1	brown CLAYEY SILT (TILL) some sand, trace gravel, occasional cobbles damp to moist, brown, very stiff to hard		5	SS	33		5	272							
			6	SS	41		6	271							
							7	270							
			7	SS	19		8	269							
							9	268							
			8	SS	21		10	267							
							11	266							
							12	265							
			9	SS	39		13	264							
							14	263							
264.4	SAND some silt, trace clay wet, grey, very dense		10	SS	24										
13.3			11	SS	83										

Continued Next Page

+3.X3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

**amec**<sup>®</sup>

2 OF 2

ORIGINATED BY JF

COMPILED BY IH

- CHECKED BY AD

JOB NO. TT2

JOB NO. TT2

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      O<sup>3</sup>% STRAIN AT FAILURE

# RECORD OF BOREHOLE No KV8

amec

1 OF 1

W.P. 192-00-00 LOCATION 4862499N 299833E ORIGINATED BY JF  
 DIST HWY 400 BOREHOLE TYPE Solid Stem Augering COMPILED BY IH  
 DATUM Geodetic DATE 15 March 2003 - 15 March 2003 CHECKED BY AD  
 PROJECT Vaughan-King Township, Road Underpass Widening, HWY 400 JOB NO. TT22852

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE ELEVATION (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa						
278.3									20 40 60 80 100						
278.0	250mm ASPHALT								20 40 60 80 100						
0.3	Sand and Gravel (FILL), trace silt damp, brown		1	AS	-			278							
277.4	Silty Sand (FILL), trace Clay damp, brown							1							
0.9								277							
276.8	dark brown Clayey Silt (FILL), trace sand, organics damp, brown, stiff		2	SS	9			2							
1.5								276							
275.4	brown CLAYEY SILT (TILL), some sand, trace gravel damp, brown, very stiff to hard		3	SS	33			3							
2.9								275							
								4							
			4	SS	29			274							
								5							
								273							
			5	SS	20			272							
								7							
			6	SS	16			271							
								270							
268.7	End of Borehole		7	SS	16			269							
9.6	Water Level in Open Bore On Completion : none														

+3.X3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

**APPENDIX 'A'**

**Borehole Logs extracted from report prepared by Department of Highways Ontario  
dated 1959  
GEOCRES File No. 30M13-2**

9 P. --- 267 - 59 ---  
 BORE HOLE NO. 1  
 LOG --- 259-57 ---  
 STATION Q73 (61' RT)  
 DATUM --- 909.0' ---  
 COUPLED BY --- JAL  
 DRIVING DATE - June 5/59  
 CHECKED BY --- M.D.

9 P. --- 267 - 59 ---  
BONE HOLE NO. --- 1

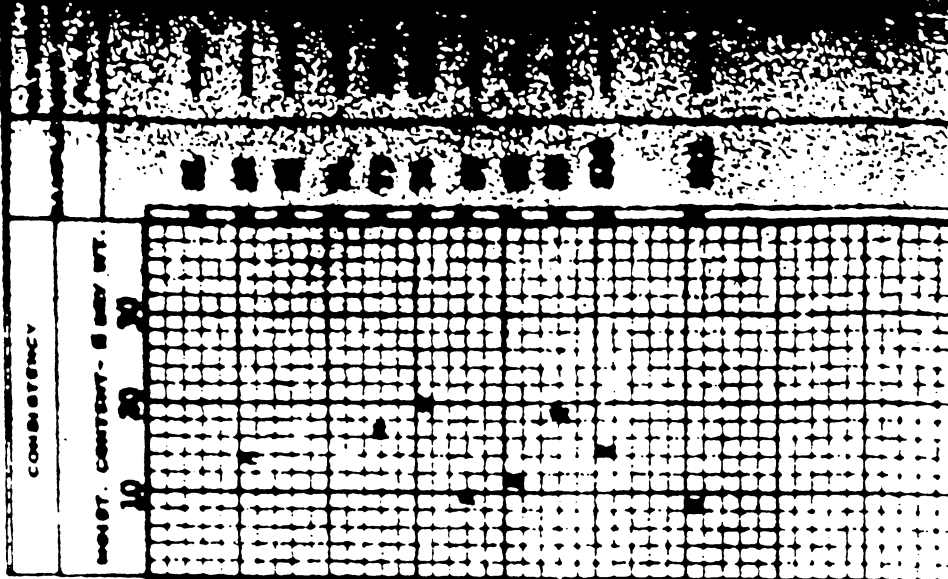
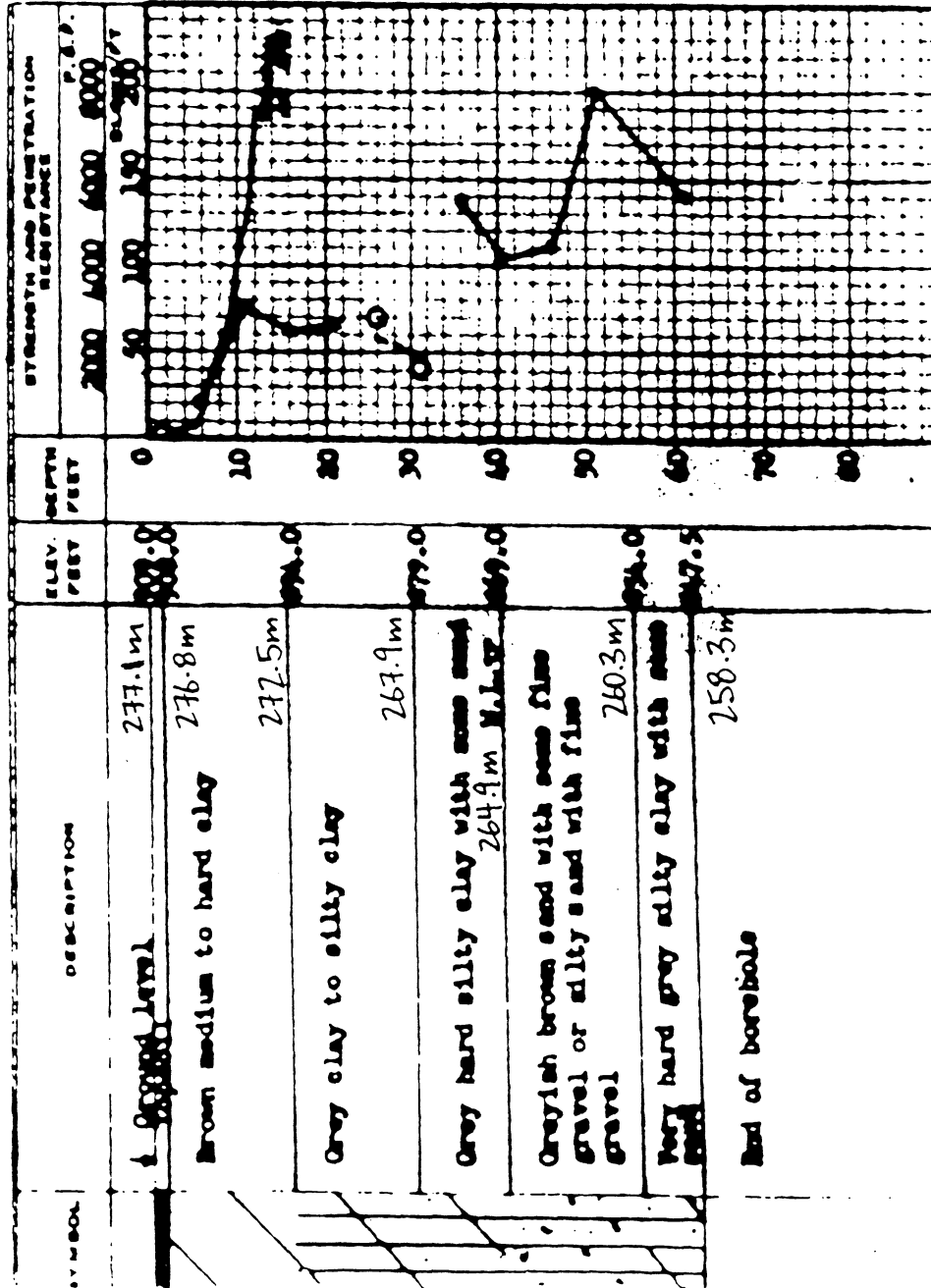
100 259-57 STATION 0-53 (65' IN) -----

DATUM - 502.0' COMPILED BY J.L.

PRINTING DATE - JUN 5/59. CHECKED BY - M.D.---

## LEGEND

1/2 LUNGOW MED COMBIBOOS (001-  
WAGE TEST(C) AND SUBSTANTIV(8)  
NATURAL MOISTURE AND  
LIGHTLY MOIST  
LIGHT GRAY  
LIGHT PLASTIC  
LIGHT



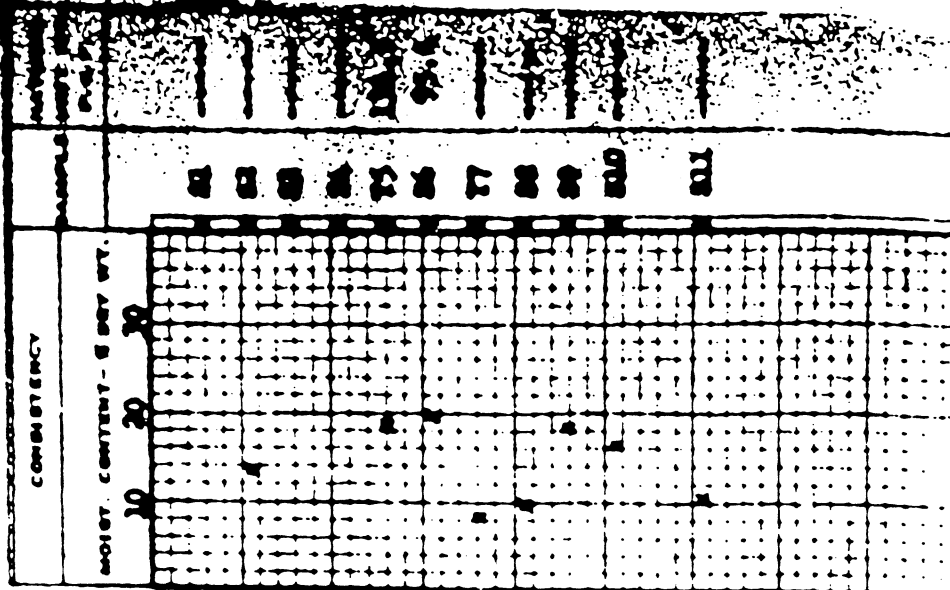
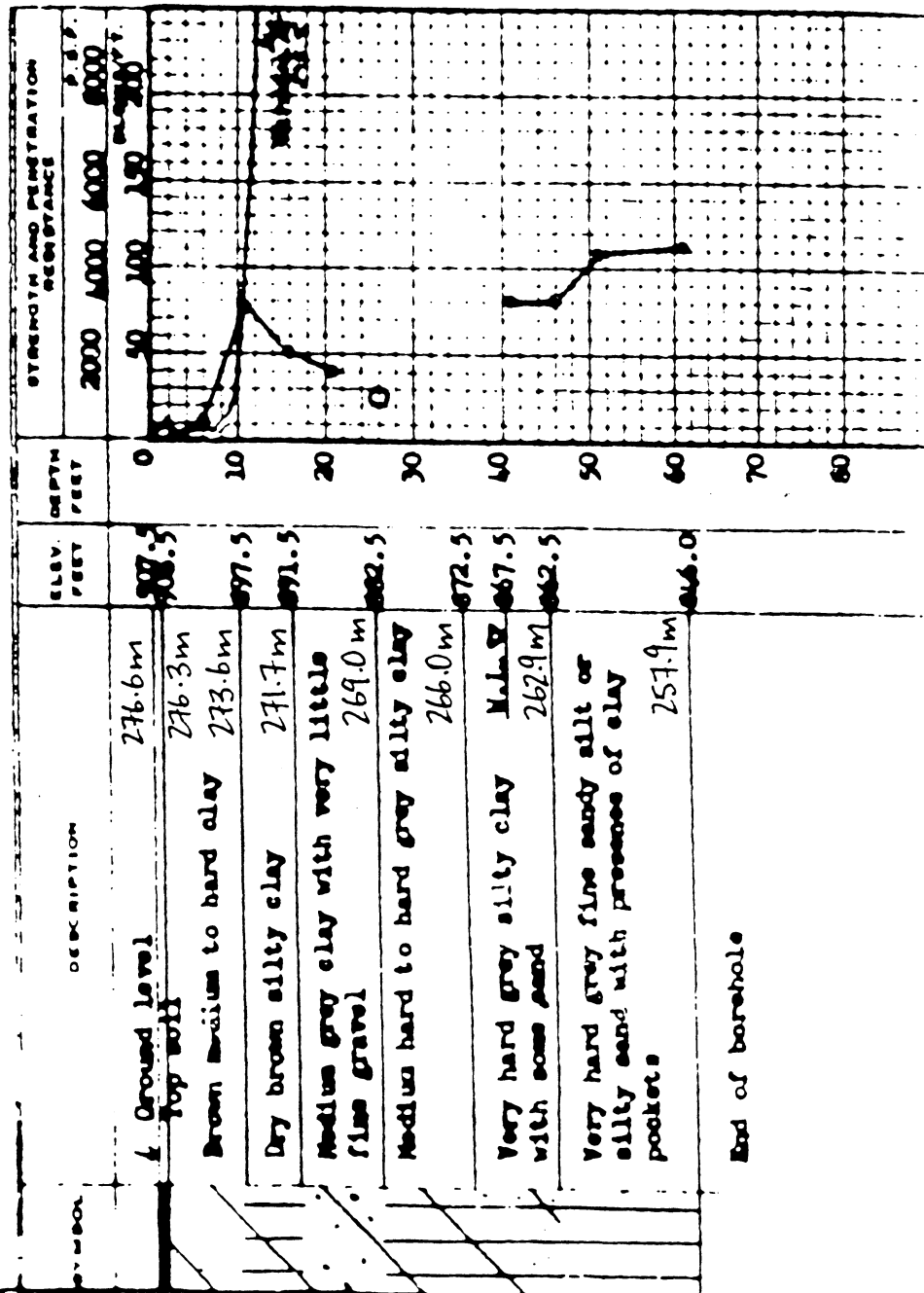
# DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 262 - 59 BORE HOLE NO. 2  
JOB 759 - 57 STATION 44+16 (62' E.C.)  
DATUM 907.5' COMPILED BY B.J.  
BORING DATE June 8/59 CHECKED BY M.D.

## LEGEND

1/2" UNCOMPIED COMPRESSOR (Q<sub>u</sub>)  
WANE TEST (C) AND SENSITIVITY (S)  
NATURAL MOISTURE AND  
LIQUIDITY INDEX  
LIQUID LIMIT  
PLASTIC LIMIT

1/2" DIA. SPLIT TUBE  
1/2" SHELBY TUBE  
1/2" DIA. CONE  
1/2" SHELBY  
CASING



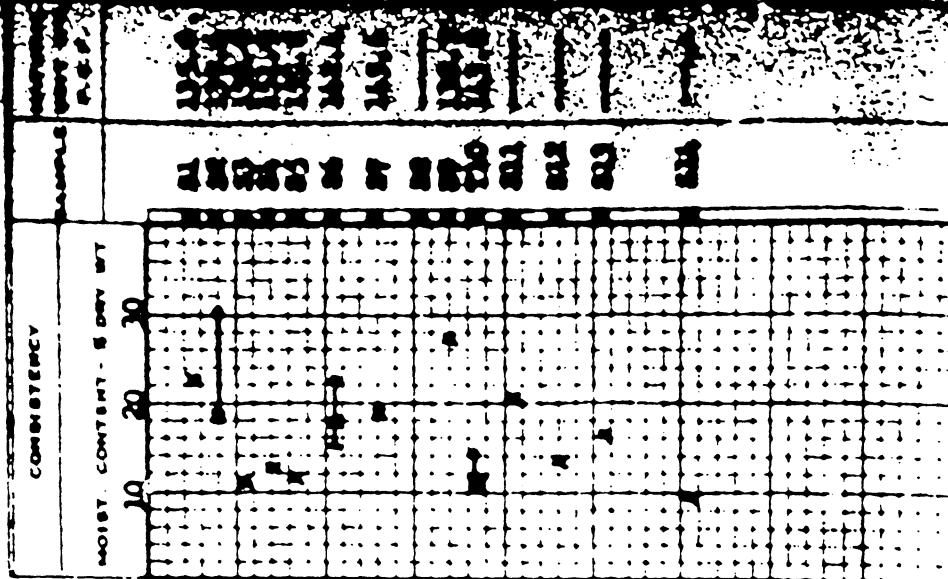
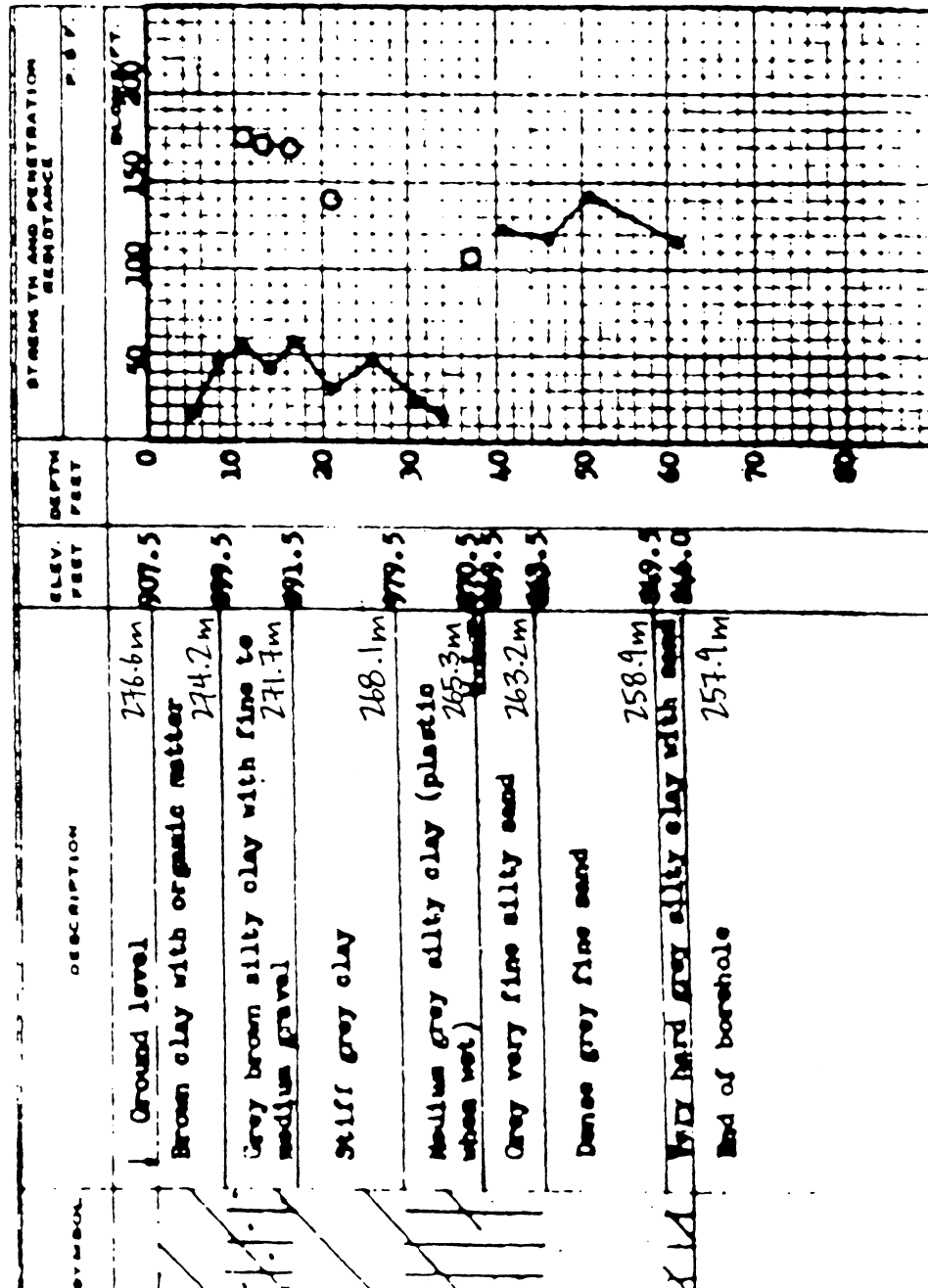
# DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. - 267 - 59 BORE HOLE NO. 1  
JOB 759 - 57 STATION 46+01 (65' RE)  
DATUM 907.5' COMPILED BY B.J.A.  
BORING DATE June 9/59 CHECKED BY B.J.A.

## LEGEND

1" DIA. SPLIT TUBE  
2" SHELBY TUBE  
3" SPLIT TUBE  
4" DIA. CONE  
5" SHELBY  
CASING

1/2 UNCONFINED COMPRESSION (Qu)  
WANE TEST (C) AND SENSITIVITY (S)  
NATURAL MOISTURE AND  
LIQUIDITY INDEX  
LIQUID LIMIT  
PLASTIC LIMIT



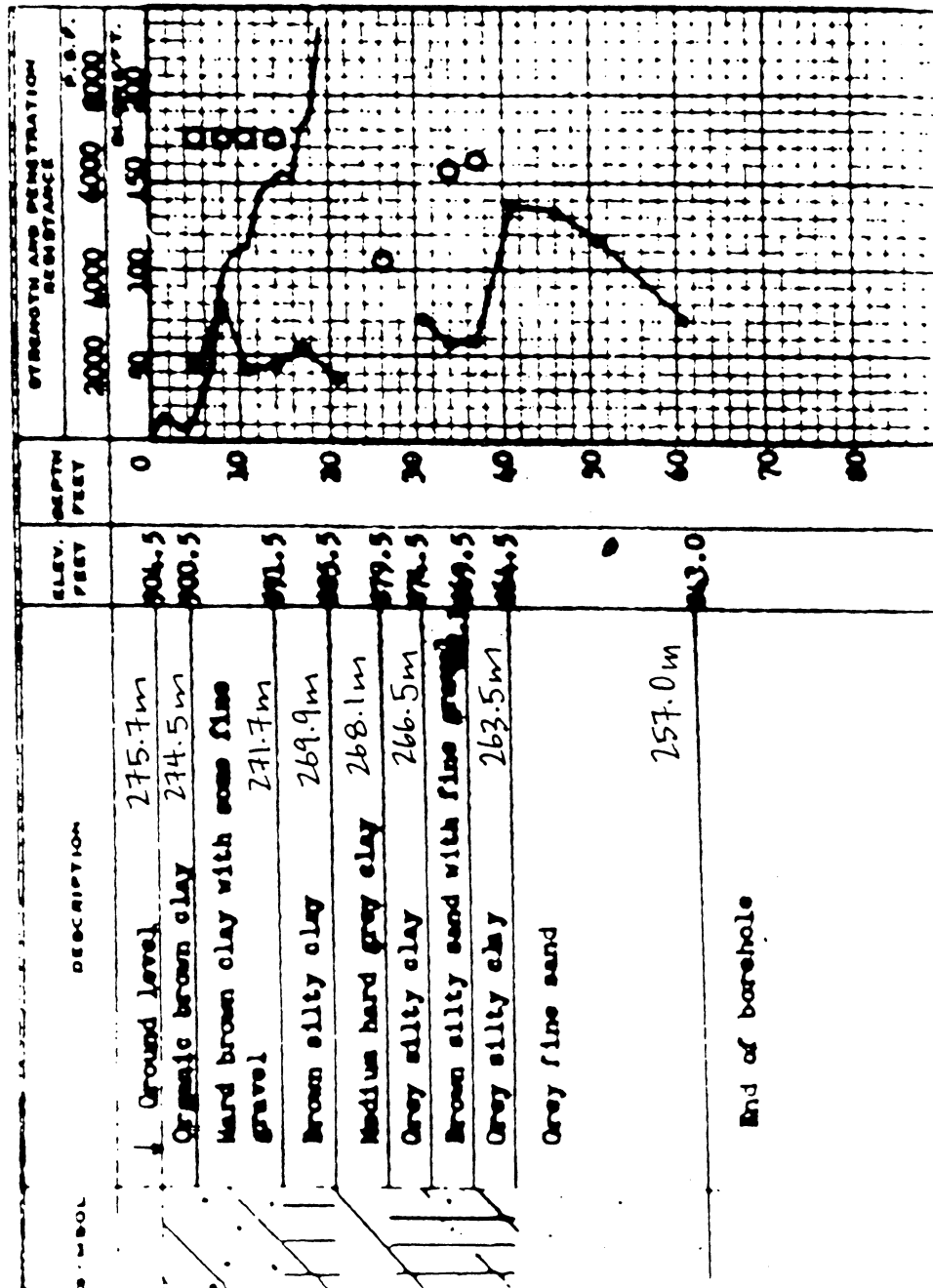
# DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 267 - 59 BORE HOLE NO. A  
JOB 758 - 57 STATION 0+00 (62' RC)  
DATUM 904.5' COMPILED BY J.L.  
BORING DATE June 9/59 CHECKED BY J.L.

## LEGEND

1" DIA SPLIT TUBE  
2" SHELBY TUBE  
3" SPLIT TUBE  
4" DIA CONE  
5" SHELBY  
6" CASING

1/2 UNCONFINED COMPRESSION (QU)  
VANE TEST (C) AND DERIVATIVES  
NATURAL MOISTURE AND  
LIQUIDITY INDEX  
LIQUID LIMIT  
PLASTIC LIMIT



End of borehole

JOB 159-57  
W P 267-58

# SUMMARY OF FIELD & LABORATORY TESTS

MOLE NO.	TEST NO.	DEPTH (FEET)	NATURAL DESCRIPTION	PERCENT MOISTURE (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	SHRINKAGE (%)	UNIT WEIGHT (PCF)	REMARKS
1	34	5' - 6.5'	Brown medium to hard clay	20	--	--	--	--	
	32	10' - 11.5'	Brown medium to hard clay	75	13.5	--	--	--	
	33	15' - 16.5'	Brown medium to hard clay	62	--	--	--	--	
	34	20' - 21.5'	Grey clay to silty clay	65	--	--	--	--	
	35	25' - 26.5'	Grey clay to silty clay	59	16.6	--	2750	136.0	
	36	30' - 31.5'	Grey hard silty clay with some sand	49	19.9	--	1650	134.1	
	37	35' - 36.5'	Grey hard silty clay with some sand	136	9.4	--	--	--	
	38	40' - 41.5'	Greyish brown sand with some gravel or silty sand with fine gravel	>100	11.1	--	--	--	
	39	45' - 46.5'	Greyish brown sand with some gravel or silty sand with fine gravel	110	18.7	--	--	--	
	310	50' - 51.5'	Greyish brown sand with some gravel or silty sand with fine gravel	200	14.5	--	--	--	
	311	60' - 61.5'	Very hard grey silty clay with some sand	140	8.6	--	--	--	
2	31	5' - 6.5'	Brown medium to hard clay	13	--	--	--	--	
	32	10' - 11.5'	Dry brown silty clay	76	13.4	--	--	--	
	33	15' - 16.5'	Dry brown silty clay	90	--	--	--	--	
	34	20' - 21.5'	Medium grey clay with very little fine	40	--	--	--	--	
	35	25' - 26.5'	Medium hard to hard grey silty clay	25	16.6	--	1070	131.5	
	36	30' - 31.5'	Medium hard to hard grey silty clay	41	19.5	--	--	95.4	
	37	35' - 36.5'	Very hard grey silty clay with some sand	68	8.2	--	--	--	
	38	40' - 41.5'	Very hard grey silty clay with some sand	80	9.6	--	--	--	
	39	45' - 46.5'	Very hard grey fine sandy silt or silty sand with presence of clay pockets	80	16.3	--	--	--	
	310	50' - 51.5'	Very hard grey fine sandy silt or silty sand with presence of clay pockets	106	16.2	--	--	--	
	311	60' - 61.5'	Very hard grey fine sandy silt or silty sand with presence of clay pockets	112	10.4	--	--	--	

JOB 158 - 57  
W.P. 267 - 57

# SUMMARY OF FIELD & LABORATORY TESTS

SOIL NO	SAMP NO	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PERCENT MOISTURE (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	SHEAR STRENGTH (PSI)	UNIT WEIGHT (PCF)	REMARKS
3	31	4' - 5.5'	Brown clay with organic matter	15	22.4	--	---	135.0	
	32	7' - 8.5'	Grey brown silty clay with fine to medium gravel	45	16.8	30.4	---	139.6	
	33	10' - 11.5'	Grey brown silty clay with fine to medium gravel	55	11.1	--	7000	150.0	
	34	13' - 14.5'	Grey brown silty clay with fine to medium gravel	42	12.6	--	6900	151.9	
	35	16' - 17.5'	Stiff grey clay	57	11.6	--	6850	153.8	
	36	20' - 21.5'	Stiff grey clay	30	17.6	15.0	9750	142.4	
	37	25' - 26.5'	Stiff grey clay	47	19.0	--	---	145.6	
	38	30' - 31.5'	Medium grey silty clay (plastic when wet)	23	--	--	---	---	
	39	33' - 34.5'	Medium grey silty clay (plastic when wet)	15	27.1	--	---	138.7	
	40	36' - 37.5'	Grey very fine silty sand	44	10.2	11.9	4280	143.6	
	41	40' - 41.5'	Grey very fine silty sand	122	20.6	--	---	---	
	42	45' - 46.5'	Dense very fine silty sand	118	13.5	--	---	---	
	43	50' - 51.5'	Dense very fine silty sand	142	16.6	--	---	---	
	44	60' - 61.5'	Very hard grey silty clay with sand	115	9.6	--	---	---	
4	51	4.0' - 5.5'	Hard brown clay with some fine gravel	44	17.0	--	7000	142.1	
	52	7.0' - 8.5'	Hard brown clay with some fine gravel	78	16.0	--	7000	137.6	
	53	10' - 11.5'	Hard brown clay with some fine gravel	42	15.0	--	7000	147.5	
	54	13' - 14.5'	Brown silty clay	44	15.5	28.1	7000	137.4	
	55	16' - 17.5'	Brown silty clay	54	14.3	--	---	---	
	56	20' - 21.5'	Medium hard grey clay	36	19.5	--	4100	136.9	
	57	25' - 26.5'	Grey silty clay	58	17.2	--	---	---	
	58	30' - 31.5'	Brown silty sand with fine gravel	72	18.8	--	---	---	
	59	33' - 34.5'	Brown silty sand with fine gravel	57	16.7	--	6150	144.8	
	60	36' - 37.5'	Grey silty clay	58	26.4	--	6570	---	
	61	40' - 41.5'	Grey fine sand	137	15.3	--	---	---	
	62	45' - 46.5'	Grey fine sand	133	17.3	--	---	---	
	63	50' - 51.5'	Grey fine sand	116	15.0	--	---	---	
	64	60' - 61.5'	Grey fine sand	70	15.8	--	---	---	

S Denotes split spoon sample

T Denotes thin wall ed Shelby tube