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Golder Associates

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
GO TRANSIT SUBWAY AT
CARRUTHERS CREEK DRIVE
TOWN OF AJAX MUNICIPALITY OF DURHAM
W.P. 124-99-00
MINISTRY OF TRANSPORTATION, ONTARIO
CENTRAL REGION, DISTRICT 6, TORONTO

Submitted to:
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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by Totten Sims Hubicki Associates (TSH) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation for the site of the proposed GO Transit subway at Carruthers Creek Drive. The project is part of the MTO plan to expand Highway 401 to a future Core / Distributor system, which requires replacement of existing and construction of proposed intersections to accommodate the Highway 401 improvements. This project addresses the proposed Carruthers Creek Drive structure under the existing GO Transit rail lines, including a permanent soldier pile retaining wall.

The purpose of this investigation is to determine the subsurface conditions at the site of the proposed underpass structures by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. A General Arrangement plan for the overpass structures was provided to us by TSH in January 2001. The terms of reference for the scope of work are outlined in our proposal P01-1141, dated May 1999.

2.0 SITE DESCRIPTION

The site is located approximately 850 m east of the Highway 401 and Harwood Avenue interchange in the Town of Ajax, Regional Municipality of Durham. The future Carruthers Creek Drive will be carried under Highway 401 and the GO Transit and CN Rail lines.

The grade of the existing GO Transit rail lines is at about Elevation 96.4 m at the bridge site and the existing ground surface varies from about Elevation 94.5 m to 96.4 m. At the north end of the site there is a relatively thin vegetation cover consisting of bushes, trees and grass and the existing Highway 401. At the south end of the site there is a farmers field.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between October 6 and November 9, 2000 at which time a total of four boreholes were put down at the site (numbered 00-13 to 00-16). Two boreholes (numbered RW-4 and RW-5) were put down as part of the previous investigation as part of the "Highway 401 and Carruthers Creek Dive Interchange" (Golder Associates Report No. 991-1158, dated January 2000). Boreholes 00-13 and 00-14 were put down at the north limits of the GO Transit structure. Boreholes 00-15 and 00-16 were put down near the south limits of the CN Rail structure located immediately south of the GO Transit structure. Boreholes RW-4 and RW-5 were advanced along the alignment of the proposed retaining wall to the north of the GO Transit rail lines. It should be noted that no boreholes were drilled between the CN Rail and GO Transit structures due to the presence of utilities.

The current investigation was carried out using a track-mounted D-50 auger drill rig for Boreholes 00-13 to 00-16 and a CME-55 drill rig for Boreholes RW-4 and RW-5, both of which were supplied and operated by Master Soil Investigations Ltd of Toronto. The boreholes were advanced to depths of between 12.5 m and 18.3 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration test (SPT) procedure. Bedrock coring was carried out in Boreholes 00-14 to 00-16 for 3 m lengths. Groundwater conditions in the open boreholes were observed throughout the drilling operations. Piezometers, consisting of a 0.3 m long slotted section threaded onto 12 mm diameter rigid PVC tubing, were installed in selected boreholes to permit monitoring of the groundwater levels at these locations. The boreholes were backfilled using bentonite pellets in accordance with the most recent MTO guidelines.

The fieldwork was supervised on a full-time basis by a member of our engineering staff who located the staked boreholes in the field, directed the drilling, sampling, rock coring, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to our laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg limits tests, and grain size analyses were carried out on selected samples.

The borehole locations were established in the field by TSH prior to our mobilization to the site. The elevations and northing and easting co-ordinates of the boreholes drilled for this investigation were provided to us by TSH; the locations are shown in plan on Drawing 1. The northing and easting co-ordinates are referenced to the UTM NAD83 co-ordinate system and the elevations are referenced to the geodetic datum.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published geologic information, the site is located in the physiographic region known as the Iroquois Plain which formed part of the lakebed of the former Lake Iroquois after the last glaciation period during the Pleistocene Epoch. The Iroquois Plain in the area of the subject site primarily consists of glacial tills of a heterogeneous mixture of sand, silt, clay and gravel with numerous cobbles and boulders. It is thought that at least two distinct glaciations have occurred in the area. The most recent tills in the area were deposited in the form of drumlins (elongated, oval shaped hills). Recent glaciolacustrine and glaciofluvial deposits of sands, silts, gravel and clays, have been deposited in the depressions between these hills. At depth, the till is underlain by bedrock consisting of thinly bedded, dark grey, calcareous shale of the Whitby Formation. (Reference; "The Physiography of Southern Ontario", 3rd Edition, Chapman and Putnam, 1984).

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected samples, are given on the attached Record of Borehole sheets and on Figures 1 to 4 following the text of this report. The relevant borehole logs (RW-4 and RW-5) from our previous investigation are found in Appendix A. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations. An interpretive model of the soil stratigraphy is shown on Drawing 1.

In summary, the subsoils in the area of the proposed bridge structures consist of topsoil and fill surficial layers of varying thickness that are underlain by an upper till deposit. The upper till varies from clayey silt / silty clay till to clayey silty sand till and contains interlayers of silty sand or silty clay found above about Elevation 91 m to 93 m. Below this elevation exists a lower deposit of silty clay to clay till that extends to at least Elevation 82.2 m in the area of the GO Transit rail lines. The till deposits are hard / very dense and are characterized by Standard Penetration Test (SPT) 'N' values of generally greater than 100 blows per 0.3 m of penetration. Shale bedrock of the Whitby Formation was encountered at about Elevation 82 m at the proposed GO Transit subway at Carruthers Creek.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Fill

A 0.6 m to 0.8 m thick layer of brown clayey silt to silty clay fill was encountered below the ground surface. In Boreholes RW-4, RW-5, 00-13 and 00-14 between Elevation 94.6 m and 94.9 m. Some sand and trace to some gravel were noted within the fill as well as trace organics in Borehole 00-13. In Boreholes 00-15 and 00-16, 0.8 m of brown clayey silt topsoil was encountered below the ground surface at Elevation 95.8 m. The Standard Penetration Test 'N' values range between 3 blows to greater than 100 blows per 0.3 m of penetration indicating a soft to hard consistency. The measured water content on selected samples of the fill range from 9 to 20 percent.

4.2.2 Upper Clayey Silt / Silty Clay (Till)

In Boreholes RW-4, RW-5, 00-13 and 00-14, a 3.6 m to 8.2 m thick deposit of grey clayey silt with sand to clayey silty sand till containing trace to with gravel was encountered below the fill at about Elevation 94.0 m. The SPT 'N' values measured were greater than 100 blows per 0.3 m of penetration indicating that the till has a hard consistency. A grain size distribution curve for selected samples of this till is shown on Figure 1. Atterberg limits testing on three samples of this deposit gave a liquid limit ranging from about 15 to 18 percent and a plasticity index of about 6 percent. Measured water contents on selected samples of the till range from 4 to 30 percent and were generally below 20 percent. Results of Atterberg Limits testing are shown on Figure 2.

In Boreholes 00-15 and 00-16, a 5.3 m thick deposit of brown to grey clayey silt to silty clay with sand, trace to some gravel and occasional cobbles was encountered below the fill at about Elevation 95.0 m. SPT 'N' values measured in this deposit range from 4 blows to greater than 100 blows per 0.3 m of penetration indicating a stiff to hard consistency and in general, the 'N' values are greater than 100 blows below 1.5 m depth. A grain size distribution curve for selected samples of this till is also shown on Figure 1. Atterberg limits testing on two samples of this deposit gave a liquid limit ranging from about 15 to 21 percent and a plasticity index between 5 and 9 percent. Measured water contents on selected samples of this till are between 6 and 9 percent. The results of the Atterberg Limits testing on this sample are also shown on Figure 2. In Borehole 00-16, a 0.8 m thick layer of silty sand was encountered at about Elevation 93.0 m.

A 0.7 m thick layer of silty sand was encountered within this deposit in Borehole 00-14 at about Elevation 91.2 m. In Borehole RW-5, a 0.8 m thick layer of silty clay with sand was encountered within the deposit at about Elevation 91.9 m. The liquid limit of one sample of this material tested was at about 20 percent with a plasticity index of 11 percent. The results of the Atterberg Limits testing on this sample are shown on Figure 2 and indicate the till is of low plasticity. The measured water content on the one sample is about 5 percent.

4.2.3 Lower Silty Clay to Clay (Till)

A deposit of grey silty clay to clay till containing trace to with sand and trace to some gravel was encountered below the upper till deposit between Elevation 85.8 m and 90.4 m. Borehole RW-4 was terminated in this deposit and the deposit was between 8.0 m and 9.3 m thick in all other boreholes. Occasional sand partings were noted within the till. SPT 'N' values measured in this deposit range from 64 blows to greater than 100 blows per 0.3 m of penetration indicating a hard consistency. A grain size distribution curve for selected samples of this till is shown on Figure 3. Atterberg limits testing on samples of the silty clay to clay till gave a liquid limit ranging from about 20 to 43 percent and a plasticity index between 11 and 23 percent. The results of the Atterberg Limits testing selected samples of this deposit are shown on Figure 4 and indicate the till is of intermediate plasticity. Measured water contents on selected samples of this till are between 8 and 25 percent.

4.2.4 Bedrock

The bedrock surface was encountered in the boreholes below the silty clay to clay till at the Elevations shown in the table below.

Borehole	*Depth (m)*	*Elevation (m)*
RW-5	13.4	81.2
00-13	13.6	81.1
00-14	13.6	81.3
00-15	15.3	80.6
00-16	14.3	81.5

The bedrock is described as dark grey, moderately to slightly weathered, fine grained, thinly bedded, slightly carbonaceous shale of the Whitby Formation containing numerous thin (less than 1 mm thick) limestone lenses. The bedrock was cored in Boreholes 00-14 to 00-16 for a length of 3 m. In Boreholes RW-5 and 00-13, the bedrock was augered for a length of about 1.7 m. The Rock Quality Designation (RQD) measured in the core was between 0 and 30 percent indicating a rock quality of very poor to poor quality.

4.3 Groundwater Conditions

Water levels were noted in the open boreholes during and upon completion of the drilling operation; these levels are shown on the attached Record of Borehole sheets. The piezometers in Boreholes RW-5 and 00-15 were sealed into the bedrock while the piezometer in Borehole 00-16 was sealed within the upper till. Details of the piezometer installations and water level measurements are shown on the attached Record of Borehole sheets. The water levels in the boreholes and piezometers are summarized in the table below.

Borehole	*On Completion of Drilling*		*Water Levels in Piezometers*					
			October 19, 1999		*December 27, 2000*		*February 9, 2001*	
	Depth (m)	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*
RW-4	Dry	---	---	---	---	---	---	---
RW-5	---	---	4.0	90.5	2.3	92.3	2.6	92.0
00-13	13.7	81.0	---	---	---	---	---	---
00-15	---	---	---	---	5.7	90.1	5.7	90.1
00-16	---	---	---	---	1.3	94.5	1.2	94.6

Based on the measured water levels, it is evident that under drainage is occurring at the site and that there is a downward hydraulic gradient through the till deposits. The groundwater table associated with the upper till is at about Elevation 94.6 m. The groundwater table associated with the bedrock is at or below about Elevation 92 m. Based on the above water levels and water levels in piezometers put down for other investigations in the general area this site, the groundwater table slopes downward towards the southwest.

It should be noted that the groundwater level is subject to seasonal fluctuations; higher groundwater level conditions might be observed after heavy rainfall or during snow melt.

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PART B

FOUNDATION DESIGN REPORT
GO TRANSIT SUBWAY AT
CARRUTHERS CREEK DRIVE
TOWN OF AJAX MUNICIPALITY OF DURHAM
W.P. 124-99-00
MINISTRY OF TRANSPORTATION, ONTARIO
CENTRAL REGION, DISTRICT 6, TORONTO

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of the foundation design of the proposed works at the proposed GO Transit bridge structure at Carruthers Creek based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

It is understood that a new interchange at Highway 401 and Carruthers Creek Drive is planned as part of the Highway 401 widening to form a core / collector distribution system in this area. Carruthers Creek Drive will extend underneath Highway 401 and the GO Transit and CN Rail tracks. The Highway 401 Carruthers Creek overpass structure was addressed in a previous report (W.P. 242-86-00). The GO Transit bridge site, which is the subject of this report, involves a single-span structure to carry the GO Transit tracks over Carruthers Creek Drive. The works include retaining walls between the CN Rail and GO Transit structures and a soldier pile retaining wall to be used as both temporary and permanent works to separate the Highway 401 ramps from the GO Transit structure. It is understood that the project staging will involve track diversions which will be at grade and located to the north of the existing GO Transit lines.

The existing GO Transit rail track grade is at about Elevation 96.4 m. The proposed grade of Carruthers Creek Drive at the subway structure will be at about 87.6 m.

5.2 Foundations

5.2.1 General

The predominant soil deposits at this site consist of an upper clayey silt / silty clay till deposit underlain by a lower, more plastic deposit of silty clay to clay till. Both deposits are hard with measured Standard Penetration Test (SPT) 'N' values of generally greater than 100 blows per 0.3 m of penetration. The upper till deposit extends to about Elevation 86 m to 90 m and the lower till extends to the shale bedrock surface at about Elevation 81 m. The measured water level associated with the till deposits is at about Elevation 94.6 m (about 7 m above the proposed Carruthers Creek Drive grade).

The native till soils at the site are generally suitable for support of the proposed abutments on shallow foundations. Due to the hard nature of the till deposits, driven steel piles are not considered appropriate since they would require near full depth pre-augering in order to advance the piles.

5.2.2 Bridge Footings

5.2.2.1 Allowable Soil Pressure

The bridge abutments may be supported on spread footings placed within the undisturbed hard silty clay to clay till deposit. Based on the General Arrangement Plan, it is understood that the east and west abutment footings will be at about Elevation 84.8 m, about 2.5 m below the future Carruthers Creek Drive grade. Therefore, the abutments for this closed abutment configuration will be founded below the groundwater level at the site. Alternatively, consideration could be given to an open configuration in which the abutments could be founded at a higher elevation.

For the site conditions, the allowable bearing capacity will be less than the allowable pressure for settlement (assuming a total settlement of 25 mm) and will govern design. An allowable soil pressure (in accordance with AREMA 1999) of 600 kPa may be assumed for design for footings placed at or below Elevation 94.6 m (i.e. below the groundwater table). For footings placed above Elevation 94.6 m (i.e. above the groundwater table), the allowable pressure for settlement will govern design and an allowable soil pressure of 1,000 kPa may be assumed for design. These values assume a footing width of 6.5 m, as based on information provided by TSH.

The above design values given should be checked against the Ontario Highway Bridge Design Code (OHBDC). For an assumed footing width of 6.5 m, a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kPa should be used for footings placed at or below Elevation 94.6 m. For footings placed above Elevation 94.6 m, a factored geotechnical resistance at ULS of 1,200 kPa should be used for design. A geotechnical resistance at Serviceability Limit States (SLS) of 1,000 kPa should be assumed for design.

The founding levels shown on the General Arrangement Plan (Elevation 84.8 m) are about 10 m below the groundwater level within the till deposits. The above soil pressures assume that appropriate construction procedures are adopted during footing construction to ensure that the founding soils are not softened / disturbed prior to concrete placement.

5.2.2.2 Resistance to Lateral Forces

A coefficient of friction equal to 0.45 may be assumed between the concrete footing and the prepared silty clay till subgrade for calculation of sliding resistance for footings placed below Elevation 89 m. For footings above Elevation 89 m, a coefficient of friction equal to 0.55 may be assumed for footings placed on the prepared clayey silt till subgrade.

5.2.2.3 Frost Protection

All footings should be provided with a minimum of 1.2 m of earth cover for frost protection purposes.

5.2.2.4 Construction Considerations

The proposed founding level of about Elevation 84.8 m will be about 10 m below the groundwater level. There is a slight downward gradient evident within the till deposits; however, some water inflow into footing excavations should be expected through the till. Pumping from well-filtered sumps placed at the base of the excavations should provide sufficient groundwater control during foundation excavations.

The founding soils are sensitive to disturbance and softening due to water seepage or ponding. Mud coat placement will be required at the base of excavation to protect the founding soils against softening due to upward water seepage. The mud coat should be placed immediately after the founding level is reached and the base cleaned. Prolonged exposure without protection of the mud coat will allow the water seepage to soften the founding soils. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the mud coat. It should be noted that the water levels could be higher during wet periods of the year.

5.2.3 Retaining Wall Footings

5.2.3.1 Allowable Soil Pressure

The retaining wall, located between the GO Transit and CN Structures, may be supported on spread footings placed within the undisturbed, hard lower till deposit. It is understood that the retaining walls between the structures are to be founded at about Elevation 85.8 m (about 1.8 m below Carruthers Creek Drive grade), which is about 8.8 m below the groundwater level at the site.

For the site conditions, the allowable bearing capacity will be less than the allowable pressure for settlement (assuming a total settlement of 25 mm) and will govern design. The allowable soil pressure (in accordance with AREMA 1999) to be assumed for design of the retaining wall located between the CN and GO Transit structures is given in the table below.

Retaining Wall	*Assumed Footing Width (m)*	*Allowable Soil Pressure (kPa)*
Between CN and GO Structures	4.5	435
	5.5	500
	6.75	550

The above design values given should be checked against the OHBDC values given in the table below:

Retaining Wall	*Assumed Footing Width (m)*	*Factored Geotechnical Resistance at ULS (kPa)*
Between CN and GO Structures	4.5	650
	5.5	750
	6.75	850

It should be noted that the for the geotechnical resistance at SLS, the pressure required to cause 25 mm of settlement will be greater than the ULS value and therefore the ULS values will govern design

These founding level for the footings on the till are about 8.5 m below the groundwater level which is at about Elevation 94.6 m. The above soil pressures assume that appropriate construction procedures are adopted during footing construction to ensure that the founding soils are not softened / disturbed prior to concrete placement.

5.2.3.2 Resistance to Lateral Forces

A coefficient of friction equal to 0.45 may be assumed between the concrete footing and the prepared silty clay till subgrade for calculation of sliding resistance.

5.2.3.3 Frost Protection

All footings should be provided with a minimum of 1.2 m of earth cover for frost protection purposes.

5.2.3.4 Construction Considerations

Considerations with regard to subgrade preparation should be taken into account as described in Section 5.2.2.4.

5.3 Lateral Earth Pressure

The lateral earth pressures acting on the bridge abutment walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings and on subsequent movement of the structures. The following recommendations are made concerning the design of the abutment walls:

- Select free-draining granular fill such as that meeting the specifications of the OPSS Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the #200 sieve, should be used as backfill behind walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm, to at least 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS 501.

- Longitudinal drains and weep holes should be installed to provide drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I from OHBDC Figure 6-7.4.1) or within a wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical from the footing (Case II from OHBDC Figure 6-7.4.4).

- If the structural support allows movement of the top of the wall of at least 0.5 percent of the retained height (unrestrained structure), "active" earth pressures may be used in the geotechnical design of the structure. If the support does not allow sufficient lateral movement (restrained structure), "at rest" pressures should be assumed for geotechnical design.

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the wall stem in accordance with OHBDC Figure 6-7.4.3. Other surcharge loadings should also be accounted for in the design, as required. Compaction equipment should be used in accordance with OPSS 501.06.

- For Case I, the lateral earth pressures are based on the in-situ soils (native till) and the following parameters may be assumed:

Soil Unit Weight	23 kN/m^3
Angle of Internal Friction	38°
Coefficient of Lateral Earth Pressure	
"active"	0.24
"at rest"	0.38

- For Case II, the lateral earth pressures are based on the granular fill as placed; the following parameters may be assumed:

	Granular 'A'	**Granular 'B'** *Type II*
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Angle of Internal Friction	35°	32°
Coefficient of Lateral Earth Pressure		
"active"	0.27	0.31
"at rest"	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Where there is sloping ground behind the wall, the coefficient of lateral earth pressure must be adjusted (increased) to account for the slope. Other aspects of the backfill requirement with respect to subdrains and frost taper should be in accordance with OPSD 3501.00 (abutments) and OPSD 3504.00 (retaining walls).

5.4 Excavations and Temporary Cut Slopes

5.4.1 General

Excavations for footing construction will extend through the fill and upper and lower till deposits consisting of clayey silt to silty clay. At the proposed bridge structure, the excavations for the footings will be up to about 11.5 m in depth below existing ground surface. Cobbles and boulders are inherent in the glacial deposits as encountered at this site and should be expected during excavation. The excavation bases will be up to 9 m below the groundwater level as measured in the piezometers. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the excavation could also be carried out within a fully braced excavation.

Water seepage inflow into the excavations through the clayey silt / silty clay till will occur but is expected to be moderate, except during periods of sustained precipitation and / or if sandier zones are intercepted within the till. Pumping from well-filtered sumps located at the base of the excavation within the glacial till should provide adequate groundwater control during foundation excavations. Sumps should be maintained outside the actual footing limits. Surface water run-off should be directed away from the excavations at all times.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health & Safety Act. The native soils at this site would be classified as Type I soil.

5.4.2 Temporary Retaining Walls

It is understood that the construction staging and track diversion will involve track diversions constructed at grade. Where track protection is required for footing construction due to the proximity of the GO Transit, CN Rail lines and diversion track lines, a temporary support system should be installed to support the sides of the excavation and permit the use of vertical cuts. The temporary support system could consist of soldier piles and lagging where the piles would be socketted into pre-augered holes extended into the hard till deposit or shale bedrock below the excavation base. Some cobbles and boulders should be expected during augering for the soldier pile installation. Given the height of the proposed wall adjacent to the structure, up to about 11.5 m, tiebacks / anchors will be required along the wall for lateral support. The anchors could be formed within the hard till deposits or within the shale bedrock.

The design of braced soldier pile and lagging walls, where the support to the wall is provided by anchors, should be based on a triangular earth pressure distribution using the design parameters given below. The temporary anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as wells as the impact of sloping ground behind the system.

Unfactored triangular earth pressure distribution (p in kN/m^2; increasing with depth), can be calculated as follows:

$$p = K_a \gamma H$$

where

H	=	the height of the excavation at any point in metres		
K_a	=	0.24	above Elevation 89 m (for level ground behind excavation)	
	=	0.31	below Elevation 89 m (for level ground behind excavation)	
γ	=	saturated soil unit weight	$= 23\ kN/m^3$	above Elevation 89 m
			$= 22\ kN/m^3$	below Elevation 89 m

The coefficient of passive lateral earth pressure, K_p, for the socket within the hard till may be taken as 4.2 above Elevation 89 m and 3.2 below Elevation 89 m. A groundwater level at Elevation 94.6 m should be assumed at the bridge footing locations. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter. If the spacing is reduced, an effective width of two times the pile socket diameter should be used to calculate passive earth pressure.

The allowable axial resistance for steel piles founded on the bedrock may be taken as 1,000 kN. For steel piles socketted within the silty clay till present below Elevation 89 m and socketted at least 2 m below the final road grade, the allowable capacity may be taken as 800 kN. These values assume a 0.9 m diameter socket.

The above design values should be checked against the OHBDC. For the assumed socket diameter at 0.9 m, the factored axial resistance at ULS for steel piles founded on the bedrock may be taken as 1,000 kN. SLS conditions do not apply for piles founded on the bedrock surface. For piles socketted into the silty clay till, the ULS and SLS resistances may be taken as 800 kN.

The horizontal resistance for the soils in front of an individual pile can be estimated using the following equation:

$$k_h = k_{s1} / 5d$$

where:

k_h	=	coefficient of horizontal subgrade reaction (MPa/m)
d	=	pile width (m)
ks_1	=	constant of horizontal subgrade reaction (MPa/m)
	=	60 MPa/m to 100 MPa/m for the hard clayey silt / silty clay till deposits

Anchors

The anchors will typically have their fixed length (bond zone) generally within the upper clayey silt till, the lower silty clay till, and the bedrock and may be sized based on the ultimate bond stresses given in the table below. The sustained working load should not be greater than 60 percent of the ultimate tensile strength of the anchor tendons or bars. A Factor of Safety equal to 2.0 should be used for the design of the temporary anchors.

Deposit	*Elevation*	*Ultimate Bond Stress Acting Between Grout and Soil / Bedrock*
Upper Clayey Silt Till	Above Elevation 89 m	250 kPa
Lower Silty Clay Till	Between Elevation 81 m and 89 m	250 kPa
Shale Bedrock	Below Elevation 81 m	700 kPa

The above ultimate bond stresses assume a fixed anchor length (bond zone) of not greater than 8 m. Where the bond zone is greater than 8 m and up to 11 m, the bond stress along the full fixed length within the clayey silt / silty clay till should be reduced to 175 kPa. It should be noted that these bond stresses within the clayey silt / silty clay till assume no post-grouting. Where secondary (post) grouting is carried out, the capacity can be increased. The fixed length of the anchor should be maintained behind a line drawn upward at 45 - $\varphi'/2$ degrees plus a minimum of 0.15H from the base of the piles. The friction angle of the silty clay till behind the wall may be taken as 30 degrees for the purpose of the wall / anchor design. A minimum anchor spacing of 3.5 x b should be used in design where b is the diameter of the fixed length of the anchor.

Because the ground-to-anchor bond in soil is highly dependent upon the installation technique, the Contractor should be held to an anchor performance specification enforced by proof tests and lift-off tests on all anchors and a performance test on at least one anchor in accordance with Special Provision 942S01. Anchor installation and testing should be carried out under the full-time inspection of a geotechnical engineer. A performance test should be carried out, to 200 percent of the design working load, on at least one anchor to confirm the design and the Contractor's installation method. In addition, each anchor should be proof-tested to 150 percent of the working load. The tensile stress in the anchor bar or strands during test loading should not exceed 80 percent of the guaranteed ultimate tensile strength of the bar or strands.

5.5 Permanent Soldier Pile Walls

It is understood that the temporary support system required along the north side of the GO Transit for footing construction will be converted for use as a permanent structure. These permanent retaining walls will extend along the Highway 401 ramps on the east and west sides of Carruthers Creek Drive. Concrete panels will be used as the structural facing.

The height of the proposed wall will be up to about 9.2 m closest to Carruthers Creek Drive. The steel H-piles will be placed in pre-augered holes extended to below the propose ramp grade which will vary along the length of the wall. Permanent tiebacks / anchors will be required to support the wall; these anchors could be formed within the hard till deposits or within the shale bedrock. Since this permanent wall is adjacent to the GO Transit, the anchors must be designed to accommodate the long-term cyclic loading from the rail lines.

The design of the wall, where the support to the wall is provided by anchors, should be based on a triangular earth pressure distribution with design parameters as given below and water pressures using the design elevation given below. The wall and the permanent anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as wells as the impact of sloping ground behind the system. Unfactored triangular earth pressure distribution (p in kN/m^2; increasing with depth), can be calculated as follows:

$$p = K_a \gamma H$$

where

H = the height of the excavation at any point in metres

K_a = 0.24 above Elevation 89 m (for level ground behind excavation)

= 0.31 below Elevation 89 m (for level ground behind excavation)

γ = saturated soil unit weight = 23 kN/m^3 above Elevation 89 m

= 22 kN/m^3 below Elevation 89 m

A design groundwater level at Elevation 94.6 m should be assumed at the site. The submerged unit weights, below the groundwater level, may be taken as 13.2 kN/m^3 and 12.2 kN/m^3 above and below Elevation 89 m, respectively.

For determining lateral resistance to the soldier pile sockets below the adjacent ramp grade, a coefficient of passive lateral earth pressure, K_p, for the hard till deposits may be taken as 4.2 above

Elevation 89 m and 3.2 below Elevation 89 m. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter. If the pile spacing is reduced, an effective width of two times the socket diameter should be used to calculate passive earth pressure.

The allowable axial resistance for steel piles socketted within the till and founded on the bedrock may be taken as 1,000 kN. For steel piles socketted within the silty clay till present below Elevation 89 m and socketted at least 2 m below the final road grade, the allowable capacity may be taken as 800 kN. These values assume a 0.9 m diameter socket.

The above design values should be checked against the OHBDC. For the assumed socket diameter at 0.9 m, the factored axial resistance at ULS for steel piles founded on the bedrock may be taken as 1,000 kN. SLS conditions do not apply for piles founded on the bedrock surface. For piles socketted into the silty clay till, the ULS and SLS resistances may be taken as 800 kN.

The horizontal resistance for the soils in front of an individual pile can be estimated using the following equation:

$$k_h = ks_1 / 5d$$

where:

k_h = coefficient of horizontal subgrade reaction (MPa/m)
d = pile width (m)
ks_1 = constant of horizontal subgrade reaction (MPa/m)
= 60 MPa/m to 100 MPa/m for the hard clayey silt / silty clay till deposits

Placement of a free draining sand layer or a geonet / miradrain is recommended behind the wall facing to provide drainage to the till deposits. The groundwater table within the till may decrease somewhat with time due to the ramp cut and the Carruthers Creek Drive cut; however, the wall drainage should be provided to ensure adequate control. The drainage layer / system should be connected to the road storm drainage system. A 50 mm thick insulation layer should also be provided behind the wall to provide full frost protection to the soils behind the wall.

The anchors will typically have their fixed length (bond zone) generally within the till deposits and the bedrock and will be below the groundwater table. The anchors may be sized based on the ultimate bond stresses and fixed anchor lengths given in the table below.

Deposit	*Elevation*	*Fixed Anchor Length (Bond Zone) (m)*	*Ultimate Bond Stress Acting Between Grout and Soil / Bedrock*
Upper Clayey Silt Till	Above Elevation 89 m	Less than 8 m – no post-grouting Less than 8 m – with post-grouting Between 8 m and 11 m – no post-grouting Between 8 m and 11 m – with post-grouting	250 kPa 350 kPa 175 kPa 275 kPa
Lower Silty Clay Till	Between Elevation 81 m and 89 m	Less than 8 m – no post-grouting Less than 8 m – with post-grouting Between 8 m and 11 m – no post-grouting Between 8 m and 11 m – with post-grouting	250 kPa 350 kPa 175 kPa 275 kPa
Shale Bedrock	Below Elevation 81 m	Less than 8 m – no post-grouting	700 kPa

A Factor of Safety equal to 2 should be used for permanent anchors except where the wall / anchors will be subject to long-term cyclic loading. A Factor of Safety of 3 should be used where long-term cyclic loading (train loading) is anticipated.

The sustained working load should not be greater than 60 percent of the ultimate tensile strength of the anchor tendons or bars. The fixed length of the anchor should be maintained behind a line drawn upward at 45 - φ'/2 degrees plus a minimum of 0.15H from the base of the piles. The friction angle of the silty clay till behind the wall may be taken as 30 degrees for the purpose of the wall / anchor design.

The installation method chosen by the Contractor must ensure that loss of ground is minimized and water inflow is controlled during installation.

Because the ground-to-anchor bond in soil is highly dependent upon the installation technique, the Contractor should be held to an anchor performance specification enforced by proof tests and lift-off tests on all anchors and performance testing on a selected number of anchors within the permanent tied back wall length in accordance with Special Provision 942S01. Anchor installation and testing should be carried out under the full-time inspection of a geotechnical engineer. The performance testing should be carried out to 200 or 300 percent of the design working load (i.e. to the ultimate load), depending on the factor of safety used. A 300 percent performance test is specified where a Factor of Safety of 3 has been used for anchors subject to long-term cyclic train loading. The performance testing should be carried out on 10 percent of the total number of permanent anchors to confirm the design and the Contractor's installation method. In addition, each anchor should be proof-tested to 150 percent of the working load. The tensile stress in the anchor bar or strands during test loading should not exceed 80 percent of the guaranteed ultimate tensile strength of the bar or strands.

GOLDER ASSOCIATES LTD.

for Sarah E.M. Poot, P.Eng.,
Geotechnical Engineer

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Principal

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SEMP/ASP/FJH/clg
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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
SS	Split-spoon
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 Cone Penetration Resistance; N_d:
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)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 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_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
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_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

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LIST OF SYMBOLS

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

I. GENERAL

π = 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

γ shear strain
Δ change in, e.g. in stress: $\Delta\,\sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ 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
γ' 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 ρ. Unit weight symbol is γ 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_α 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
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction = tan δ
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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
$\parallel$ - Parallel To	
$\perp$ - Perpendicular To	

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Foundation Design

RECORD OF BOREHOLE No 00-13

METRIC

PROJECT 001-8019F
W.P. 124-99-00 LOCATION N 4857127.0; E 344062.0 ORIGINATED BY PKS
DIST 6 HWY 401 BOREHOLE TYPE 108mm Solid Stem Augers COMPILED BY DKB
DATUM Geodetic DATE Nov.3/00 CHECKED BY ASP

Soil profile:

ELEV / DEPTH	DESCRIPTION
94.7	GROUND SURFACE
0.0	Clayey Silt, some sand and gravel, occasional organics; Soft to hard; Brown; Moist; (Fill)
94.0 / 0.7	Clayey Silt with sand, trace gravel, occasional cobbles; Hard; Grey; Moist; (Till)
90.4 / 4.3	Silty Clay, trace to some sand and gravel with sand partings; Hard; Grey; Moist; (Till)
81.1 / 13.6	Dark grey SHALE BEDROCK. (Whitby Formation)

Samples:

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	3		
~~2~~	~~SS~~	~~100/.10~~		
3	SS	100/.15		0 39 52 9
~~4~~	~~SS~~	~~100/.08~~		
~~5~~	~~SS~~	~~100/.10~~		
~~6~~	~~SS~~	~~100/.05~~		
7	SS	75/.15		
8	SS	80/.15		
9	SS	75/.15		6 13 39 42
10	SS	100/.15		
11	SS	75/.15		
12	SS	110		
~~13~~	~~SS~~	~~100/.05~~	∇	

ELEVATION SCALE: 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

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RECORD OF BOREHOLE No 00-13

METRIC

PROJECT 001-8019F
W.P. 124-99-00
LOCATION N 4857127.0; E 344062.0
ORIGINATED BY PKS
DIST 6 HWY 401
BOREHOLE TYPE 108mm Solid Stem Augers
COMPILED BY DKB
DATUM Geodetic
DATE Nov.3/00
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
79.4			14	SS	120/.02						
15.3	END OF BOREHOLE Note: 1. Water level measured in open borehole at 13.7m depth (Elev.81.0m) upon completion of drilling.						79				

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

ON_MOT 001-8019.GPJ ON_MOT.GDT 15/5/01

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RECORD OF BOREHOLE No 00-14

METRIC

PROJECT 001-8019F
W.P. 124-99-00
LOCATION N 4857115.0; E 344024.0
ORIGINATED BY PKS
DIST 6 HWY 401
BOREHOLE TYPE 108mm Solid Stem Augers
COMPILED BY DKB
DATUM Geodetic
DATE Oct.6/00
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT / LIMITS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.9	GROUND SURFACE					
0.0	Clayey Silt, some sand and gravel; Firm; Brown; Wet; (Fill)	1	SS	5	o	
94.1 / 0.8	Clayey Silt with sand, trace to some gravel, occasional cobbles; Hard; Grey; Moist; (Till). Silty sand layer between 3.7m and 4.4m depth.	2	SS	100		
		3	SS	118/.15	o⊢	
		4	SS	100/.13		
		5	SS	100/.15	o	
		6	SS	100/.10		
		7	SS	100/.05	o	
89.3 / 5.6	Silty Clay, trace to with sand, trace gravel with sand partings; Hard; Grey; Moist; (Till)	8	SS	100/.10		
		9	SS	100/.13	o⊢	1 32 41 26
		10	SS	100/.13		
		11	SS	97		
		12	SS	115/.15	o	
81.3 / 13.6	Dark grey SHALE BEDROCK. (Whitby Formation)	13	SS	125/.03		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

ON_MOT 001-8019.GPJ ON_MOT.GDT 15/5/01

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 00-14

METRIC

PROJECT 001-8019F			
W.P. 124-99-00	LOCATION N 4857115.0; E 344024.0	ORIGINATED BY PKS
DIST 6 HWY 401	BOREHOLE TYPE 108mm Solid Stem Augers	COMPILED BY DKB
DATUM Geodetic	DATE Oct.6/00	CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
79.6 / 15.3	Dark grey, modererately to slightly weathered, fine grained, thinly bedded SHALE bedrock (Whitby Formation), occasional thin limestone beds. Bedrock cored from 15.3m to 18.3m. For bedrock coring details refer to Record of Drillhole 14.		~~14~~	~~SS~~	~~100/.02~~		79 78 77				
76.6 / 18.3	END OF BOREHOLE										

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

ON_MOT 001-8019.GPJ ON_MOT.GDT 15/5/01

PROJECT: 001-8019F

RECORD OF DRILLHOLE: 00-14

SHEET 1 OF 1

LOCATION: N 4857115.0; E 344024.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 6, 2000

DRILL RIG: Bombadier D-50

DRILLING CONTRACTOR: Master Soils

DATUM:

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous Page	79.60			
16	Rotary	Dark grey, moderately to slightly weathered, fine grained, thinly bedded SHALE bedrock (Whitby Formation) with occasional thin limestone beds (15.2m-15.3m). Sandstone bed 15.3m-15.4m,, light grey, moderately to slightly weathered shaley rock, medium grained. Moderately to closely bedded, 15.3m-18.3m, core broken 17.3m-17.4m.	15.26	1	PL	
17				2	BC	
18			76.57 18.29		PL	
		END OF HOLE				

Depth scale marks: 19, 20, 21, 22, 23, 24, 25

Other column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY k, cm/sec; DIAMETRAL POINT LOAD INDEX (MPa)

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PKS

CHECKED:

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 00-15

METRIC

PROJECT	001-8019F		
W.P.	124-99-00	LOCATION	N 4857092.0; E 344085.0
DIST	6 HWY 401	BOREHOLE TYPE	108mm Solid Stem Augers
DATUM	Geodetic	DATE	Nov.7-8/00

ORIGINATED BY PKS
COMPILED BY DKB
CHECKED BY ASP

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED 20 40 60 80 100 | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION
95.8	GROUND SURFACE
0.0	Clayey Silt Topsoil Stiff Brown Moist
95.1 0.8	Clayey Silt to Silty Clay with sand, trace to some gravel, occasional cobbles Stiff to hard Brown to grey below 3.8m Moist (Till)
89.7 6.1	Silty Clay, trace to some sand and gravel with fine sand partings Hard Grey Moist (Till)

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	14				
2	SS	14				
3	SS	100/.15				
4	SS	60/.08				
5	SS	60/.05	20	38	29	12
6	SS	100/.15				
7	SS	100/.15				
8	SS	129				
9	SS	100/.15	1	8	41	50
10	SS	100/.15				
11	SS	100/.13				
12	SS	113				
13	SS	100				

Elevation scale: 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 00-15

METRIC

PROJECT 001-8019F

W.P. 124-99-00 | LOCATION N 4857092.0; E 344085.0 | ORIGINATED BY PKS

DIST 6 HWY 401 | BOREHOLE TYPE 108mm Solid Stem Augers | COMPILED BY DKB

DATUM Geodetic | DATE Nov.7-8/00 | CHECKED BY ASP

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION — CONTINUED FROM PREVIOUS PAGE —	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
80.6			14	SS	100/.05						
15.3	Dark grey SHALE BEDROCK. (Whitby Formation) Dark grey, moderately to slightly weathered, fine grained, thinly bedded SHALE bedrock (Whitby Formation), slightly carbonaceous with many thin carbonaceous (limestone) lenses. Bedrock cored from 15.3m to 18.3m. For bedrock coring details refer to Record of Drillhole 15.						80 79 78				
77.5 18.3	END OF BOREHOLE Note: 1. Water level measured in piezometer at 5.7m depth (Elev.90.1m) on Dec.27/00 and Feb.9/01.										

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

PROJECT: 001-8019F

RECORD OF DRILLHOLE: 00-15

SHEET 1 OF 1

LOCATION: N 4857092.0; E 344085.0

DRILLING DATE: November 8, 2000

DATUM:

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Bombadier D-50

DRILLING CONTRACTOR: Master Soils

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous Page		80.50												
16–18	Rotary	Dark grey, moderately to slightly weathered, fine grained, thinly bedded SHALE bedrock (Whitby Formation), slightly carbonaceous with many thin carbonaceous (limestone) lenses <1mm thick. Moderately to closely bedded 15.3m-16m, closely bedded 16m-16.8m, moderately to closely bedded 16.8m-17.6m, closely to moderately bedded 17.6m-18.3m.		15.29	1											
					2								PL			SAND
				77.50 18.29												
		END OF HOLE														

Depth scale: 16, 17, 18, 19, 20, 21, 22, 23, 24, 25

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PKS

CHECKED:

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 00-16

METRIC

PROJECT 001-8019F			
W.P. 124-99-00	LOCATION N 4857080.0; E 344047.0		ORIGINATED BY PKS
DIST 6	HWY 401	BOREHOLE TYPE 108mm Solid Stem Augers	COMPILED BY DKB
DATUM Geodetic	DATE Nov.9/00		CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
95.8	GROUND SURFACE				
0.0	Clayey Silt Topsoil Very stiff Brown Moist	1	SS	20	
95.0 / 0.8	Clayey Silt to Silty Clay with sand, trace to some gravel, occasional cobbles Hard Brown Moist (Till)	2	SS	37	
	Cobble at 2.4m depth.	3	SS	61	
	Silty sand layer from 2.8m to 3.6m depth.	4	SS	30/0.0	
		5	SS	100/.15	
		6	SS	100/.15	
		7	SS	105/.15	4 38 44 14
89.7 / 6.1	Silty Clay, trace sand and gravel with fine sand partings Hard Grey Moist (Till)	8	SS	75/.15	
		9	SS	75/.08	
		10	SS	85/.15	
		11	SS	116	0 9 38 53
81.4 / 14.3	Dark grey SHALE BEDROCK. (Whitby Formation)				

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

Elevation scale: 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 00-16

METRIC

PROJECT	001-8019F				
W.P.	124-99-00	LOCATION	N 4857080.0; E 344047.0	ORIGINATED BY	PKS
DIST	6 HWY 401	BOREHOLE TYPE	108mm Solid Stem Augers	COMPILED BY	DKB
DATUM	Geodetic	DATE	Nov.9/00	CHECKED BY	ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION — CONTINUED FROM PREVIOUS PAGE —	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
80.5			12	SS	100/.08						
15.3	Dark grey, moderately to slightly weathered, fine grained, thinly bedded SHALE bedrock (Whitby Formation), slightly carbonaceous with many very thin carbonaceous (limestone) lenses. Bedrock cored from 15.3m to 18.3m. For bedrock coring details refer to Record of Drillhole 16.						80 79 78				
77.5 18.3	END OF BOREHOLE Note: 1. Water level measured in piezometer at 1.3m depth (Elev.94.5m) on Dec.27/00. 2. Water level measured in piezometer at 1.2m depth (Elev.94.6m) on Feb.9/01.										

ON_MOT 001-8019.GPJ ON_MOT.GDT 15/5/01

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: 00-16

PROJECT: 001-8019F

SHEET 1 OF 1

LOCATION: N 4857080.0; E 344047.0

DRILLING DATE: November 10, 2000

DATUM:

INCLINATION: -90°

AZIMUTH: ---

DRILL RIG: Bombadier D-50

DRILLING CONTRACTOR: Master Soils

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous Page		80.50												
16 17 18	Rotary	Dark grey, moderately to slightly weathered, fine grained, thinly bedded SHALE bedrock (Whitby Formation), slightly carbonaceous with many very thin carbonaceous (limestone) lenses <1mm thick. Moderately closely bedded 15.3m-16.2m, closely bedded 16.2m-18.3m.		15.26 77.47 18.29	1 2								PL PL PL PL			
		END OF HOLE														

DEPTH SCALE 1 : 50

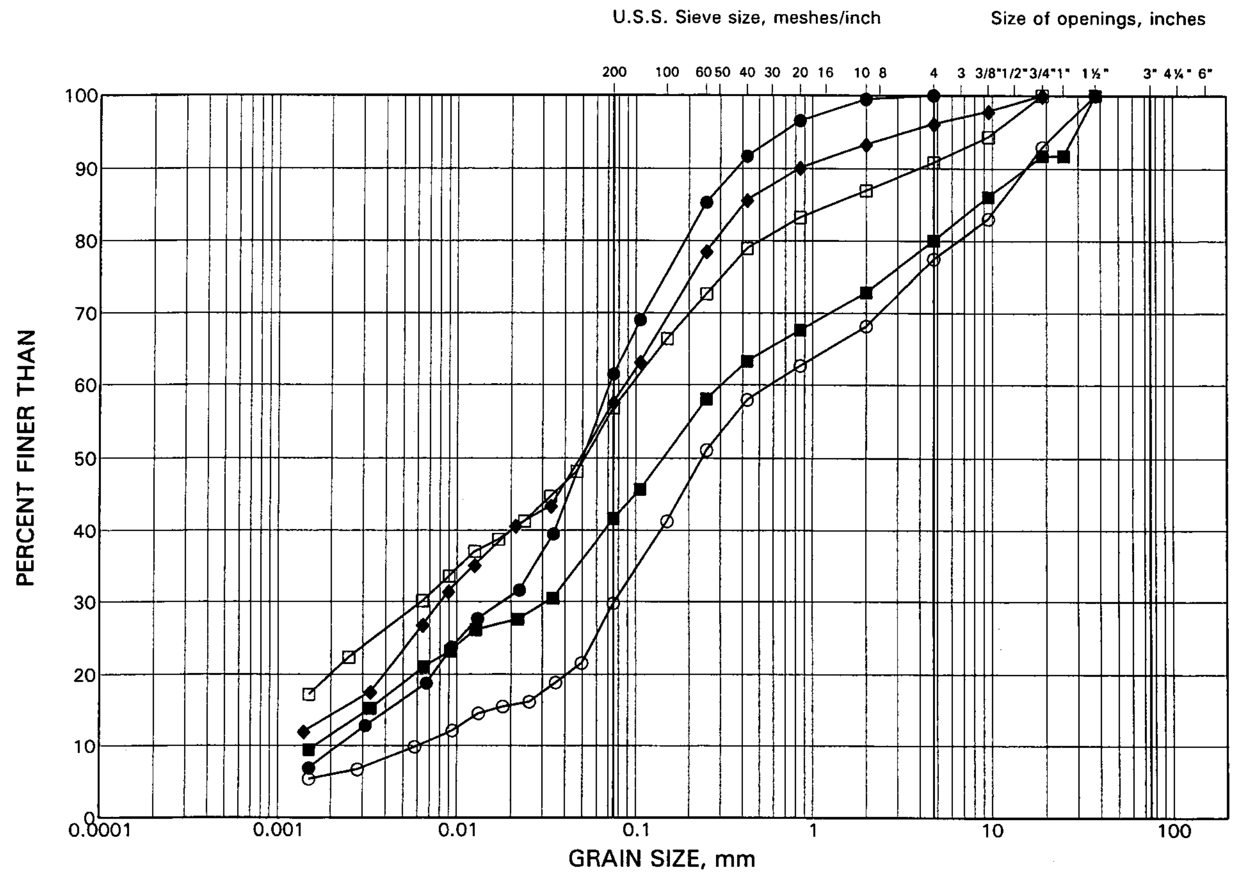
Golder Associates

LOGGED: PKS

CHECKED:

GRAIN SIZE DISTRIBUTION
Upper Clayey Silt / Silty Clay (Till)

FIGURE 1



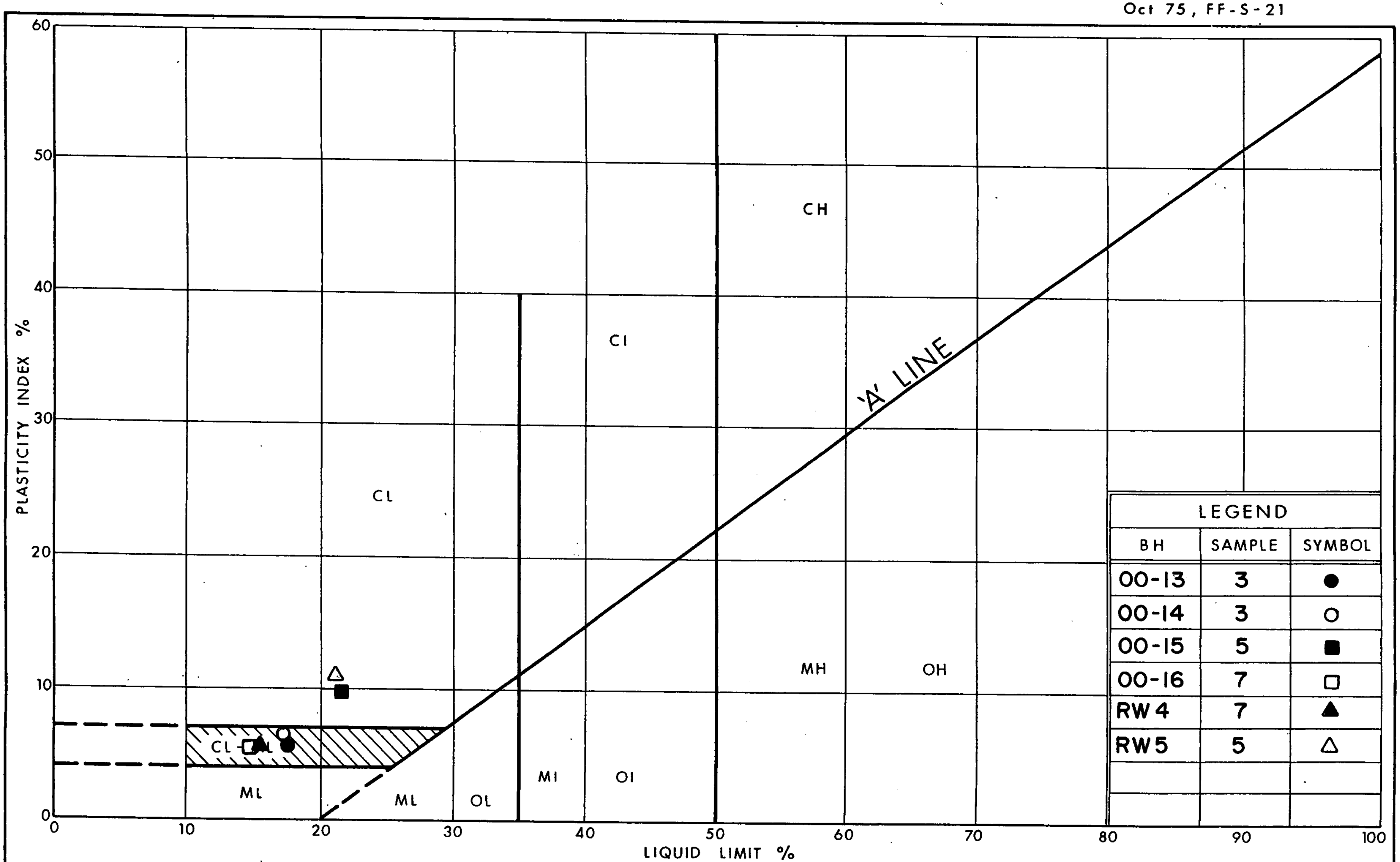
SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	00-13	3	93.0
■	00-15	5	92.5
◆	00-16	7	90.9
○	RW4	5	91.3
□	RW5	5	91.5

Golder Associates

Oct 75, FF-S-21



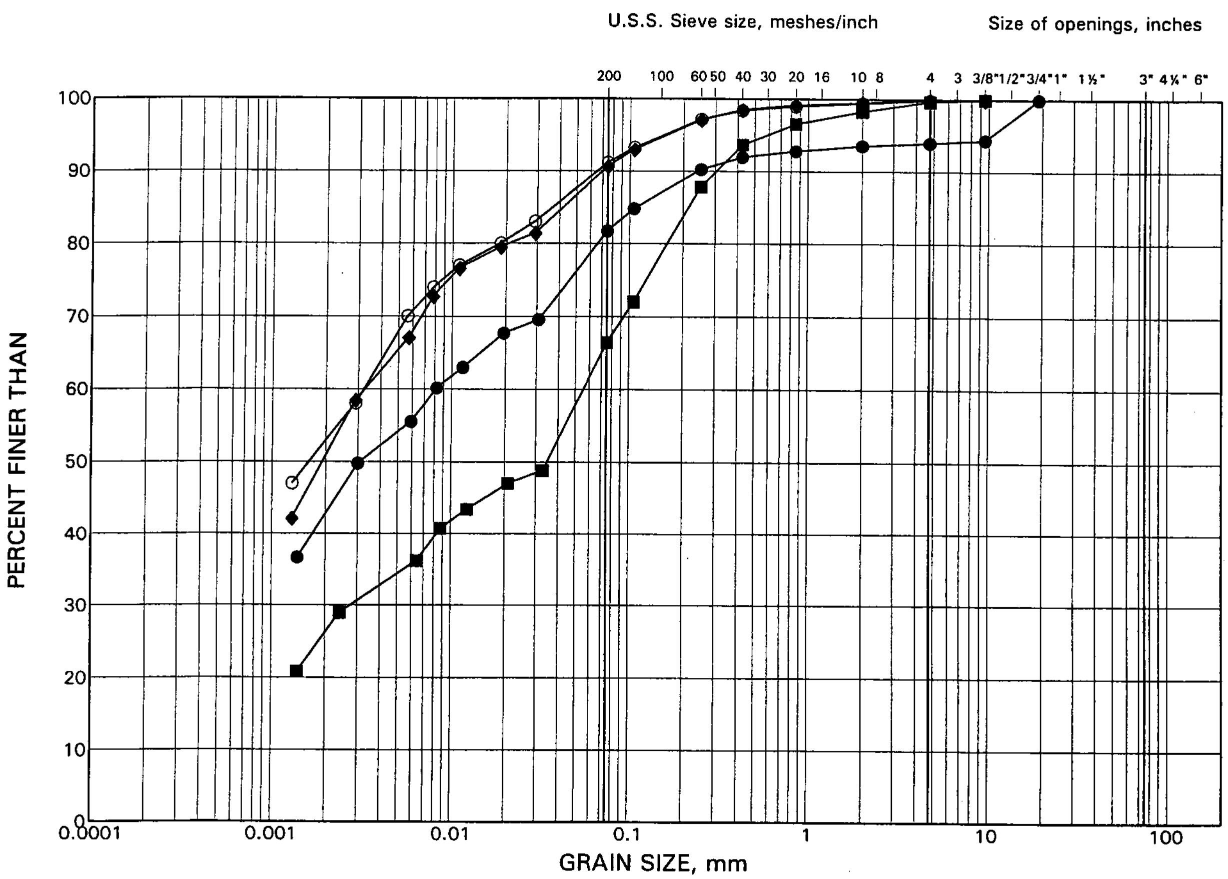
Ministry of Transportation
Ontario

PLASTICITY CHART
UPPER CLAYEY SILT / SILTY CLAY (TILL)

FIG No	2
W P	124-99-00

GRAIN SIZE DISTRIBUTION
Lower Silty Clay to Clay (Till)

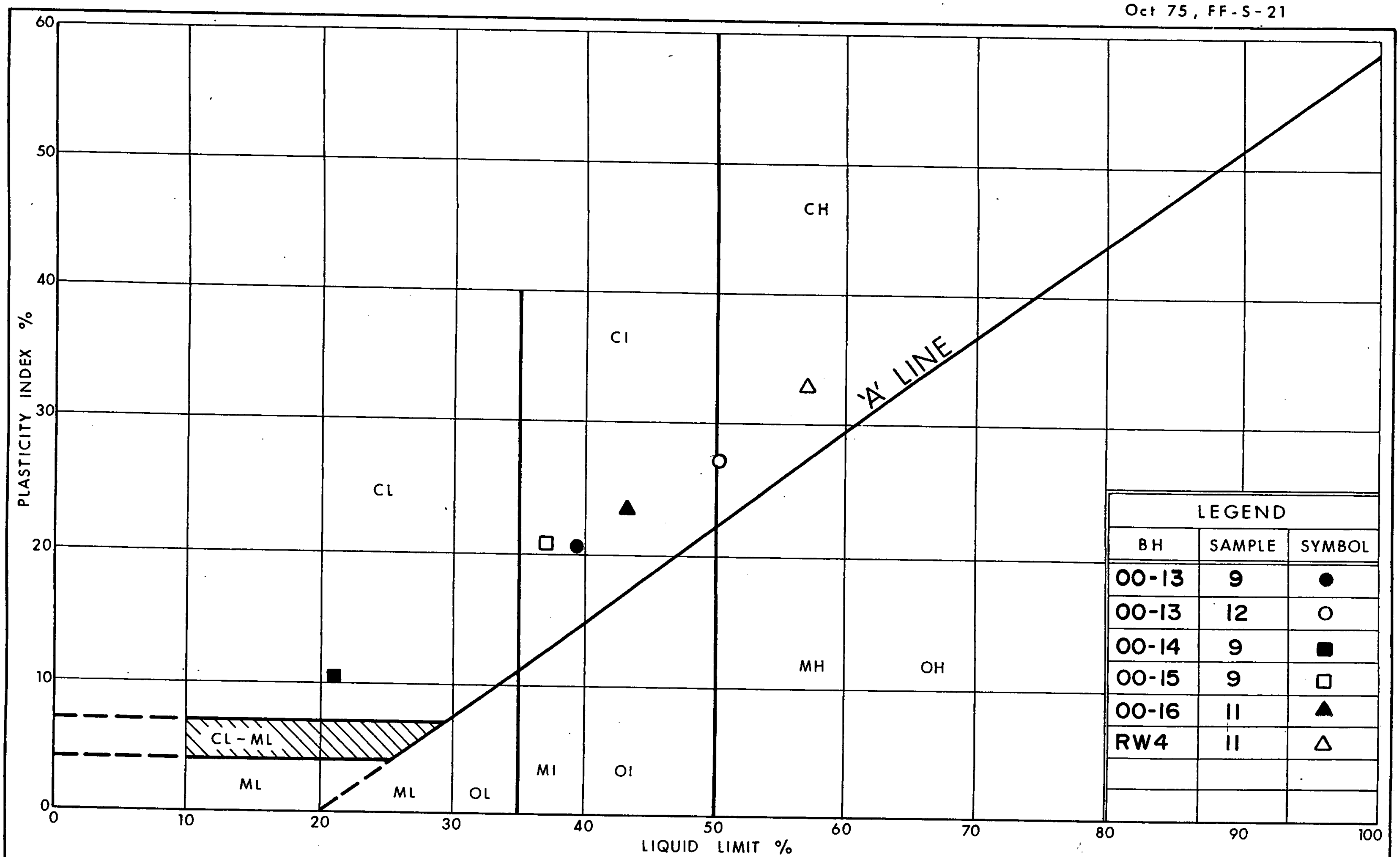
FIGURE 3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	00-13	9	86.8
■	00-14	9	87.1
◆	00-15	9	87.9
○	00-16	11	83.2



BH	SAMPLE	SYMBOL
00-13	9	●
00-13	12	○
00-14	9	■
00-15	9	□
00-16	11	▲
RW4	11	△

Ministry of Transportation
Ontario

PLASTICITY CHART
LOWER SILTY CLAY TO CLAY
(TILL)

FIG No	4
W P	124-99-00

APPENDIX A

RECORD OF BOREHOLE SHEETS (RW-4 AND RW-5) FROM GOLDER ASSOCIATES REPORT 991-1158 DATED JANUARY 2000

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No RW-4

METRIC

PROJECT 991-1158
W.P. 242-86-00 LOCATION N 4857115.00; E 344023.00 ORIGINATED BY SB
DIST 6 HWY 401 BOREHOLE TYPE CME 55 Bombardier COMPILED BY AMP
DATUM Geodetic DATE 16/09/1999 CHECKED BY AMP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.7	GROUND SURFACE				
0.0	Silty Clay, some sand, trace gravel Hard Brown (Fill)	1	50 DO	33	
94.0 0.7	Clayey Silty Sand with gravel Very dense Grey (Till)	2	50 DO	149	
		3	50 DO	100/.13	
		4	50 DO	100/.13	
		5	50 DO	141	23 47 24 6
		6	50 DO	100/.15	
		7	50 DO	100/.08	
		8	50 DO	100/.15	
		9	50 DO	100/.10	
86.3 8.4	Silty Clay to Clay, some sand, trace gravel Hard Grey (Till)	10	50 DO	100/.13	
		11	50 DO	64	
		12	50 DO	100/.13	
82.2 12.5	END OF BOREHOLE Note: Open borehole dry on completion of drilling.				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

ON_MOT 991-1158.GPJ ON_MOT.GDT 15/2/01

RECORD OF BOREHOLE No RW-5

METRIC

PROJECT 991-1158			
W.P. 242-86-00	LOCATION N 4857127.00; E 344062.00		ORIGINATED BY SB
DIST 6	HWY 401	BOREHOLE TYPE CME 55 Bombardier	COMPILED BY AMP
DATUM Geodetic	DATE 16/09/1999		CHECKED BY AMP

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
94.6	GROUND SURFACE				
0.0	Silty Clay, some sand, trace gravel Hard Brown (Fill)	1	50 DO	135	
94.0 / 0.6	Clayey Silty Sand, some gravel Very dense Grey (Till)	2	50 DO	100/.13	
		3	50 DO	100/.13	
		4	50 DO	100/.05	
91.9 / 2.7	Silty Clay with sand, some gravel Hard Grey (Till)	5	50 DO	100/.13	9 34 37 20
91.1 / 3.5	Clayey Silty Sand, some gravel Very dense Grey (Till)	6	50 DO	100/.08	
		7	50 DO	101	
		8	50 DO	104	
		9	50 DO	101	
85.8 / 8.8	Silty Clay, some sand, trace gravel Hard Grey (Till)	10	50 DO	165	
		11	50 DO	100/.15	
		12	50 DO	155/.23	
81.2 / 13.4	Weathered, dark grey Shale Bedrock. (Whitby Formation)				

ELEVATION SCALE: 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No RW-5

METRIC

PROJECT 991-1158			
W.P. 242-86-00	LOCATION N 4857127.00; E 344062.00		ORIGINATED BY SB
DIST 6	HWY 401	BOREHOLE TYPE CME 55 Bombardier	COMPILED BY AMP
DATUM Geodetic	DATE 16/09/1999		CHECKED BY AMP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
79.3											
15.2	END OF BOREHOLE Note: Water level in piezometer at Elev.90.5m on Oct.19, 1999.						79				

ON_MOT 991-1158.GPJ ON_MOT.GDT 15/2/01

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

DIST. 6 HWY. 401
CONT No.
WP No. 124-99-00

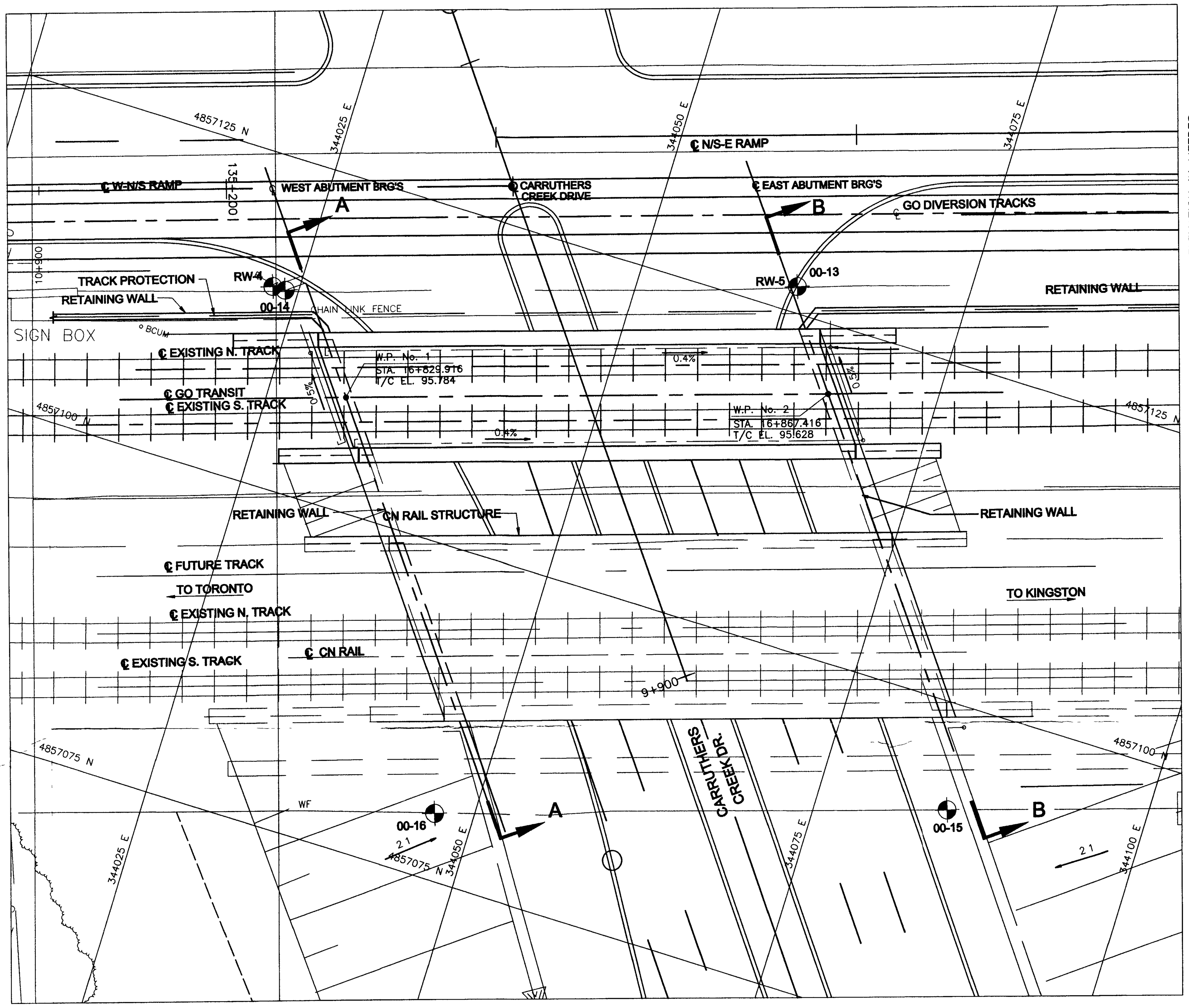
CARRUTHERS CREEK DRIVE GO TRANSIT SUBWAY

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

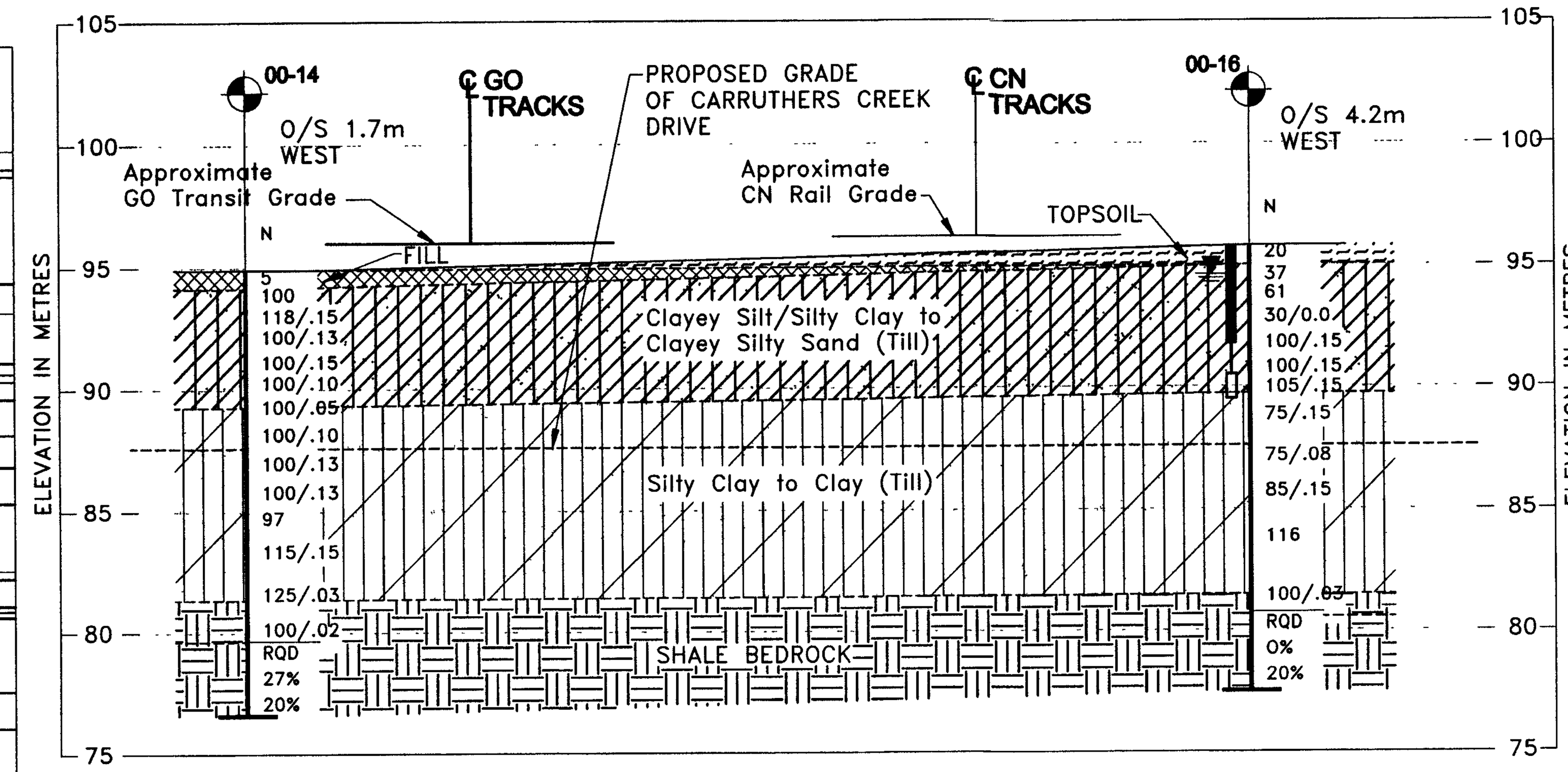
Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



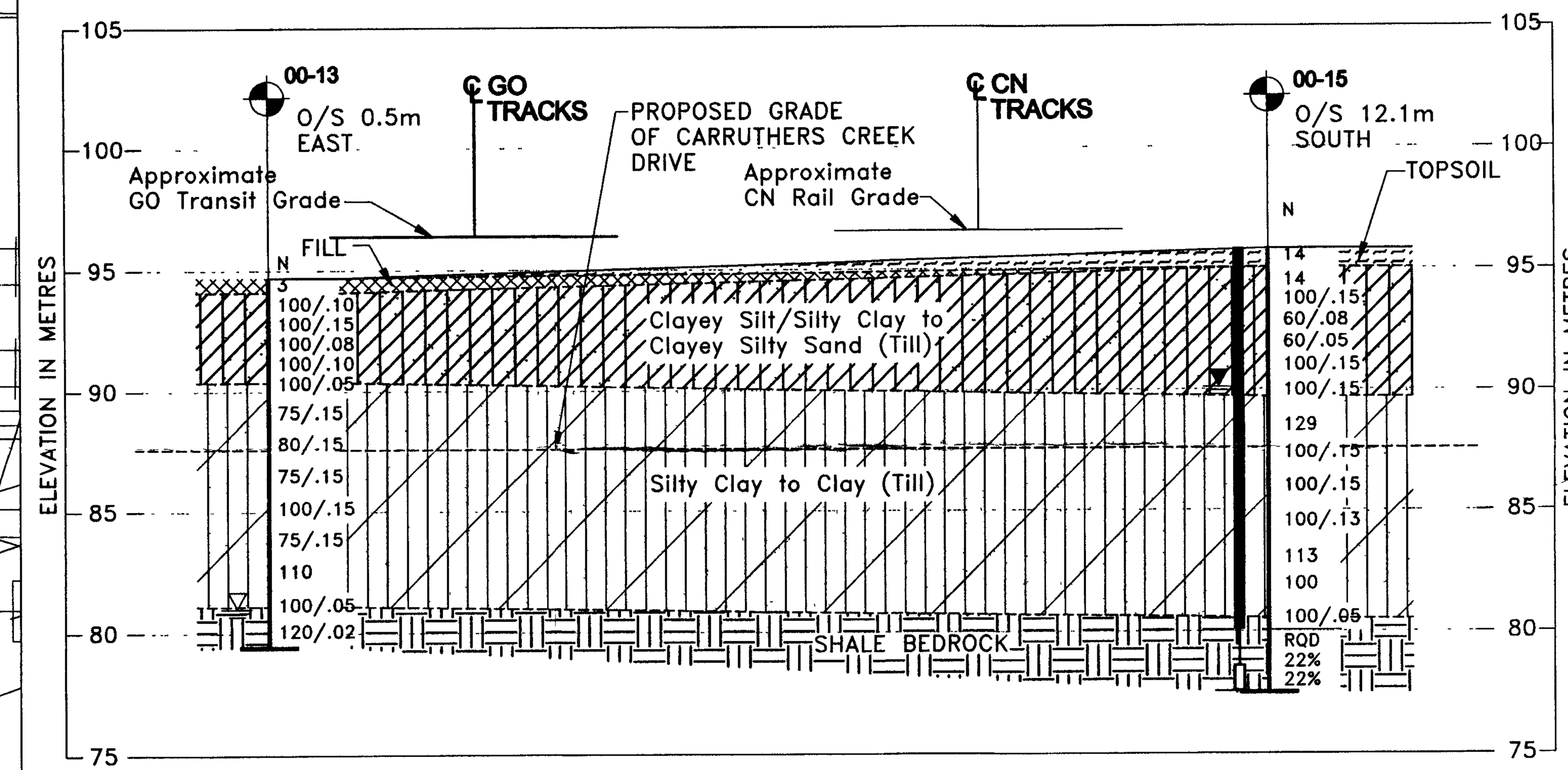
PLAN

0 2 4 6 8 10 15 20 25m
SCALE



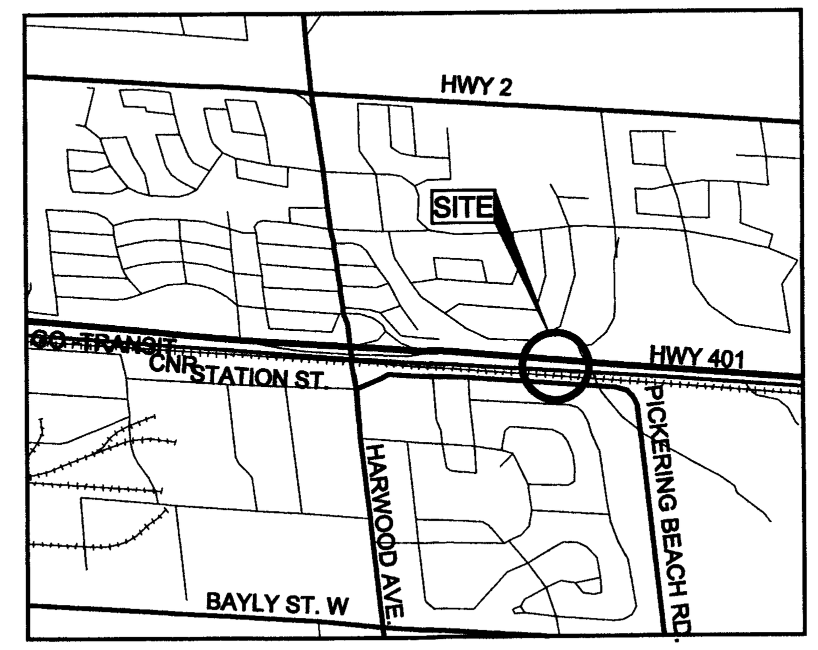
SECTION A-A

0 2 4 6 8 10 15 20 25m
SCALE



SECTION B-B

0 2 4 6 8 10 15 20 25m
SCALE



KEY PLAN

LEGEND

- Borehole
- Probehole
- Seal
- Piezometer
- N — Standard Penetration Test value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, February 9, 2001
- WL upon completion of drilling

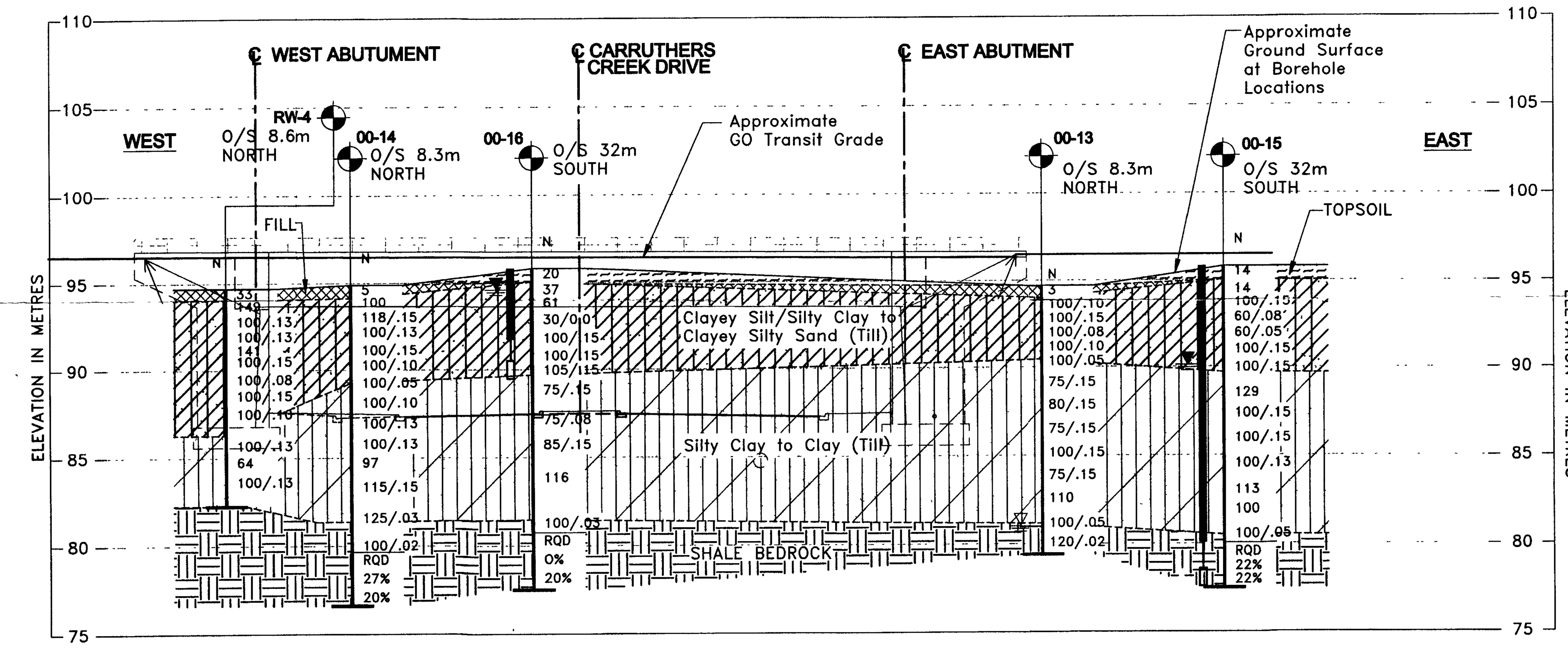
No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
00-13	94.7	4857127.0	344062.0
00-14	94.9	4857115.0	344024.0
00-15	95.8	4857092.0	344085.0
00-16	95.8	4857080.0	344047.0
RW-4	94.7	4857115.0	344023.0
RW-5	94.6	4857127.0	344062.0

NOTES

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

REFERENCE

This drawing was created from digital file "S1.dwg" Titled "CARRUTHERS CREEK DRIVE GO TRANSIT SUBWAY GENERAL ARRANGEMENT" provided by Totten Sims Dated January 3, 2001.



PROFILE ALONG CENTRELINE GO TRANSIT

0 2 4 6 8 10 15 20 25m
SCALE

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

Geocres No.

HWY. 401		PROJECT NO. 001-8019F-3	DIST. 6
SUBM'D. SEP	CHKD. ASP	DATE: MAY 2001	SITE:
DRAWN: JFC	CHKD. SEP	APPD.	DWG. 1

P8019F-3001.DWG
1"=1" PS (1:200 MS)