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Wo 96 110001

**REPORT ON**

**GEOTECHNICAL INVESTIGATION AND BASELINE  
TWIN TUNNELED STORM SEWERS  
VELORE WOODS AND HIGHWAY 400  
VAUGHAN, ONTARIO**

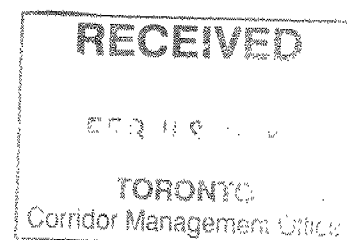
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## 1.0 INTRODUCTION

This is the geotechnical report for the proposed twin tunnelled sewers beneath Highway 400 in Vaughan, Ontario. It is understood that two 2,550 mm inside diameter (I.D.) storm sewers, spaced at 6 m between centres, will be put underneath Highway 400 as part of the Vellore Woods Community development along the west side of the highway.

Golder Associates Ltd. was retained by Schaeffer & Associates Ltd. to provide input to the geotechnical engineering and design issues specifically associated with the tunnelling work. The scope of the project is outlined in our proposal letter No. P81-1528, dated November 25, 1998. Appendix A of that proposal letter contains recommended guidelines for a subsurface investigation at the subject site. Golder Associates Ltd. was requested by Schaeffer & Associates Ltd. to carry out a subsurface investigation according to the provided guidelines and as outlined in our facsimile transmittal dated November 26, 1998.

The purpose of this report is to describe subsurface conditions anticipated along the sewer alignment, to present recommendations and comments on the geotechnical aspects of design of the works and to provide an interpretation of the ground behaviour in relation to anticipated construction operations. This report establishes the geotechnical baseline for the tunnelling contract and is intended to assist in evaluating the requirements for tunnelling, excavation, temporary support and groundwater control.

The content of this report is based on subsurface information obtained from boreholes put down along the proposed sewer alignment from the current Golder Associates Ltd. investigation and relevant subsurface information for this area obtained by others. This report has been specifically prepared for use on the project described above by Schaeffer & Associates Ltd. and the current project owner. Golder Associates Ltd. disclaim responsibility for any use of the information, data, interpretations or recommendations provided in this report by any other parties or for any other project.



## 2.0 SITE AND PROJECT DESCRIPTION

The site is located along Highway 400 in the City of Vaughan, north of Toronto between Rutherford Road and Major Mackenzie Drive (Figure 1).

Construction of the Vellore Woods Community development requires a storm water management system. As part of this system, twin 2,550 mm I.D. storm sewer pipes have been proposed to be placed by tunnelling methods under Highway 400 to convey run-off away from the development located on the west side of the Highway. In addition to the sewers, working shafts are proposed for the west and east sides of Highway 400 to provide access for construction. It is understood that the Ministry of Transportation, Ontario (MTO) has stipulated the shafts be placed at a minimum of 5 m from the proposed edge of shoulders of the future 10-lane Highway 400 corridor. This restriction and the ultimate Highway 400 design width require the length of tunnelling to be about 70 m. The proposed invert elevation is about 215.0 m at the west tunnel limit with the sewer pipes sloping downward at 0.3 per cent towards the east. Overburden ground cover ranges from 4 m to 4.5 m beneath the existing highway limits, to as little as 2 m just beyond the east shaft location.

The proposed sewer alignment and shaft locations are shown on Figure 2.

### **3.0 INVESTIGATION PROCEDURES**

#### **3.1 Pre-existing Information**

Two geotechnical investigations were carried out by others in the vicinity of the subject area in 1998. A pavement design report for the proposed Highway 400 widening was issued in August 1998 by Strata Engineering Corporation (W.P. 475-91-00) following a subsurface investigation from Langstaff Road to just south of Major Mackenzie Drive. The scope of this investigation focused primarily on the existing Highway 400 roadway structure, with shallow boreholes (less than 4 m deep) put down within the existing right-of-way. The report for this investigation indicated the presence of a storm sewer pipe offset to the east of the median centreline of Highway 400 between Rutherford Road and Major Mackenzie Drive. The existing storm pipe is reported to be along the middle of the existing fully paved inside shoulder. The depth to the crown of the sewer pipe is about 1.6 m below the existing shoulder pavement. The Strata Engineering Corporation report recommends the removal and replacement of the existing median catch basins and sewer pipe, and manhole installations as part of the ultimate 10-lane Highway 400 corridor.

A second investigation was carried out in May 1998 and August 1998 by AGRA Earth and Environmental Ltd. (Report No. TT98-4-19, dated June 1998 and a supplementary report dated September 1998). This investigation consisted of advancing boreholes along the proposed Jane-Rutherford sanitary sewer alignment. The sanitary sewer alignment crosses Highway 400 about 10 m south of the proposed tunnel locations. The results of two boreholes (Boreholes 17A and 18) put down in this area as part of the sanitary sewer investigation have been reviewed in the current Golder Associates Ltd. investigation to assist in the interpretation of the subsurface conditions.

The above mentioned reports were provided to Golder Associates Ltd. by Schaeffer & Associates Ltd.

#### **3.2 Historical Data**

A small creek crosses under Highway 400 north of the proposed tunnel alignment (see Figure 1). Golder Associates Ltd. conducted a review of topographic maps for the area to determine the nature and alignment of the creek prior to construction of the Highway. A map from 1934

obtained from the University of Toronto map library indicates the creek was not a significant feature at that time and it appears not to have been present west of the Highway 400 alignment.

### **3.3 Golder Associates Ltd. Investigation**

The fieldwork for this investigation was carried out between December 17 and 21, 1998 at which time three boreholes were advanced at the site as shown in Figure 2. Boreholes 1 and 2 were staked in the field by Schaeffer & Associates Ltd. personnel and correspond to the proposed shaft locations. Borehole 3, located near the Highway 400 centreline, was marked in the field at approximately centreline of the twin tunnels. Borehole 3 was located by Golder Associates Ltd. personnel using Boreholes 1 and 2 as reference for alignment. The ground surface elevations at the borehole locations were obtained from a plan and profile drawing provided by Schaeffer & Associates Ltd. in digital format. The elevations are understood to be referenced to the geodetic datum.

Boreholes 1 and 2 were advanced with a bombardier-mounted CME 55 power auger rig and Borehole 3 with a truck-mounted B-57 power auger rig using 108 mm I.D. continuous flight hollow stem augers, supplied and operated by Longyear Canada Inc. The boreholes were advanced to depths between 9.8 m and 12.8 m below the existing ground surface. Soil samples were generally obtained at 0.75 m and 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure.

The groundwater conditions in the open boreholes were observed during and following completion of drilling. Two 25 mm I.D. piezometers were sealed in each borehole; one at the base and one intersecting the silt interlayer encountered in each boring. Installation details are provided on the attached Record of Borehole sheets.

The field work for this investigation was directed by a member of our engineering staff who supervised the drilling and logged the boreholes. The samples obtained were placed in labelled containers and transported to our laboratory for further examination and testing. These samples will be stored for three months from the date of completion of the fieldwork at which time they will be discarded unless other instructions are received.

## **4.0 SUBSURFACE CONDITIONS**

### **4.1 Local Geology**

The subject site is located within the Peel Plain physiographic region of southern Ontario as described by Chapman and Putnam (1984). This region covers about 800 km<sup>2</sup> extending over parts of York, Peel, and Halton Regional Municipalities. The subsoils of this region are predominately glacially derived, consisting of clayey and silty tills and granular outwash materials. Cobble and boulder sized inclusions within the till are characteristic of the Peel Plain tills.

### **4.2 Local Topography**

The topography of the subject area ranges from level to undulating. The surface elevation gently slopes south towards Lake Ontario from an Elevation of 230 m north of Major Mackenzie Drive to 220 m near Rutherford Road. The native ground surface elevation is about 221 m adjacent to the proposed tunnel alignments.

### **4.3 Soil Stratigraphy**

The subsoil and groundwater conditions encountered in each of the boreholes, together with the results of the laboratory testing conducted on representative samples, are given on the attached Record of Borehole sheets and Figures 4, 5, 6 and 7. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact planes of geological change. Further, conditions will vary between and beyond the borehole locations.

#### **4.3.1 General**

The interpreted stratigraphy along the centreline of the twin tunnel alignment is illustrated on Figure 3. The boundaries illustrated on Figure 3 represent the baseline condition for tendering purposes. However, the investigation was limited because of highway access restraints; since the most significant property of glacial deposits is their inherent variability; variations in presence and locations of granular inclusions and variations in the deposit boundaries from those illustrated must be anticipated along and between the section line. Therefore, construction equipment and procedures must be selected to accommodate typical variations in the deposit boundaries.

The subsoils along the proposed tunnel alignments generally consist of road and embankment fills overlying a Clayey Silt Till with in turn is underlain by a Sand / Silt deposit. Silt interlayering was found through the Clayey Silt Till. None of the boreholes fully penetrated the Lower Sand / Silt deposit, and bedrock was not encountered in this area.

The following sections provide a brief description of the subsoil and groundwater conditions encountered during this investigation.

#### **4.3.2 Fills**

A thin (80 mm to 100 mm thick) surficial topsoil layer was encountered in Boreholes 1 and 2. A clayey silt fill containing a trace to some sand and trace gravel underlies the topsoil. Occasional fine rootlets were noted. A Standard Penetration Test (SPT) carried out within the fill gave a "N" value of 16 blows per 0.3 m of penetration, indicating a very stiff consistency. The fill is about 0.3 m and 0.7 m thick at these borehole locations and is likely the result of previous Highway 400 construction works.

About 250 mm of asphalt underlain by 1.2 m of granular fill was encountered in Borehole 3 as part of the Highway 400 southbound median lane road structure. The granular material is underlain by a 0.8 m thick layer of sandy silt fill consisting of thin (less than 10 mm in thickness) clayey silt layers and trace gravel. Standard Penetration Testing carried out within these fills measured "N" values of 17 blows and 15 blows per 0.3 m of penetration, indicating a compact state of packing.

#### **4.3.3 Clayey Silt Till**

A Clayey Silt Till deposit was encountered in all boreholes. Silt interlayering within the Clayey Silt Till divides this deposit into upper and lower regions; the Silt Interlayer is discussed further in Section 4.3.4.

The Clayey Silt Till deposit contains trace to some sand and gravel and oxidation staining was noted in the upper deposit area. Grain size distribution curves from selected samples of this layer are shown on Figure 4. Atterberg limits testing carried out on representative samples are shown on Figure 7 and indicate the Clayey Silt Till is of low plasticity.

Standard Penetration Test "N" values were measured between 15 blows to 52 blows per 0.3 m of penetration, corresponding to a very stiff to hard consistency. The recorded "N" values for this deposit are generally lower below the Silt Interlayer; the higher "N" values above the Silt Interlayer are indications of a weathered "crust". The deposit was fully penetrated in Boreholes 1 and 2 at Elevations of 209.7 m and 211.1 m, respectively.

#### **4.3.4 Silt Interlayer**

A Silt Interlayer that separates the Clayey Silt into upper and lower regions was encountered in all boreholes. The Silt Interlayer is variable, ranging in composition from sandy silt to clayey silt.

The 0.5 m thick silt interlayer found in Borehole 1 contains some sand and traces of clay and gravel. The Silt Interlayer is thicker (1.5 m) and has a higher sand content at the Borehole 3 location. Standard Penetration testing carried out within the interlayer at this location gave "N" values of 41 blows and 97 blows per 0.3 m of penetration, indicating a dense to very dense state of packing. The interlayer is greater than 2 m thick at Borehole 2, located on the east side of Highway 400. At this location it consists of a cohesive clayey silt. Standard Penetration testing carried at within the interlayer at Borehole 2 gave "N" values ranging from 32 blows to 46 blows per 0.3 m of penetration, indicating a hard consistency. Grain size distribution curves for selected samples of the silt interlayer are shown on Figure 5.

#### **4.3.5 Lower Sand / Silt**

The Lower Sand / Silt deposit was encountered below the Clayey Silt Till in Boreholes 1 and 2. Grain size distribution curves presented in Figure 6 show the variable sand and silt contents of selected samples from this deposit. A 0.6 m thick clayey silt interlayer was found in Borehole 1 within the Lower Sand / Silt deposit. Standard Penetration Test "N" values were recorded between 14 blows to 41 blows per 0.3 m of penetration, indicating a compact to dense state of packing.

#### **4.3.6 Groundwater Conditions**

Observations of groundwater conditions encountered during drilling and monitoring of the piezometers for each of the boreholes are recorded on the attached Record of Borehole sheets. Groundwater was measured in the piezometer installed in Borehole 17A at Elevation 215.3 m on

January 8, 1999. Borehole 18 could not be located in the field at the time of this investigation; the water level shown on the stratigraphic plot for this borehole represents the May 27, 1998 measurement by AGRA Earth and Environmental Ltd.

A main water table at depth and a perched water table above characterize groundwater conditions along the proposed sewer tunnel alignment. The lower water table is associated with the Lower Sand / Silt Deposit, while the perched water table exists within the Silt Interlayer. The groundwater level in the Lower Sand / Silt Deposit was measured at Elevation 215.4 m, and is located in the lower half of the proposed tunnel horizon. The upper perched groundwater condition was measured between Elevations 217.3 m and 218.4 m, or about 2.3 m to 3.4 m above the proposed tunnel invert.

Groundwater conditions are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

## 5.0 GEOTECHNICAL ENGINEERING DESIGN RECOMMENDATIONS

### 5.1 Pipe Design Loads

As discussed in Section 6, it is anticipated that precast concrete pipe segments will be jacked under the highway to form the proposed sewer. Given the relatively shallow cover depth of the proposed tunnels, it is expected that jacking forces will govern the selection and design of the pipe. However, the pipe design should be checked for the permanent soil and surcharge load condition and if a cast-in-place concrete pipe alternative is considered, its design should be based on the permanent load condition provided below.

A jacked concrete pipe or cast-in-place concrete pipe can be assumed to behave as a rigid pipe for design purposes and should be designed to resist the vertical stress at the tunnel crown, given by:

$$p_v = \gamma H + q \text{ (kPa)}$$

and the horizontal stress at the tunnel spring line given by:

$$p_h = (p_v + \frac{\gamma d}{2} + q) K \text{ (kPa)}$$

where	q	is surcharge load at depth (kPa)
	$\gamma$	is the unit weight of soil = 21 kN/m <sup>3</sup>
	H	is amount of soil cover above the pipe (m)
	d	is the pipe diameter (m)
	K	is earth pressure coefficient = 0.3 to 0.5

The horizontal earth pressure coefficient is dependent on a number factors, including the timing and completeness of grout placement between the pipe and soil. As such, the pipe design should be checked for the range of earth pressure coefficients (K) provided above. The surcharge load at depth may be calculated from point or line load highway vehicle loadings in accordance with the following relationships:



For point loads:

$$q = Q_p/d^2$$

For line loads:

$$q = Q_l/d$$

where  $Q_p$  is point load (kN)  
 $Q_l$  is line load (kN/m)  
 $d$  is depth below ground surface

## 5.2 Temporary Works

### 5.2.1 Excavation Support

Requirements for construction excavations are included in Ontario Regulation 213/91 under the Occupational Health and Safety Act (OHSA). For the purposes of interpreting the OHSA for excavations which are less than 6 m deep, subsurface conditions at the site, which include surficial fill and a saturated silt interlayer, are classified as Type 3 soils.

Support for excavation, including the headwalls for the tunnel entry and exit shaft should be designed to resist the earth and surcharge load distribution provided by Figure 8, with

Under Weight of Soil,  $\gamma = 21 \text{ kN/m}^3$  and  
Earth Pressure Coefficient,  $K = 0.25$

The distribution provided on Figure 8 should be applied for both the design of the vertical support members and the calculation of horizontal restraint loads.

### 5.2.2 Thrust Block Resistance

It is anticipated that the launch shaft for pipe-jacking will be located on the east side of Highway 400, allowing jacking and mining to proceed "up-gradient". It is anticipated, therefore, that thrust blocks constructed at the rear of the launch shaft will bear against the Clayey Silt Till and the cohesive clayey silt portion of the Silt Interlayer.

The thrust block may be sized in accordance with the passive resistance pressure calculated from the following equation:

$$p_p = \alpha S_u$$

where  $p_p$  is the passive pressure in kPa  
 $S_u$  is the undrained shear strength of the soil which can be assumed to be 150 kPa for thrust block design purposes  
 $\alpha$  is a factor that depends on the ratio of thrust block height to depth of the bottom of the thrust block below ground surface (h/H)

Typical values for  $\alpha$  are provided below

$\alpha$	h/H
2	1
3	.5
4	.33
5	.25

The passive pressure derived above should be reduced by a Factor of Safety of 2 for calculation of allowable jacking stresses.

### 5.2.3 Groundwater Control

The base of the excavation for the proposed exit shafts on the west side of Highway 400 will be near the interface between the Clayey Silt Till and the Lower Sand / Silt. The groundwater level in the Lower Sand / Silt at this location is about 1 m above the tunnel invert and would be 1 m to 2 m above the base of the launch shaft excavation. It is expected that dewatering of the Lower Sand / Silt will be required to maintain a stable excavation base. Dewatering works would have to be installed and operating prior to excavation below the groundwater level of about Elevation 216 m.

Given the depth of the Lower Sand / Silt and the relatively fine grained nature of this deposit, it is expected that an eductor well system or a vacuum well point system will be required to achieve the necessary drawdown. If a vacuum well point system is used, it would have to be installed in a

trench surrounding the excavation, sized such that the "lift" between the tip of the well point and the header pipe and pump assembly did not exceed about 5 m.

The Lower Sand / Silt displays a relatively broad range in gradation, with the anticipated permeability of the deposit ranging from  $1 \times 10^{-8}$  m/s to  $1 \times 10^{-5}$  m/s. It is expected that the lower end of the range would be used to assess required well spacings while the upper end of the range would be used to assess pumping flow rates and to size settling tanks.

Chemical analysis of groundwater quality was not undertaken as part of the site investigation; however, based on the chemical analyses of soil samples, it is expected that the dewatering discharge would be suitable for discharge as storm water, providing the water is properly filtered in the well and allowed to settle in a tank to remove suspended solids.

## 6.0 GROUND BEHAVIOUR IN RELATION TO CONSTRUCTION

The tunnelling constructor is responsible for choosing the method and equipment for tunnelling. These should be chosen to ensure that ground movements do not exceed 25 mm. The description of the anticipated ground behaviour provided in this section only applies to anticipated construction methods described herein and in the previous section. Ground behaviour will vary if methods different from those considered in the report are adopted. It should not be construed that the Contractor is restricted to the particular methods considered herein, although in the event of alternative methods, the Contractor must make his own interpretation of the anticipated ground behaviour, based on the factual information provided herein.

Descriptions of anticipated soil behaviour such as "ravelling" ground are based on Terzaghi's<sup>1</sup> classifications of soils for tunnelling as referenced below.

### 6.1 Tunnelling

#### 6.1.1 Anticipated Methods

It is feasible to construct the tunnel using either pipe-jacking methods or more traditional two-pass methods incorporating a primary liner of ribs and lagging, followed by the installation of a cast-in-lace permanent liner or a grouted in place pre-cast concrete liner. Face stability is a key to controlling ground movements for both methods and is discussed in more detail in Section 6.1.2.

Pipe-jacking provides for the installation of the rigid and impervious ground support immediately behind the tunnelling shield, if side friction is reduced by suitable lubrication of the annulus between the pipe and the ground left by the shield tail void this can be a satisfactory method of installation. The alternative two-pass system will require the added installation steps of ring expansion and the provision of a filtering medium behind the lagging boards to prevent ingress of fine sands and silts from the Silt Interlayer.

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<sup>1</sup> Terzaghi, K. "Geologic Aspects of Soft Ground Tunnelling", Chapter 2 of Applied Sedimentation, P.D. Trask ed; John Wiley & Sons, New York, 1950.

It is understood that preliminary inquiries made by the Owner suggest that pipe-jacking at this site is a lower cost tunnelling alternative. Given this, this report is written with the assumption that the tunnel will be installed by pipe-jacking methods.

#### **6.1.2 Face Stability**

The overload factor for the proposed tunnel alignment in the Clayey Silt Till is less than 1, indicating that this deposit is expected to behave as firm to bouldery ground if excavated using an open-face shield. However, the Silt Interlayer is water-bearing. Upon initial exposure, it is expected to be stable due to dilation in response to stress relief. However, if left unsupported, the material will behave as slow ravelling ground. The Silt Interlayer is expected to be encountered over a portion of the face over the entire tunnel length. Although not encountered by the investigation, it should be assumed that the Silt Interlayer will contain localized zones of coarse material that will behave as flowing ground if exposed.

To maintain face stability and minimize ground movements it is recommended that mining operations continue non-stop once started. If it is necessary to stop tunnelling operations for any reason, the face should be completely supported by breasting boards. Such face support should be pre-cut and assembled prior to the start of tunnelling so that it can be readily installed, if required. Further, filter fabric, straw and other packing materials should be available on site to contain any localized occurrences of flowing ground.

Because of the relatively shallow ground cover and the presence of the Silt Interlayer, a "plug" of soil must be maintained within the shield at all times; excavation ahead of the shield must not occur.

#### **6.1.3 Settlement and Settlement Control**

The measures described above must be implemented to control settlement above the tunnels; however, the effects of stress relief at the tunnel face and partial closure of the over-cut between shield and pipe will result in some settlement of the ground and overlying highway. It is anticipated that ground surface settlement can be maintained at 25 mm or less, if:

- the measures to control face stability as described in Section 6.1.2 are implemented.

- the over-cut between the tunnelling shield and the pipe is 12 mm or less.
- suitable lubricant is applied directly behind the shield to minimize friction between the pipe and the ground.
- the gap created between the soil and the pipe is grouted with cement grout at the completion of jacking.

To achieve appropriate grouting, it is recommended that grout ports around the circumference of the pipe be not further than 2 m apart.

To verify that tunnelling movements are maintained within specified levels, to verify the suitability of the Contractor's tunnelling methods and to provide an early warning that will allow tunnelling methods to be modified to meet the specified settlement limits, it is essential that a settlement monitoring program as described in Section 7.0 be implemented during tunnelling.

#### **6.1.4 Jacking Resistance**

In addition to the subsurface conditions and pipe geometry, the jacking forces required to advance the pipe are dependent upon a number of factors directly related to construction equipment and methodology, including:

- the size of the shield over-cut;
- the use of lubricants and the timing of lubricant injection;
- the alignment maintained during jacking;
- the rate of mining achieved; and
- the frequency and duration of stoppages in the work.

For these reasons and considering the natural variability of the ground, it is not possible to predict actual jacking forces prior to construction. However, assuming that lubrication will be injected through the pipe throughout the jacking operation and that mining will be carried out on a continuous basis, it is expected that the unit jacking resistance on the surface area of the pipe will be in the range of 5 kPa to 15 kPa. Resistance at the shield face is expected to be within the range of 500 kN to 1,500 kN, assuming that the shield steel is less than 25 mm thick. Higher shield

face resistance will be met where boulders are encountered. In this event the boulder will have to be removed by careful excavation at the face.

Given the uncertainties in predicting jacking forces and the limitations on jacking forces imposed by the pipe strength, it is recommended that provision be made for intermediate jacking stations along the pipe length. Jacking forces should be monitored throughout the tunnelling operation to ensure that allowable pipe stresses are not exceeded and to determine if jacking from the intermediate jacking stations is necessary.

### **6.1.5 Obstructions**

The soils at the site are glacially derived, and thus, are anticipated to contain boulders (pieces of rock unable to pass through a 0.3 m square opening). Although the drilling investigation program was carried out without encountering instances of "hard" drilling that would suggest the presence of boulders, a large pile of boulders was observed just west of the site. It is assumed that these boulders were removed from excavations made during construction work in the adjacent development.

It is, therefore, anticipated that boulders will be encountered during excavation work for the tunnels and access shafts. Our experience with subway construction work in Toronto suggests that boulders in the till could typically comprise about 0.2 per cent to 1 per cent of the in-place soil volume. For baseline purposes on this project, 0.5 per cent boulders by volume can be assumed. It is anticipated that boulders will not be greater than 3 m in size and that the size distribution will be such that an average of about 9 boulders will have a combined volume of 1 m<sup>3</sup>. Therefore, for the proposed twin 70 m long, 3 m diameter tunnels, a total of 45 boulders can be assumed for baseline purposes.

### **6.2 Pile and Wall Installation**

It is assumed that construction of the headwalls for the access shafts will involve drilling of augered holes for soldier pile installation or interlocking concrete tangent pile wall construction. The augered holes will penetrate through the Fill, Clayey Silt Till, Silt interlayer and on the west side of Highway 400, are expected to terminate in the Lower Sand / Silt Deposit.

The Lower Sand / Silt Deposit and portions of the Silt Interlayer are cohesionless and are expected to flow and ravel into the augered hole if it is unsupported. Therefore, the use of casing during pile installation is considered necessary. Piles drilled on the west side of Highway 400 are expected to terminate in the Lower Sand / Silt, which is saturated and under groundwater pressure. Installation and operation of the required dewatering works such that the groundwater level is drawn down below the pile base prior to drilling will stabilize the hole bottom. Otherwise, it is expected that it will be necessary to utilize a column of water or drilling mud within the casing to maintain base stability within the Lower Sand / Silt or it will be necessary to extend the cased pile hole into the Clayey Silt Interlayer that is present well below the proposed excavation base.

Boulders are expected to be encountered during pile drilling and the baseline number can be determined on an excavated volume basis as described in Section 6.1.5. Cobbles and boulders should also be anticipated during drilling of dewatering wells or drilling for instrumentation installation at the site. Because of scale effects, the probability of encountering a cobble or a boulder in such a small diameter hole is greater on a volume basis than for tunnelling or pile installations. For baseline purposes it can be assumed that 13 cobble or boulder obstructions will be encountered for every 100 m of drilling.

### **6.3 Excavation**

#### **6.3.1 Open-Cut Slopes**

Open-cut slopes could be used on the west side of Highway 400 to retrieve the jacking shields at the end of tunnelling. The open-cut would advance through the surficial Fill, the Clayey Silt Till and the Silt Interlayer. Such cut slopes should be formed at a gradient no steeper than 1 horizontal to 1 vertical. Seepage from the Silt Interlayer is expected to cause it to ravel into the excavation; this loss can be controlled by constructing a filtered drain along the exposed side slopes, at the base of the Silt Interlayer. The soils exposed in an open-cut excavation at the site will be susceptible to erosion from surface run-off. Erosion can be reduced by diverting surface water away from the excavation side slopes. If construction proceeds in the winter, the slope surface can be expected to freeze and sloughing and slumping of the slope surface must be anticipated during the freeze-thaw cycles.



### **6.3.2 Sheathing Lagging Installation**

A vertical headwall will be required in the jacking pit. Directional control of the pipe jacking operation may require a collar to be constructed as part of this headwall. The headwall may consist of soldier piles and lagging or timber sheathing.

The headwall will be installed to support the surficial Fill, the Clayey Silt Till and the Silt Interlayer. These soils are expected to be stable upon initial exposure; however, in the case of soldier pile of lagging, lagging lift heights should be limited to a maximum of 1.2 m and lagging should be installed promptly to limit raveling of the Fill and Silt Interlayer and desiccation and sloughing of the Clayey Silt Till. If timber sheathing is used, similar precautions regarding exposed vertical faces of soil need to be taken and any loss of ground behind the sheathing should be replaced with compacted backfill.

When lagging / sheathing is installed adjacent to the Silt Interlayer it should be backed by a non-woven geotextile filter fabric, with the void behind the lagging packed with a granular soil that will minimize migration of fine soil particles.

### **6.3.3 Trench Box Installation**

Where a trench box (with expandable struts) is used to support the excavation sides it is expected that the site soils will stand unsupported for a limited time period, requiring prompt installation of the trench box support. The Silt Interlayer is expected to ravel after initial exposure, which if left unsupported would undermine the overlying Clayey Silt Till, causing chunks of it to fall into the excavation. Soil removed from the excavation should be cast away from edge of the excavation by a distance at least equal to the excavation depth to minimize surcharges on the unsupported excavation sides. Stockpiling of material immediately adjacent to the excavation will increase the risk that the sides will slump in prior to trench box installation.

### **6.4 Surplus Soils**

The management and disposal of soil is governed by a number of provincial and municipal regulations. It is expected that during the tender period, bidders will use the available chemical test data to determine soil disposal sites, in consultation with site operators. No statement made

herein should be construed as relieving the Contractor of his duty to comply with all applicable regulations to the disposal of soil.

Three soil samples were submitted for chemical analysis. The results are presented in Appendix A, and have been compared to the MOE "Criteria for the Management of Inert Fill" (1998). The soil samples tested meet the criteria for Class I inert fill.

## 7.0 INSTRUMENTATION AND MONITORING

An instrumentation program is necessary on this project to:

- document the effects of tunnelling on the overlying Highway;
- obtain prior warning of ground movements that could occur due to the construction methods and equipment or unforeseen ground conditions;
- verify the Contractors compliance with the settlement limits imposed in the Contract; and
- allow adjustments to be made to the tunnelling methods such that the settlement limits established are not exceeded.

Monitoring of settlement instruments on this project is constrained by the continuous and high traffic volume and the limited periods during which access to the highway can be obtained. Therefore, a monitoring program consisting of in-ground settlement points over each tunnel at the median and at other locations adjacent to the highway, and surface settlement points on the highway is proposed and is illustrated by Figure 2. The in-ground settlement points would consist of a sleeved iron bar, set 1.2 m to 1.5 m below ground surface. The elevation of the top of the bar would be read using conventional precision levelling equipment. The in-ground monitoring points provide the best measure of the ground settlement affects of tunnelling, as they are unaffected by frost heave, thaw settlement or the bridging action of the pavement structure.

By necessity, settlement points on the road must be read remotely and the use of EDM equipment reading reflectors installed on the highway has been suggested. Positioning of the equipment to read the instruments at this site is constrained by the elevated pavement surface relative to the surrounding ground. It is expected that an elevated platform will have to be constructed to allow the reflective targets to be read. It is understood that a specialist surveying firm will be retained to confirm the set-up and to carry out the settlement monitoring during construction; their equipment and procedures must be capable of surveying the settlement point elevation to within  $\pm 1$  mm of the actual elevation.

All monitoring points should be read at least twice before the start of tunnelling to establish a pre-construction baseline. All points behind the tunnel face and those within 10 m of the front of the face should be read every 4 hours over the duration of the tunnel drives.

**GOLDER ASSOCIATES LTD.**



Dan K. Breeze, B.Sc.



John Westland, P.Eng.  
Associate

DKB/JW/JRB/dkb/clg  
WORD S/FINALDAT/1100/981-1179/1999/81179AR1

## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I GENERAL

$\pi$	= 3.1416
$\ln x$	natural logarithm of x
$\log_{10} x$ or $\log x$	logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

### II STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is $\rho$ . Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

#### (a) Index Properties (con't.)

w	water content
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	plasticity Index = $(w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_p) / I_p$
$I_C$	consistency index = $(w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (c) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (d) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (overconsolidated range)
$C_s$	swelling index
$C_\alpha$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

Notes: 1.  $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

## LIST OF ABBREVIATIONS

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

### I SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

#### Dynamic Penetration Resistance; $N_6$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils

Consistency	$C_u, S_u$ kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
w <sub>p</sub>	plastic limit
w <sub>l</sub>	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
D <sub>R</sub>	relative density (specific gravity, G <sub>s</sub> )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

PROJECT: 981-1179

## RECORD OF BOREHOLE 1

SHEET 1 OF 2

LOCATION: REFER TO FIGURE 2

BORING DATE: DEC.17/98

DATUM:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV.	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	nat V - + Q - ● rem V - ⊕ U - ○			WATER CONTENT, PERCENT Wp — W — Wi 10 20 30 40
				DEPTH (m)								
0	CME 55 BOMBARDIER 108mm I.D. HOLLOW STEM AUGERS	GROUND SURFACE		220.70								
		TOPSOIL		0.00								
		Very stiff, dark brown to mottled brown and grey clayey silt, some sand, trace gravel, occ. fine rootlets, occ. orange oxidation staining. (FILL)		0.10	1	50 DO	18				CONCRETE	
				220.01							BENTONITE SEAL	
1			0.69	2	50 DO	16						
		Very stiff to hard, brown and grey CLAYEY SILT, trace to some sand, trace gravel, occ. orange oxidation staining. (TILL)									BACKFILL	
2					3	50 DO	15					
		Clayey silt seam/layer noted at 1.52m depth (100mm thick)										
						4	50 DO	38				
3				217.73								
		Dense, brown SILT, some sand, trace clay and gravel.		2.97	5	50 DO	44				MH	
			217.19									
4		Very stiff to hard, grey CLAYEY SILT, some sand and gravel, occ. orange oxidation staining at 3.51m depth. (TILL)	3.51							SAND		
					6	50 DO	46					
5					7	50 DO	19				MH	
					8	50 DO	16					
6				9	50 DO	20				BENTONITE SEAL		
			213.92									
7			6.78	10	50 DO	14				MH		
8		Compact to dense, grey sandy SILT, trace clay and gravel.		11	50 DO	21				CAVED		
9		Clayey silt seam/layer noted at 6.25m depth (25mm-50mm thick)										
					12	50 DO	32				SAND	
10		CONTINUED ON NEXT PAGE										

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: DKB

CHECKED: JW

N1179001 BHS

DATA INPUT: PS JAN.18/99

SOL146





PROJECT: 981-1179

## RECORD OF BOREHOLE 2

SHEET 1 OF 2

LOCATION: REFER TO FIGURE 2

BORING DATE: DEC.21/98

DATUM:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



N1179002 BHS

DATA INPUT: PS JAN 18/99

SOIL M6

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	nat V - + Q - ● rem V - ⊕ U - ○	WATER CONTENT, PERCENT Wp  -----  W  -----  Wl 10 20 30 40		
0	CME 55 BOMBARDIER 108mm I.D. HOLLOW STEM AUGERS	GROUND SURFACE		219.50							
		TOPSOIL		0.08							
		Soft, brown clayey silt, trace to some sand, trace gravel, occ. fine rootlets. (FILL)		219.20	1	50 DO	2				
				0.30							
1					2	50 DO	27				
		Stiff to hard, brown to grey CLAYEY SILT, trace sand and gravel, occ. orange oxidation staining. (TILL)			3	50 DO	32				
2					4	50 DO	22				
					5	50 DO	32				
3					6	50 DO	46				
		Hard, grey CLAYEY SILT, trace sand.  Silt seam/layer noted at 4.88m depth (50mm-100mm thick)			7	50 DO	35				
4					8	50 DO	32				
				9	50 DO	27					
5				10	50 DO	24					
	Very stiff to hard, grey CLAYEY SILT, trace sand and gravel. (TILL)			11	50 DO	26					
6											
7											
8											
	Compact to dense silty SAND to SILT and SAND.										
9											
10		CONTINUED ON NEXT PAGE									

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: DKB

CHECKED: JW

PROJECT: 981-1179

## RECORD OF BOREHOLE 2

SHEET 2 OF 2

LOCATION: REFER TO FIGURE 2

BORING DATE: DEC.21/98

DATUM:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	nat V - + Q - ● rem V - ⊗ U - ○	WATER CONTENT, PERCENT Wp — W — Wi		
10	CME 55 BOMBARDIER 100mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
11		Compact to dense silty SAND to SAND and SILT.		12	SO	41					
		END OF BOREHOLE	208.22 11.28								
12											
13											
14											
15											
16											
17											
18											
19											
20											

Note:  
1. Water level measured in open borehole at 9.12m depth (Elev.210.37m) upon completion of drilling.  
2. Water level measured in deep piezometer at 4.03m depth (Elev.215.4m) and in shallow piezometer at 1.49m depth (Elev.218.0m) on Jan.8/99.

DEPTH SCALE

1 to 50

LOGGED: DKB

CHECKED: JW

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PROJECT: 981-1179

## RECORD OF BOREHOLE 3

SHEET 1 OF 2

LOCATION: REFER TO FIGURE 2

BORING DATE: DEC.18/98

DATUM:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION			
		DESCRIPTION	STRATA PLOT	ELEV.	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	nat V - + rem V - ⊗			Q - ● U - ○	WATER CONTENT, PERCENT Wp — W — Wl	
				DEPTH (m)										
0	D-57 TRUCK MOUNT 108mm I.D. HOLLOW STEM AUGERS	GROUND SURFACE		222.10								CONCRETE		
		ASPHALT		0.00									BACKFILL	
				221.85										
1			Compact, brown sand, trace to some gravel, trace silt. (Granular FILL)			1	50 DO	17					BENTONITE SEAL	
					220.65									
					1.45									
2			Compact, brown sandy silt, occ. clayey silt layers (<10mm thick), trace gravel. (FILL)			2	50 DO	15						
					219.89									
					2.21									
3						3	50 DO	26						
						4	50 DO	52						
4			Very stiff to hard, brown to grey CLAYEY SILT, trace to some sand, trace gravel, occ. orange oxidation staining. (TILL)			5	50 DO	22						
					6	50 DO	30							
5					7	50 DO	27							
				218.08										
6				6.02										
		Dense to very dense, grey sandy SILT, occ. clayey silt seams/ layers (<10mm thick).			8	50 DO	97							
7					9	50 DO	41							
				214.56										
				7.54										
8		Very stiff to hard, grey CLAYEY SILT, trace sand and gravel, occ. sand seams (<5mm thick). (TILL)			10	50 DO	30							
					11	50 DO	29							
9					12	50 DO	32							
				212.35										
		END OF BOREHOLE		9.75										
10		CONTINUED ON NEXT PAGE												

DEPTH SCALE

1 to 50

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SOILM6

PROJECT: 981-1179

## RECORD OF BOREHOLE 3

SHEET 2 OF 2

LOCATION: REFER TO FIGURE 2

BORING DATE: DEC.18/98

DATUM:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLAT	ELEV. DEPTH (m)	NUMBER TYPE BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	nat V - + Q - ● rem V - ⊕ U - ○	WATER CONTENT, PERCENT Wp ——— W ——— Wl 10 20 30 40			
10		CONTINUED FROM PREVIOUS PAGE									Notes: 1. Open borehole dry upon completion of drilling. 2. Water level measured in deep piezometer at 5.6m depth (Elev.216.5m) and in shallow piezometer at 5.6m depth (Elev.216.5m) on Jan.8/99.
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: DKB

CHECKED: JW

N1179003 BHS

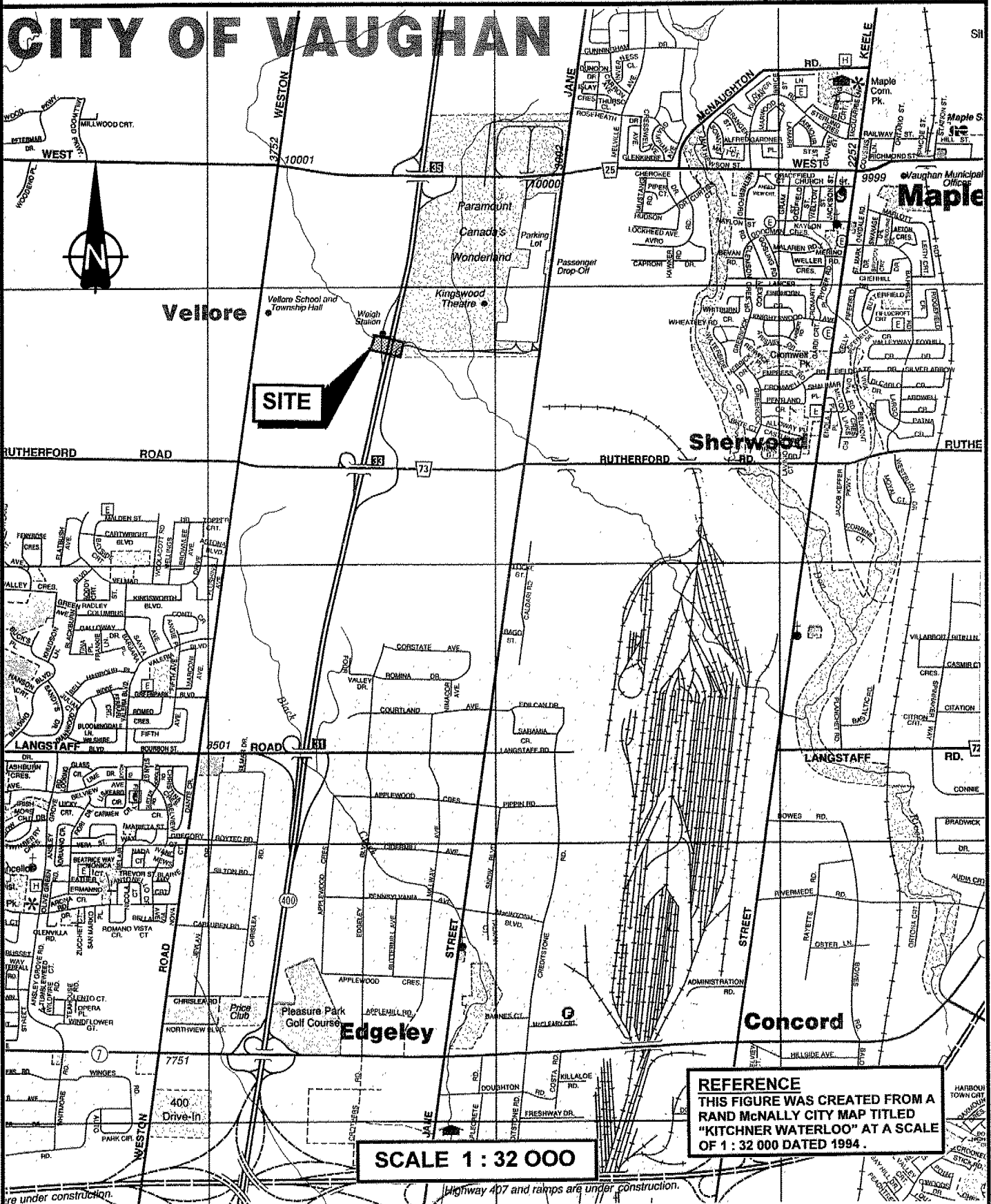
DATA INPUT: FS JAN.18/99

SOIL M6

# SITE LOCATION MAP

FIGURE 1

## CITY OF VAUGHAN



Date JANUARY, 1999.

Project 981-1179

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Drawn R.B.C.

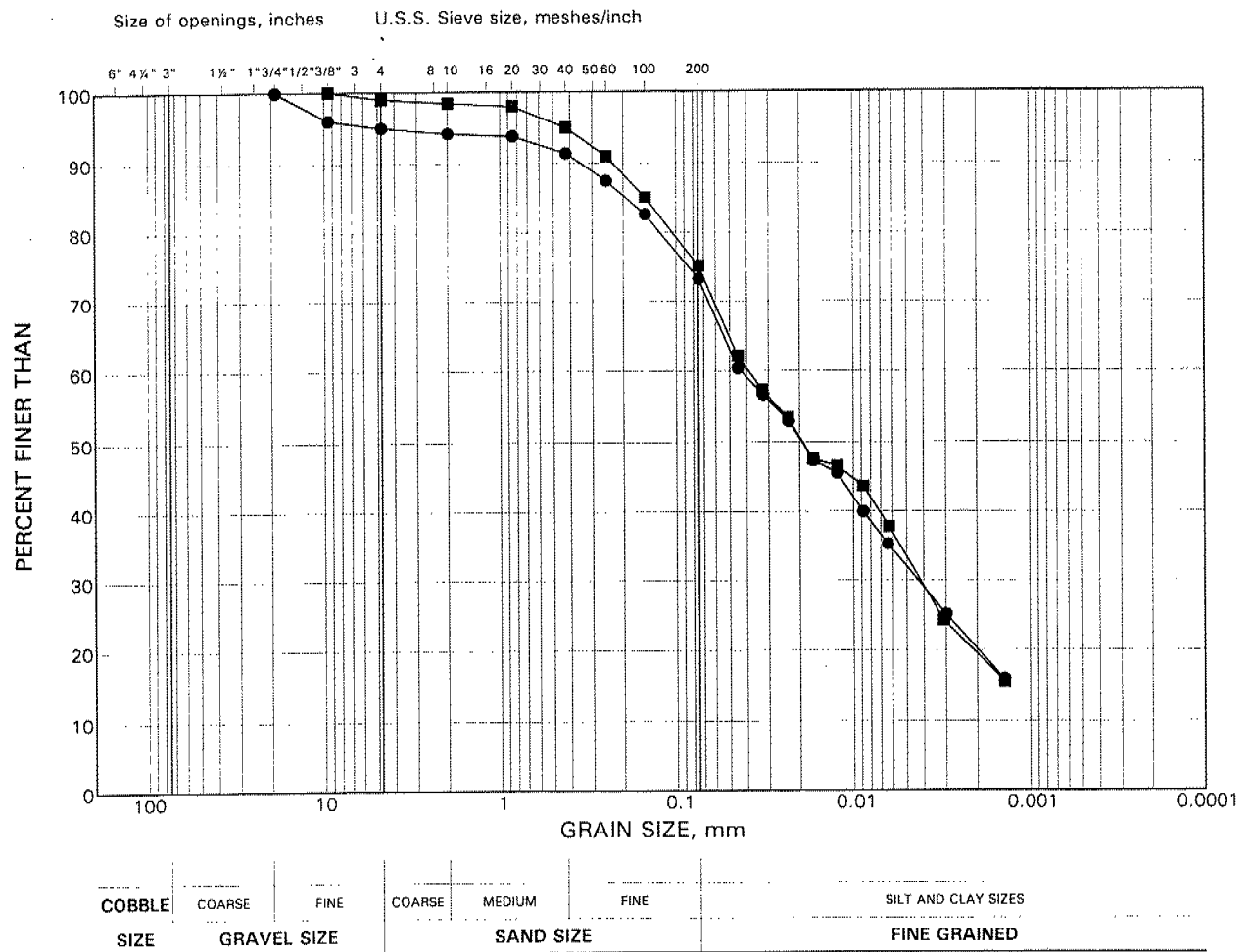
Chkd. P.K.B.

OVERSIZE  
DRAWING(S)

# GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE 4

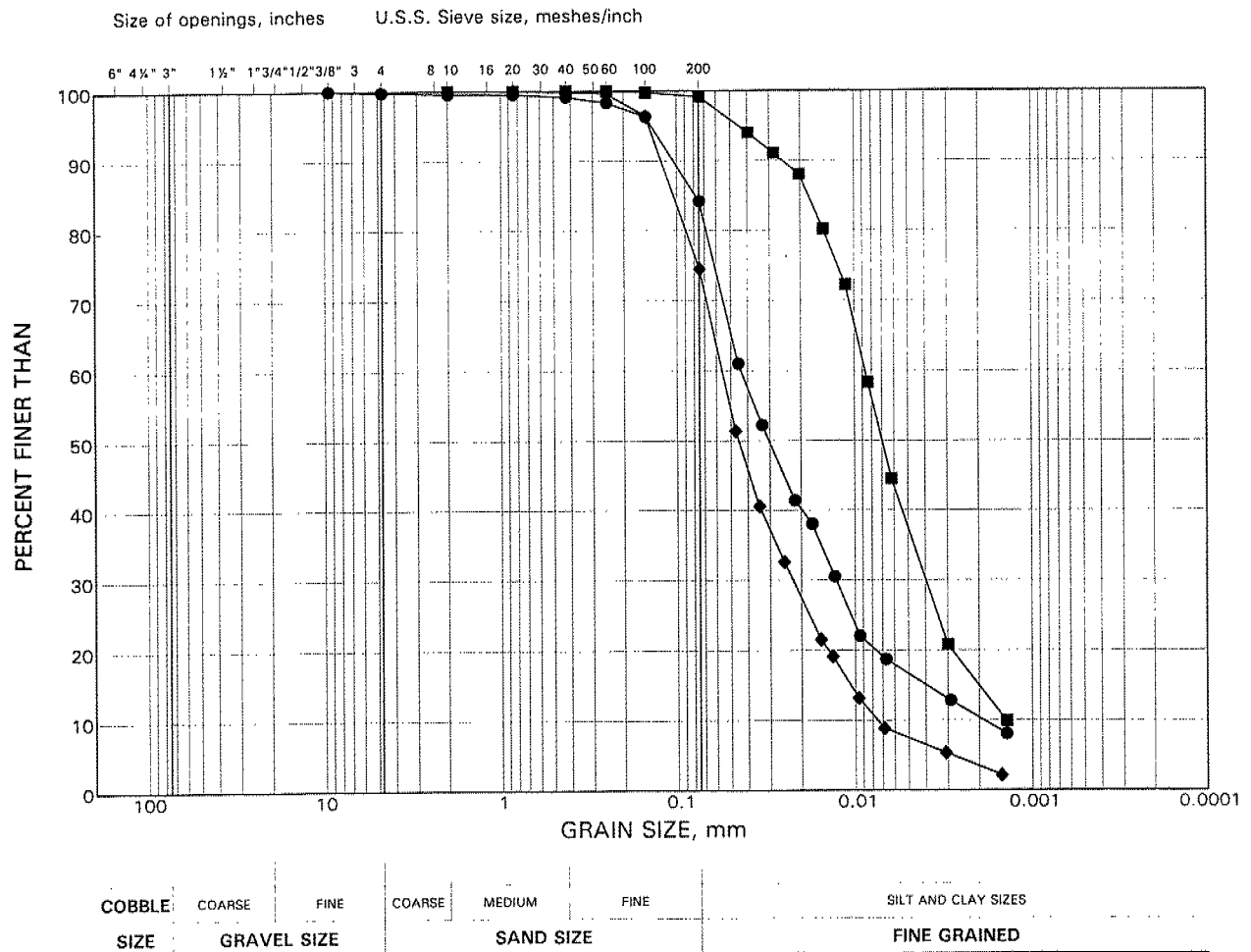


## LEGEND

SYMBOL	BOREHOLE	SAMPLE ELEVATION(m)
●	1	7 215.5
■	3	7 217.0

# GRAIN SIZE DISTRIBUTION Silt Interlayer

FIGURE 5



## LEGEND

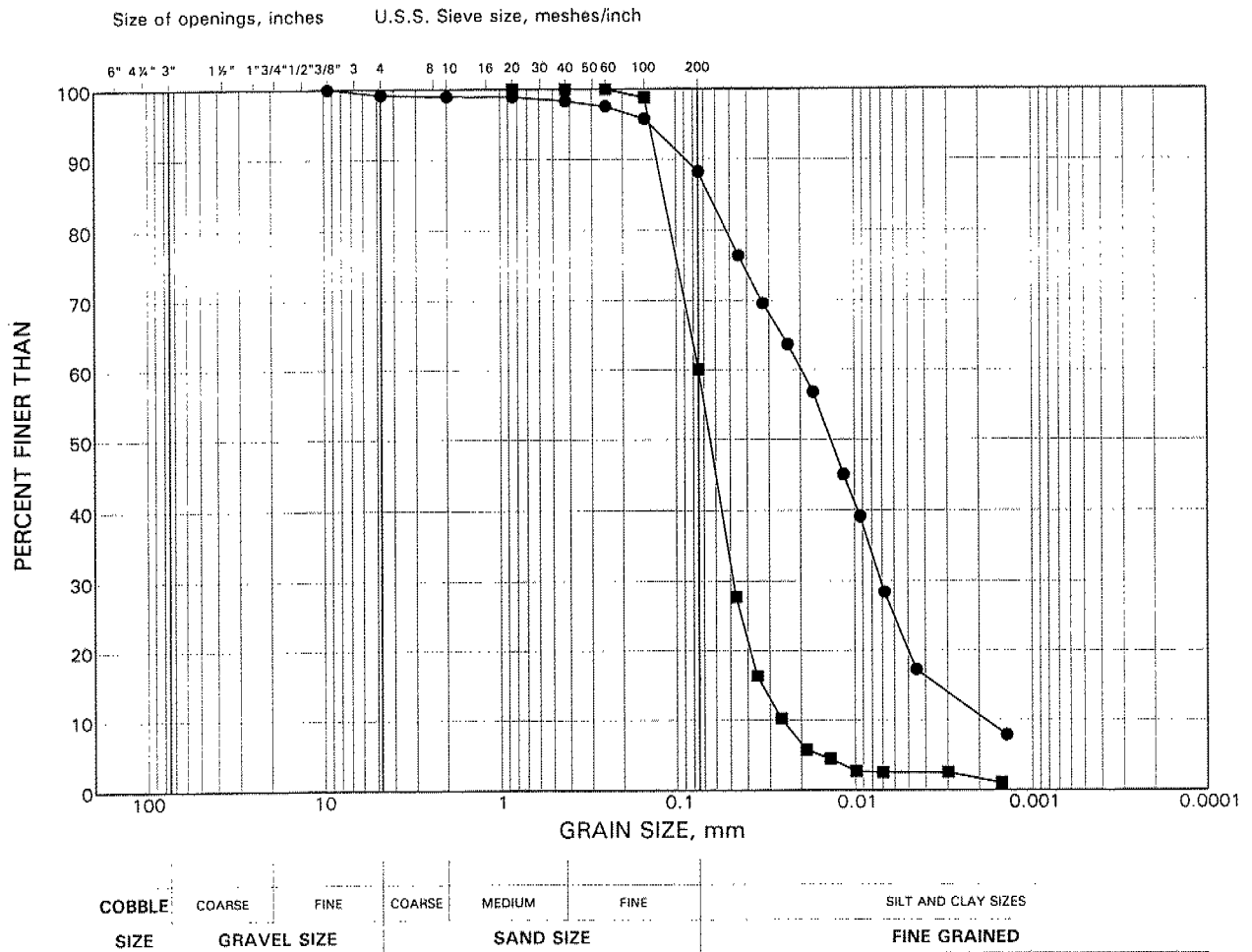
SYMBOL	BOREHOLE	SAMPLE ELEVATION(m)
●	1	5A 217.2
■	2	7 214.3
◆	3	8 216.3



# GRAIN SIZE DISTRIBUTION

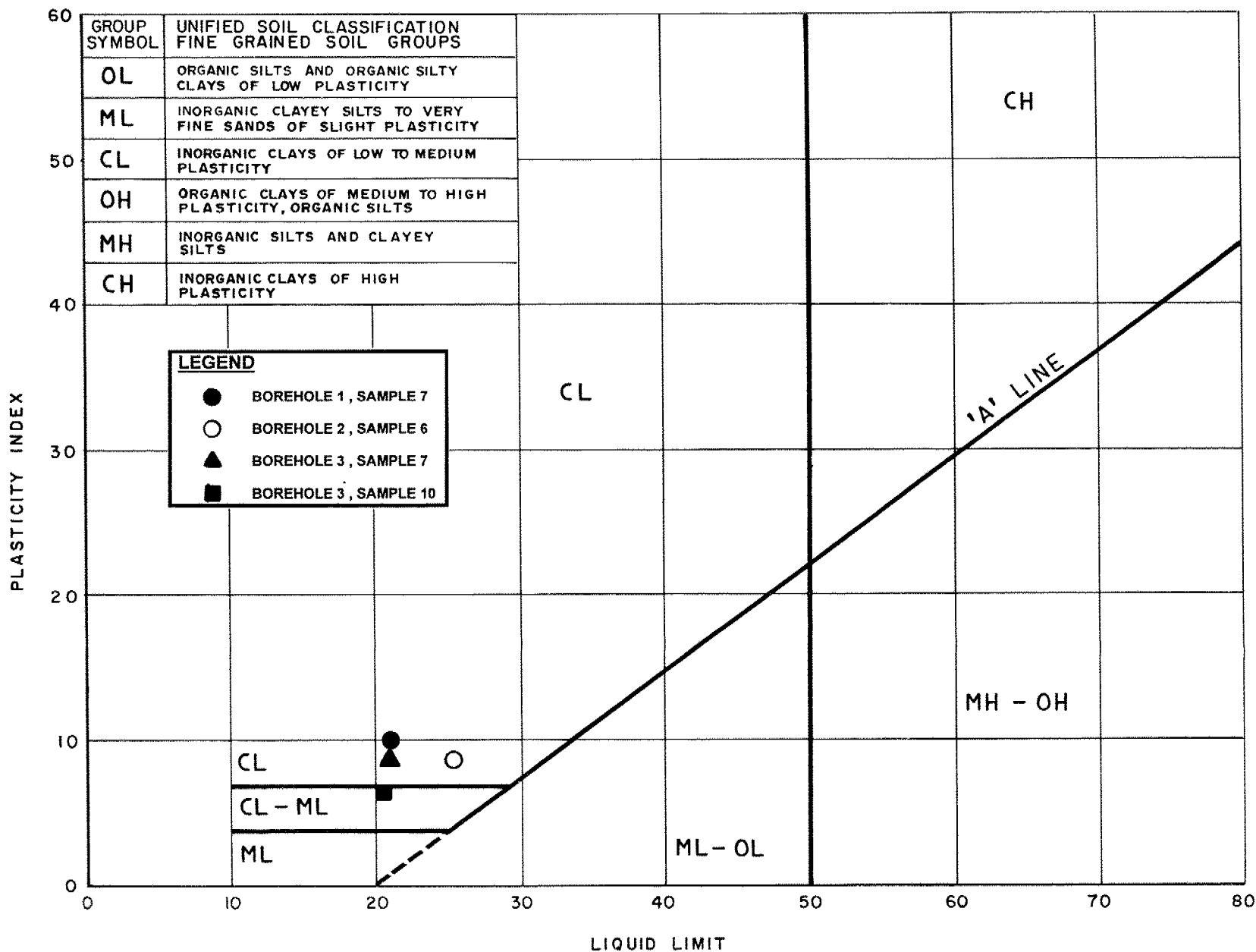
Lower Sand / Silt

FIGURE 6



# PLASTICITY CHART CLAYEY SILT

FIGURE 7



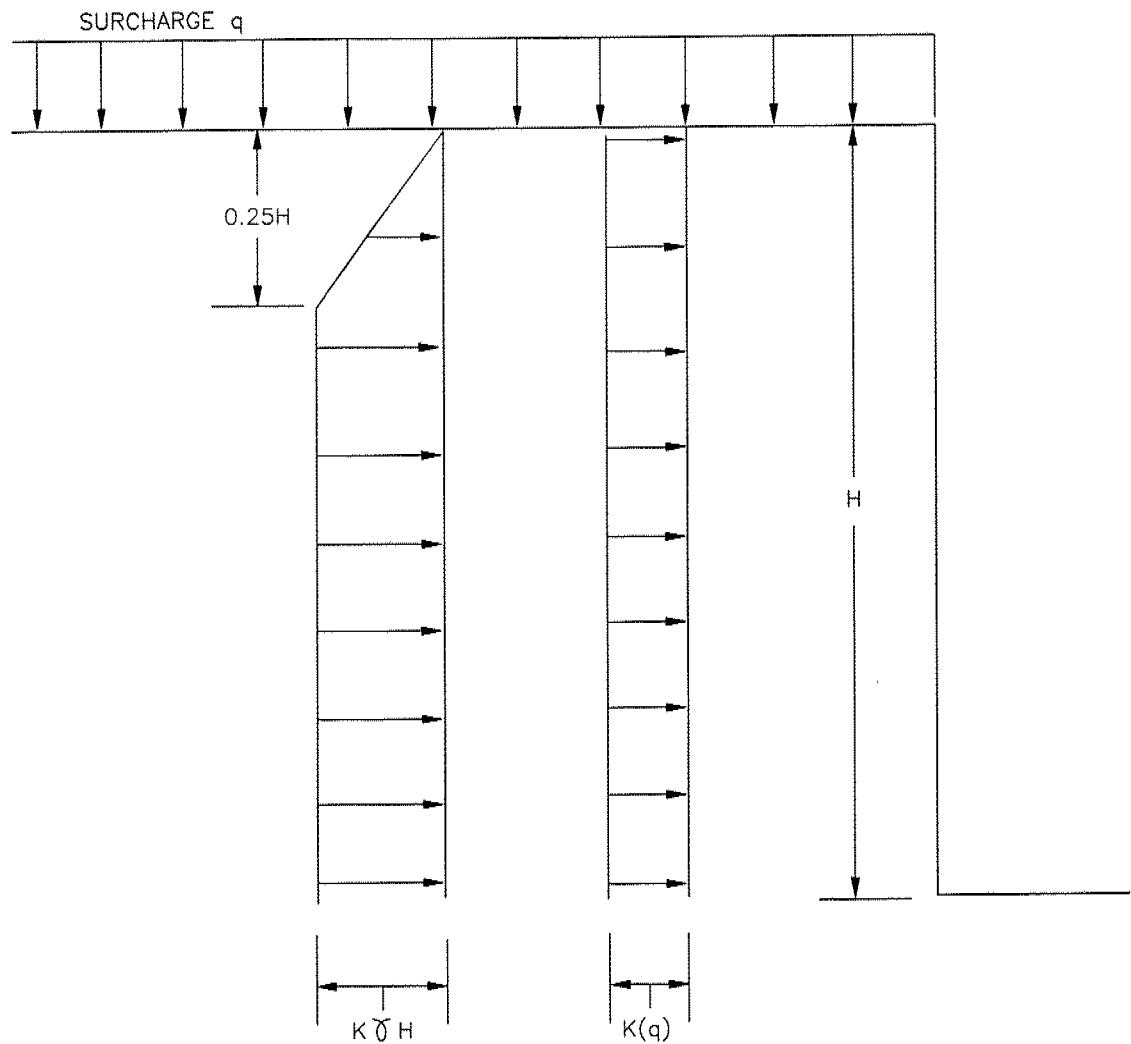
Date JANUARY, 1999.  
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Drawn R.B.C.  
Chkd D.K.B.

# DESIGN LATERAL EARTH PRESSURE DISTRIBUTION FOR BRACED EXCAVATIONS

FIGURE 8



$\gamma$  = UNIT WEIGHT OF SOIL

$K$  = EARTH PRESSURE COEFFICIENT

Date JANUARY...1999

Project 981-1179....

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Chkd ..JW.....

## **APPENDIX A**

### **CHEMICAL TEST RESULTS OF SOIL SAMPLES**



18-Jan-99

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L5N 5S3

Page: 1  
Copy: 2 of 2  
Set: 1

Attn: Dan Breele/John Westland  
Project: 981-1179

PO #:

Received: 11-Jan-99 13:55

Job: 9950123

Status: Final

			BH1 SA10	BH2 SA3	BH3 SA8	Blank	Standard (found)	Standard (expected)
Parameter			soil	soil	soil			
Antimony	SW 7041	ug/g	<0.2	<0.2	<0.2	<0.2	0.8	0.6
Arsenic	SW 7061	ug/g	1.6	1.4	3.3	<0.2	11.5	11.2
Barium	SW 6010 mod.	ug/g	27	41	47	<5	128	129
Beryllium	SW 6010 mod.	ug/g	0.2	0.2	0.4	<0.2	0.4	0.4
Cadmium	SW 6010 mod.	ug/g	<0.3	<0.3	0.3	<0.3	1.7	1.8
Chromium	SW 6010 mod.	ug/g	9	11	14	<1	18	17
Chromium (6+)	SW 7196	ug/g	<1	<1	<1	<1	0	0
Cobalt	SW 6010 mod.	ug/g	3	4	<2	<2	9	9
Copper	SW 6010 mod.	ug/g	11	14	22	<1	27	28
Free Cyanide	SM 4500I	ug/g	<0.02	<0.02	<0.02	<0.02	0.06	0.06
Lead	SW 6010 mod.	ug/g	<5	6	8	<5	85	85
Mercury	SW 7470	ug/g	0.02	0.02	0.03	<0.01	0.18	0.19
Molybdenum	SW 6010 mod.	ug/g	<3	<3	<3	<3	<3	<3
Nickel	SW 6010 mod.	ug/g	8	10	17	<2	28	27
Selenium	SW 7741	ug/g	<0.2	0.6	<0.2	<0.2	2.0	1.5
Silver	SW 6010 mod.	ug/g	<0.5	<0.5	0.6	<0.5	<0.5	<0.5
Vanadium	SW 6010 mod.	ug/g	17	17	18	<1	32	31
Zinc	SW 6010 mod.	ug/g	22	27	46	<5	155	157
pH	SM 4500B	pH Units	8.08	8.07	8.30	---	7.07	6.62
EC	SM 2510B	mS/cm	0.532	0.671	1.430	0.002	2.640	2.800
S.A.R.	Calculation	None	0.34	1.12	6.45	0.24	1.05	1.08



18-Jan-99

GOLDER ASSOCIATES LTD.  
2180 Meadowvale Boulevard  
Mississauga, ON  
L5N 5S3

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Set: 1

Attn: Dan Breele/John Westland  
Project: 981-1179

PO #:

Received: 11-Jan-99 13:55

Job: 9950123

Status: Final

Parameter	Repeat BH1 SA10
Antimony	SW 7041 ug/g <0.2
Arsenic	SW 7061 ug/g 1.5
Barium	SW 6010 mod. ug/g 28
Beryllium	SW 6010 mod. ug/g 0.2
Cadmium	SW 6010 mod. ug/g <0.3
Chromium	SW 6010 mod. ug/g 9
Chromium (6+)	SW 7196 ug/g <1
Cobalt	SW 6010 mod. ug/g 4
Copper	SW 6010 mod. ug/g 11
Free Cyanide	SM 4500I ug/g <0.02
Lead	SW 6010 mod. ug/g <5
Mercury	SW 7470 ug/g 0.01
Molybdenum	SW 6010 mod. ug/g <3
Nickel	SW 6010 mod. ug/g 8
Selenium	SW 7741 ug/g <0.2
Silver	SW 6010 mod. ug/g <0.5
Vanadium	SW 6010 mod. ug/g 16
Zinc	SW 6010 mod. ug/g 21
pH	SM 4500B pH Units 8.13
EC	SM 2510B mS/cm 0.542
S.A.R.	Calculation None 0.36



18-Jan-99

GOLDER ASSOCIATES LTD.  
2180 Meadowvale Boulevard  
Mississauga, ON  
L5N 5S3

Page: 3  
Copy: 2 of 2

Attn: Dan Breele/John Westland  
Project: 981-1179

PO #:

Received: 11-Jan-99 13:55

Job: 9950123

Status: Final

All work recorded herein has been done in accordance with normal professional standards using accepted testing methodologies and QA/QC procedures. Philip Analytical is limited in liability to the actual cost of the pertinent analyses done unless otherwise agreed upon by contractual arrangement. Your samples will be retained by PASC for a period of 30 days following reporting or as per specific contractual arrangements.

Job approved by:

Signed:

.....  
Ralph Siebert, B.Sc.  
Project Manager

**Golder Associates Ltd.**

2180 Meadowvale Boulevard  
Mississauga, Ontario, Canada L5N 5S3  
Telephone (905) 567-4444  
Fax (905) 567-6561



April 08, 1999

981-1179

Schaeffer & Associates Ltd.  
64 Jardin Drive  
Concord, Ontario  
L4K 3P3

ATTENTION: Mr. H. Tozcu, P.Eng.

**RE: RESPONSE TO MINISTRY OF TRANSPORTATION (MTO)  
REVIEW COMMENTS  
TWIN TUNNELLED STORM SEWERS  
VAUGHAN, ONTARIO**

Dear Sirs:

This letter provides our response to your facsimile transmittal of March 30, 1999 to which a copy of a letter from the Ministry of Transportation (MTO) was attached. The MTO letter provides comments on the design submission for the proposed twin tunnelled storm sewers under Highway 400 to service the Vellore Woods subdivision. We have only addressed the comments as circled on your transmittal and understand that you will address the other issues identified in the MTO letter. Our responses to the identified comments and recommended actions to address the comments are provided below:

**Geotechnical Issues ①**

Part one of this comment concerned the spacing of the tunnels which in our report were assumed to be 6 m centre-to-centre. For this configuration we undertook an assessment of potential ground surface settlement and settlement patterns. This assessment suggested that there would be minimal overlap between the settlement troughs induced by the two tunnels; this conclusion is in part a function of the somewhat limited cover above the tunnels. However, there are uncertainties in such settlement assessments and the risk of the settlement troughs overlapping significantly would be reduced by increasing the centre-to-centre spacing. Further, we understand that there are not significant cost or property implications for increasing the spacing on this contract. Therefore, it is recommended that the tunnel spacing be increased to 9 m centre-to-centre, as suggested by MTO.

In the second part of this comment the MTO recommended that the jacking operations be ceased if 15 mm of movement are experienced. We do not recommend that this provision be adopted because settlement in the range of about 5 mm to 15 mm is inevitable due to relaxation of stresses at the face of the machine and partial closure of the gap between the jacking shield and the pipe. The 15 mm settlement limit is intended to serve as a "Review Level" at which the tunnelling operations and ground conditions should be assessed. The specification presently indicates that the Engineer is to be informed when settlement reaches 15 mm and that measures be implemented to ensure that settlement does not exceed 25 mm. It is recommended that this



Schaeffer & Associates Ltd.  
Mr. H. Tozcu, P.Eng.

- 2 -

April 08, 1999  
981-1179

provision be expanded such that the Contractor be required to provide a written field report indicating the measures to be adopted to control settlement within the 25 mm maximum limit, once 15 mm of settlement is measured.

An important consideration in the actions to be taken will be the pattern of settlement relative to the face of the machine. We propose to manage and plot settlement data during construction such that the settlement pattern is updated through the tunnelling process. Clearly, if significant ground movement were occurring ahead of the face, the ground conditions and excavating practices would have to be critically examined and stopping the tunnelling operation and boarding up the face would have to be considered. However, it is possible that relatively small but progressive movement will occur behind the tunnel as the pipe is jacked and with the pipe in place, there would be very little risk of excessive and uncontrolled settlement occurring. Further, starting and stopping the tunnel operations could act to increase settlement above the pipe. Therefore, it is not recommended that the Contractor be contractually required to cease tunnelling operations if settlements of 15 mm are measured.

#### Geotechnical Issues ②

This comment requested information regarding repair to the Highway 400 pavement structure after tunnelling.

The settlement induced by tunnelling operations is likely to induce a slight dip in the Highway surface and such a dip could affect ride quality and road safety. It is recommended that after the tunnels have passed below the highway a Profilograph survey be made to measure the extent of any dip created in the pavement surface. We understand that repair would be necessary if a 10 mm deflection was measured over a 7.5 m span.

Two repair options are feasible if an excessive dip in the road is caused by the tunnelling:

- i) grinding down of the pavement at the outer edges of the dip to smooth it out; or
- ii) grinding down the pavement over the full width of the dip and providing a level overlay above the area of the dip. For this option the minimum thickness of overlay should be 40 mm.

We understand that widening of the Highway is planned in the near future. If the widening will occur shortly after tunnelling is complete, then Option (i) is considered a suitable short-term repair. If a longer-term repair is necessary, then it is recommended that Option (ii) be adopted.

**Structural Issues ①** This comment indicated an apparent discrepancy between the geotechnical report (Golder Associates Report 981-1179 dated January 1999) and the contract submission. The report assumed the tunnels were to be 6 m apart centre-to-centre and the contract submission apparently indicated a spacing of 5 m centre-to-centre. As indicated under Geotechnical Issues ① it is now understood that the tunnels will be spaced at 9 m centre-to-centre. The recommendations of the geotechnical report do not need to be altered for the proposed wider spacing.

Schaeffer & Associates Ltd.  
Mr. H. Tozcu, P.Eng.

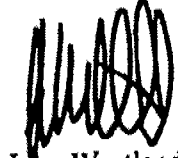
- 3 -

April 08, 1999  
981-1179

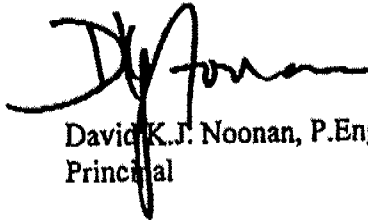
We trust that this letter adequately addresses the issues raised by the MTO review. If further information is required, please do not hesitate to contact us. We look forward to continuing to work with you on this project.

Yours truly,

GOLDER ASSOCIATES LTD.



John Westland, P.Eng.  
Associate



David K.J. Noonan, P.Eng.  
Principal

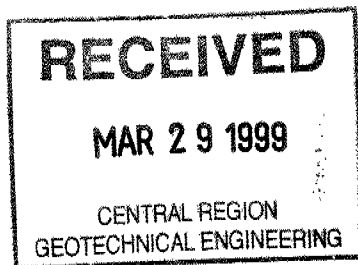
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WORD SPINAL DATE 1106/981-1179/1999/1179DL1

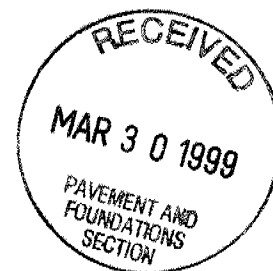
Phone: (416) 235-4269  
Fax: (416) 235-4267  
E-mail: mikolajc@mto.gov.on.ca

Central Region  
Corridor Management Office  
1st Floor, Atrium Tower  
1201 Wilson Avenue  
Downsview, Ontario  
M3M 1J8

SCHAEFFERS  
Consulting Engineer  
64 Jardin Drive  
Concord, Ontario  
L4K 3P3



March 25, 1999  
File# BLOCK 32



Attention: Hacik Tozku

**RE: Proposal for twin 2550 mm Dia. Sewer Tunnels under Hwy 400, Block 32 between  
Rutherford Rd. and Major MacKenzie Drive**

This is in response to your February, 1999 submission.  
We have completed the review and would like to offer the following comments:

**GEOTECHNICAL ISSUES:**

In March 1996 when this particular proposal was brought to our attention, we recommended that the developer carry out a detailed foundation investigation. The investigation has been conducted and recommendations provided by the consultant. Based on the foundation report and the drawings, the following comments are forwarded:

1. It is preferable that the earth cover above the tunnel crown be greater than 1 to 1.5 times the tunnel diameter and that the tunnel spacing be greater than two tunnel diameters. The close proximity of the twin tunnels implies that, for design, the tunnels are treated as a single tunnel having a equivalent diameter of approximately 3.75m. The earth cover in the vicinity of Highway 400 is in the order of 4.0m. Given these very tight constrains, our concern is that during construction, it will be difficult to ensure that settlement or heave will be less than 25mm.

The construction spec forwarded with the drawings, states that the Engineer should be notified if movements of 15 mm are experienced. If they reach 25mm the jacking operation should stop and the contractor should develop a plan to mitigate the settlement and control future settlements. The developer/contractor should have a plan in place prior to construction addressing excessive ground loss. It is recommended that the jacking operation be ceased after 15 mm of movement are experienced in order to avoid any immediate or future problems to Highway 400.

2. The issue of how damages to the pavement structure of Hwy 400 will be repaired must be addressed.
3. The foundation report assumes that the tunnels will be constructed by pipe-jacking using pre-cast concrete sections. A cross-section of the tunnels including concrete thickness should be detailed.

#### **STRUCTURAL ISSUES:**

1. The Foundations Report prepared by Golder Associates for Schaeffer and Associates states an assumption that the sewers are 6.0m apart, centre to centre. Actually they are just 5.0 m apart. The confirmation is required that the recommendations as stated in the report are still valid for the closer spacing of pipes.
2. This Ministry must be advised 3 weeks prior to construction and information requested below should be provided at that time.
3. Prior to the construction, consultant must provide details on jacking procedure, monitoring program, shoring for the jacking pits and traffic protection for Hwy 400 traffic. As a minimum elevations of Hwy 400 pavement and subsurface should be taken as per Golder's report. Existing elevations should be provided prior to any excavation and regularly therefore during construction.
4. Justification is required with regards to the need of intermediate manholes. There are no manholes shown between inlet and outlet and with the ends of the sewer being on a curve, it will be difficult to inspect the sewer.
5. If possible, we would like to be provided with the details of 825mm diameter sanitary sewer just to the south and how it will be installed?
6. The headwalls are not on MTO property however the wingwalls have a key at the front of the footing. For scour protection, 300 mm of riprap with filter cloth is indicated but no cut-off wall. We are concern that in the event of a significant storm, the invert in front of the wingwalls will scour and compromise the stability of the wall itself.

#### **HIGHWAY ENGINEERING ISSUES:**

1. Edge of proposed shaft must be outside grading limit for ultimate widening not 5.0m minimum from the edge of shoulder as shown.
2. Hwy 400 R.O.W on the profile is incorrect
3. There is no indication of a fence on Hwy 400 east property limit, if there is an existing fence, it needs to be replaced to MTO standards after the completion of construction.
4. The proposed east shaft creates disturbed earth under the ultimate Hwy 400 widening. The shaft must be located 5.0 m east of the present location (this may require temporary easement on the Canada's Wonderland property).


5. The 1.5m chain link fence located on the west property line must be constructed to MTO standards.
6. During construction operations, traffic protection should be provided for the shaft used during the boring process.

#### **DRAINAGE ISSUES:**

1. The Stormwater Management Design Report deviated from the hydrologic analysis in the preliminary report, which used the 12 hour SCS rainfall distribution to calculate the runoff hydrographs. In contrast, the Design Report used the 3 hour Chicago distribution. Unfortunately, with the information provided, we are not able to determine if the 3 hour Chicago storm is less severe than the 12 hour CSC rainfall. If this is the case, then it is likely that the flows and water levels indicated in the report are underestimated. We request that this be clarified and/or corrected as soon as possible, so that we are in a position to assess the report in this regard.
2. The Report further shows three ponds, rather than four as shown in the preliminary reports. Unfortunately, with the information provided, we are not able to determine the sequence of the hydrologic routing and by association the adequacy of the hydrologic calculations with respect to flow and water levels at Hwy 400. We therefore request that MTO be provided with a revised OTT 89 Schematic, to illustrate the hydrologic/hydraulic routing, so that we are in a position to assess the Report in this regard.
3. The proposed diversion of a part of the drainage area from Black Creek to the Don River is not the responsibility of MTO to comment on. However we suggest that TRCA may wish to fully examine all impacts of the proposed diversion, in particular, downstream riparian rights.
4. The drawing for "Plan and Profile of Valley Lowering & Culvert Improvements (P-), is extremely difficult to read. It would be appreciated if we could be provided with a full size drawing for our records.

If you have any questions or require further clarification, please contact me at the number listed above at your earliest convenience.

Sincerely,



Margaret Mikolajczak, CET  
Corridor Management Technician

c.c Roy McQuillin City of Vaughan  
Roy Mason KLM  
David Fallows HE  
Ram Dharamdial Planning&Environmental  
Wade F. Young Structural Engineering  
Betty Bennett Geotechnical

**From:** Betty Bennett  
**To:** MTOCR.DOWNSVCR.Trudeau, MTOCR.DOWNSVCR.Mikolajc  
**Subject:** Hwy 400 - Twin Tunnels Sewer Crossing

Rene, Margaret

I have reviewed the drawings for the proposed twin tunnel crossing of Highway 400 together with the Foundations Report prepared by Golder Associates for Schaeffer and Associates.

This particular proposal was first brought to our attention in March 1996 at which time we recommended that the developer carry out a detailed foundation investigation. The investigation has been conducted and recommendations provided by the consultant. Based on the foundation report and the drawings, the following comments are forwarded.

1) In general, it is preferable that the earth cover above the tunnel crown be greater than 1 to 1.5 times the tunnel diameter and that the tunnel spacing be greater than two tunnel diameters. The close proximity of the twin tunnels implies that, for design, the tunnels are treated as a single tunnel having a equivalent diameter of approximately 3.75 m. The earth cover in the vicinity of Highway 400 is in the order of 4 m. Given these very tight constraints, our concern is that, during construction, it will be difficult to ensure that settlement or heave will be less than 25 mm.

The construction spec forwarded with the drawings states that the Engineer should be notified if movements of 15 mm are experienced. If they reach 25 mm the jacking operation should stop and the contractor should develop a plan to mitigate the settlement and control future settlements. It is felt that the developer/contractor should have a plan in place prior to construction addressing excessive ground loss. It is recommended that the jacking operation be ceased after 15 mm of movement are experienced in order to avoid any immediate or future problems to Highway 400.

2) The issue of how damages to the pavement structure will be repaired should be addressed.

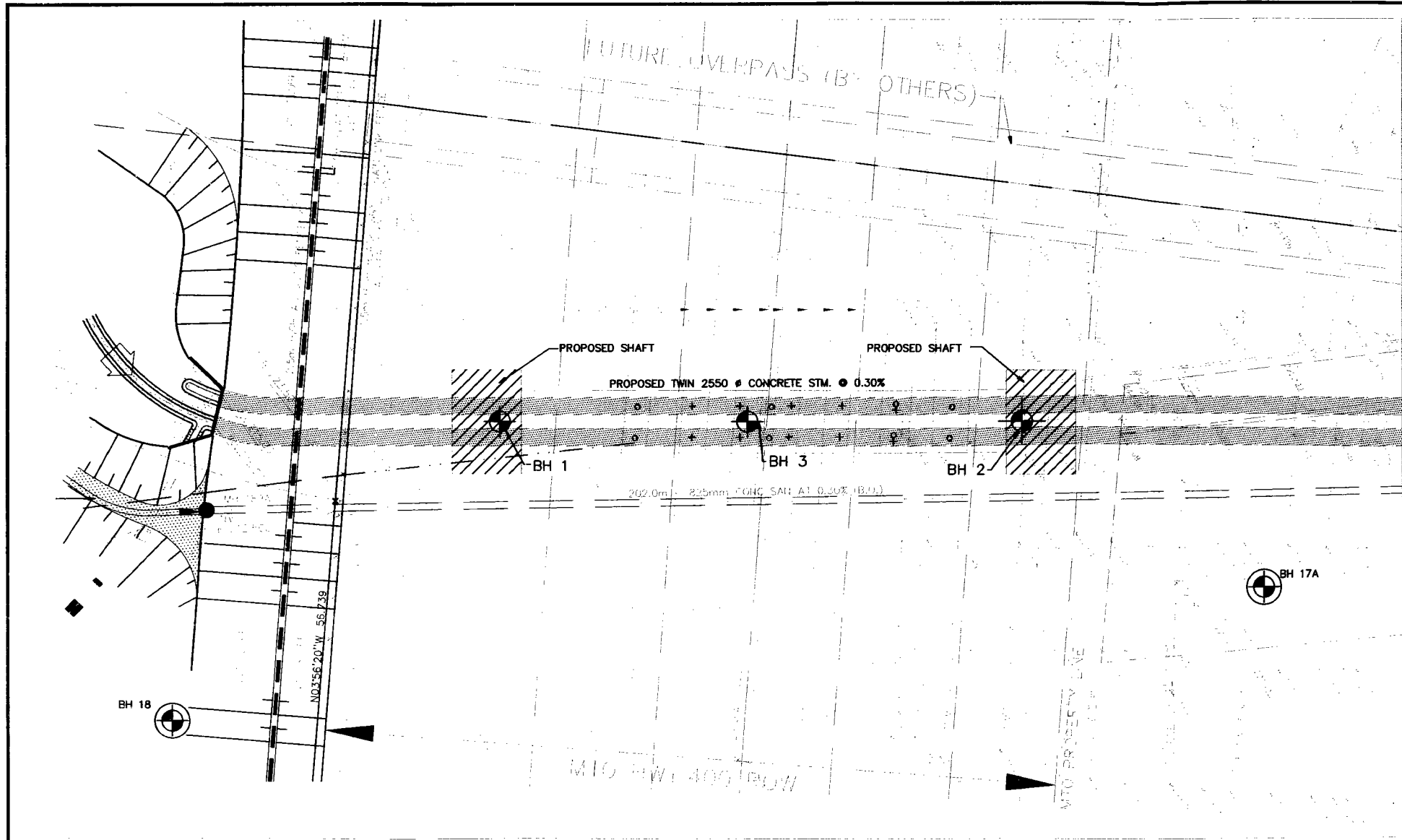
3) When the proposed sewer crossing was reviewed in 1996, Wade Young covered the structural aspects of the review. It may be prudent, if not done so already, to get his input on this as well.

4) The proposal from 1996 included a drawing of the tunnel cross-section showing both a primary and secondary lining. The foundation report assumes that the tunnels will be constructed by pipe-jacking using pre-cast concrete sections. A cross-section of the tunnels including concrete thickness should be detailed.

If there are any questions regarding the comments, please call.

Betty  
(-4333)

**CC:** Dundas



# BOREHOLE LOCATION PLAN

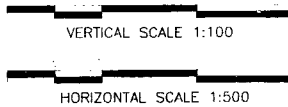
FIGURE 2

## LEGEND:

- BOREHOLE LOCATION IN PLAN, CURRENT GOLDER ASSOCIATES LTD. INVESTIGATION.
- BOREHOLE LOCATION IN PLAN, 1998 INVESTIGATION, AGRA EARTH AND ENVIRONMENTAL LIMITED, REPORT No. TT98-4-19, DATED JUNE 1998 AND MEMO No. TT98-4-19, DATED SEPTEMBER 1998.
- PROPOSED LOCATIONS OF IN-GROUND MONITORING POINTS
- PROPOSED LOCATIONS OF SURFACE MONITORING POINTS

## NOTES:

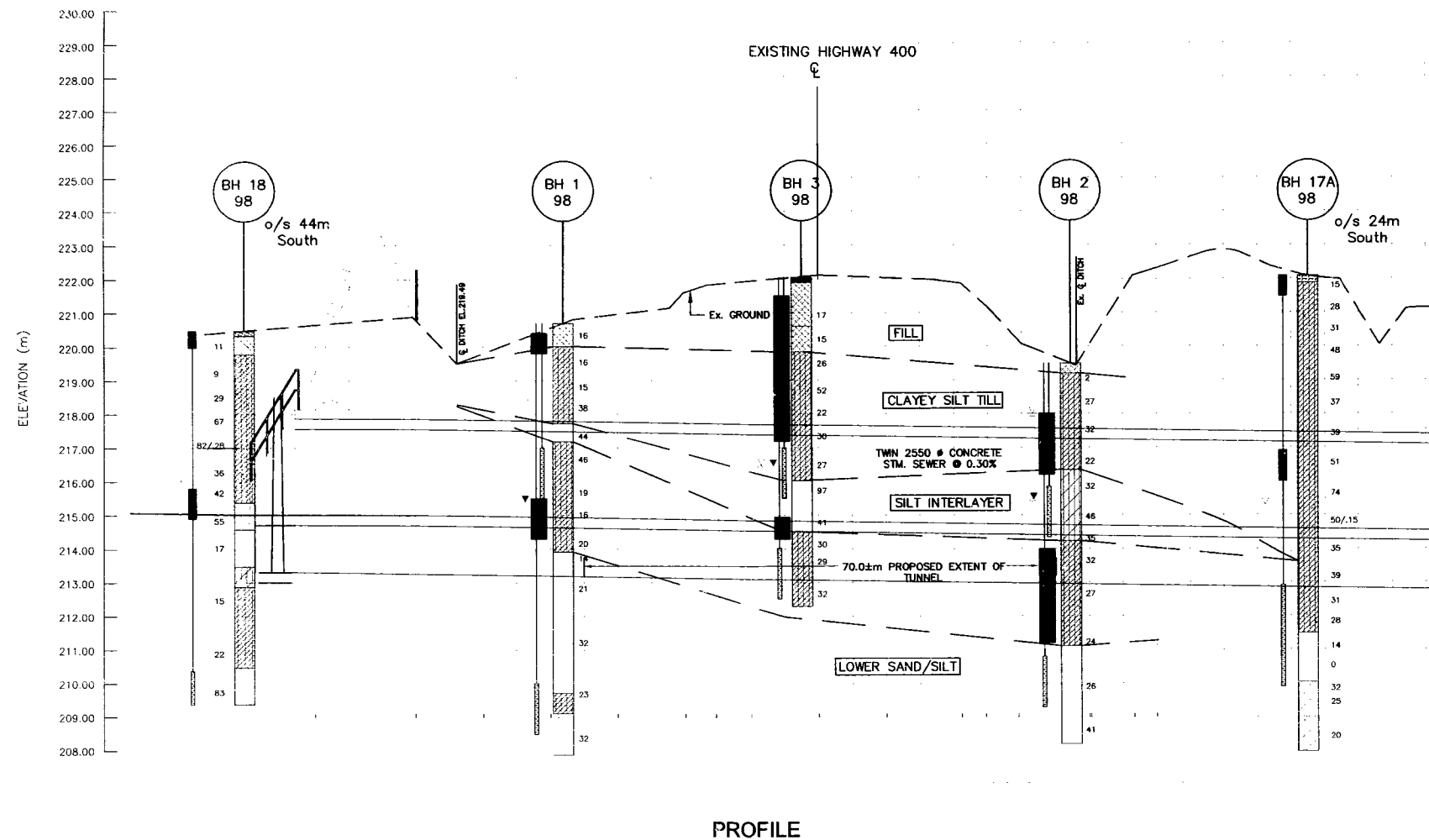
- PROPOSED TUNNEL ALIGNMENT OBTAINED FROM DRAWING No. PP-1, PROJECT No. 96-E-1869, DATED OCTOBER 1998 PROVIDED BY SCHAEFFERS AND ASSOCIATES IN DIGITAL FORMAT.



Golder Associates

# INTERPRETED STRATIGRAPHY ALONG TUNNEL ALIGNMENT

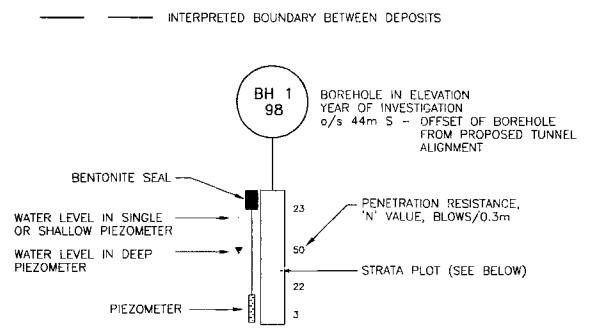
FIGURE 3



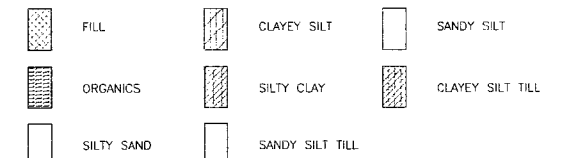
## NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE GEOTECHNICAL REPORT No. 981-1179, DATED JANUARY 1999.
2. THIS INTERPRETED STRATIGRAPHY DRAWING IS A SIMPLIFICATION OF THE SUBSURFACE CONDITIONS. DETAILED DESCRIPTIONS OF THE CONDITIONS ENCOUNTERED AT THE BOREHOLE LOCATIONS ARE FOUND ON THE RECORD OF BOREHOLES ATTACHED TO THIS REPORT AND IN THE GEOTECHNICAL REPORTS PREPARED BY OTHERS THAT ARE REFERENCED IN SECTION 3.
3. DEPOSITS ARE DELINEATED BY THE BOUNDARY LINE IDENTIFIED IN THE LEGEND. THE BOUNDARY LINES REPRESENT A GRADUAL TRANSITION FROM ONE SOIL TYPE TO ANOTHER AND ARE ILLUSTRATED TO PROVIDE A BASELINE. HOWEVER, VARIATION IN THE DEPOSIT BOUNDARIES FROM THOSE ILLUSTRATED MUST BE ANTICIPATED BOTH PARALLEL AND PERPENDICULAR TO THE SECTION LINES.
4. THE CHARACTERISTICS AND VARIABILITY ANTICIPATED WITHIN THE SOIL DEPOSITS ARE DESCRIBED IN THE TEXT OF THE REPORT. LENSES AND INTERLAYERS NOT DETECTED BY THE SUBSURFACE INVESTIGATION WILL BE PRESENT BETWEEN BOREHOLES.
5. CONSTRUCTION EQUIPMENT AND PROCEDURES MUST BE SELECTED TO ACCOMMODATE VARIATION IN THE DEPOSIT BOUNDARIES AS WELL AS VARIATIONS WITHIN THE DEPOSITS AS DESCRIBED IN THE REPORT TEXT. WHERE PRECISE DETERMINATION OF DEPOSIT BOUNDARIES AND DEPOSIT VARIABILITY ARE CRITICAL FOR SAFETY AND/OR STABILITY, THEY SHOULD BE VERIFIED BY INVESTIGATION DURING CONSTRUCTION.
6. BOREHOLE WIDTH IN PROFILE IS NOT TO SCALE.
7. PROPOSED TUNNEL GROUND SURFACE PROFILE OBTAINED FROM DRAWING No. PP-1, PROJECT No. 96-E-1869, DATED OCTOBER 1998, PROVIDED BY SCHAEFFER AND ASSOCIATES IN DIGITAL FORMAT.
8. BOREHOLE ELEVATIONS INTERPRETED FROM GROUND SURFACE PROFILE PROVIDED ON ABOVE REFERENCE DRAWING.

## LEGEND:



## STRATA PLOTS:



VERTICAL SCALE 1:100

HORIZONTAL SCALE 1:500

Golder Associates