

**GEOTECHNICAL INVESTIGATION
PROPOSED HIGHWAY 400 UNDERPASS
HIGHWAY 400 AND VAUGHAN MILLS BOULEVARD
CITY OF VAUGHAN, ONTARIO
FOR
THE MILLS CORPORATION
c/o THE AFFINITY CORPORATION**

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May 1999

Peto MacCallum Ltd.
C O N S U L T I N G E N G I N E E R S

May 12, 1999

PML Ref: 99TF023

The Mills Corporation
c/o Mr. Mark Walk
The Affinity Corporation
59 W. Seegers Road
Arlington Heights
Illinois, USA 60005

Dear Mr. Walk

Geotechnical Investigation
Proposed Highway 400 Underpass
Highway 400 and Vaughan Mills Boulevard
City of Vaughan, Ontario

We are pleased to present the results of the geotechnical investigation recently completed at the above-referenced site. Authorization for the assignment was given verbally by Mr. Mark Walk of The Affinity Corporation on behalf of The Mills Corporation on February 22, 1999. A Peto MacCallum Ltd. Engineering Service Agreement dated April 16, 1999 is to be signed and returned.

The project involves the design and construction of a two span structure over the existing Highway 400 to carry the traffic of the contemplated Vaughan Mills Shopping Mall.

The report provides details of the field and laboratory testings together with geotechnical comments and recommendations regarding design and construction of foundations, abutments, approaches and embankments.

Soil testing and sampling at the proposed location of center pier in the median of Highway 400 was not within our scope of service in this investigation.

The subsurface stratigraphy underlying the proposed east and west abutment locations, as revealed in the boreholes, comprises a surficial topsoil layer overlying a discontinuous fill layer which was in turn underlain by silty clay till, clayey silt till and sand deposits.

The water level readings in the piezometers indicated that the sand in the east and west boreholes at a depth of 11.4 and 13.1 m below existing grade, respectively, was under artesian pressure.

It is understood that integral abutment foundations are the preferred foundation alternatives for this project. The subsurface conditions are generally adequate for integral abutments or for footings on engineered fill.

Construction of integral abutments supported on steel H-piles driven through the approach fill is feasible, based on the results of the investigation. Supporting the proposed structure on conventional spread footings founded on the native overburden at nominal depth is not considered to be economical, due to low bearing capacity available in the upper level of the overburden. Alternatively, spread footings could be constructed on structural fill placed as part of the approach embankment. Recommended footing and pile bearing capacities in Ultimate Limit State and Serviceability Limit State are provided in the report.

It is understood that the height of fill to be placed on the approaches on both sides of the proposed underpass is about 9 to 10 m. Based on the information revealed in the boreholes, the anticipated subgrade will consist fill and native firm to very stiff silty clay till. No settlement or bearing capacity problems due to placing approach fill on the inorganic native soils are anticipated within about 20 m of the abutment locations, providing that all soft and deleterious materials are excavated.

Settlement of the fill embankment caused by the consolidation of the fill material under its own weight should be minimized by strict placement and compaction controls.

For embankment construction, the earth fill should be compacted to at least 98% of the standard Proctor maximum dry density to minimize settlement of the embankment fill. Embankment slopes inclined at 2 horizontal to 1 vertical should be stable for the proposed 9 to 10 m height of fill. The recommendation should be reviewed if the fill heights exceed 10 m to verify bearing capacities of the native soils. Measures to control surface runoff and minimize erosion of the embankment slope should be established.

Backfill behind the abutment walls may be accomplished using granular material, such as OPSS Granular "A" or Granular "B", Type I and compacted in thin lifts to at least 98% of the standard Proctor maximum dry density.

For shallow depth of excavation envisaged, the artesian pressure noted should not pose any special problems during excavation. No major groundwater problems are anticipated during construction. Any seepage, which may occur from precipitation, from water which may be perched in the fill, or from more pervious seams in the native deposits can be controlled by pumping from temporary sumps.

In view of variable depths of pile founding soils at the proposed east and west abutment locations (20 and 26 m below existing grade, respectively), supplementary investigations should be carried out to determine the subsurface condition at the proposed location of centre pier in the median of Highway 400 and along the proposed road embankment alignments. Variations of pile length within the same abutment and wing wall foundations should also be expected.

Proposed Highway 400 Underpass
The Mills Corporation

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May 12, 1999

We trust that this report has been completed within our terms of reference. If you have any questions regarding the information presented, or when we may be of service during construction phase of the project, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



Brian R. Gray, M.Eng., P.Eng.
Vice President
Geotechnical Engineering and
Geo-Environmental Services

BRG:lr

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List of Abbreviations

Log of Borehole Sheets

Drawing 1: Borehole Location Plan

Figures 1 and 2: Grain Size Distribution Analyses Charts

Figure 3: Plasticity Chart

Figure 4: Abutment on Compacted Fill Showing Granular "A" Core

1. INTRODUCTION

Peto MacCallum Ltd. was retained by The Mills Corporation to carry out a geotechnical investigation for the proposed Highway 400 underpass at Vaughan Mills Boulevard in the City of Vaughan, Ontario.

The project involves the design and construction of a two span structure over the existing Highway 400. The proposed structure will carry the traffic for the contemplated Vaughan Mills Shopping Mall. The proposed structure will run roughly east-west and connect to the future Vaughan Mills Boulevard.

Road grade on Vaughan Mills Boulevard over the proposed structure will be near elevation 219.7, some 9 to 10 m above the existing ground surfaces and 8 m above the road grades of the existing Highway 400. These estimated grades were based on a site plan provided by McCormick Rankin Corporation on March 14, 1999, titled "Vaughan Mills Boulevard, Highway 400 Underpass, General Arrangement", dated March 1999 and existing ground surface elevations determined at the borehole locations.

The purpose of the investigation was to determine the subsurface soil and groundwater conditions underlying the proposed east and west abutment locations, and based on the factual data, provide geotechnical recommendations pertaining to the design and construction of the foundation, abutment walls and approach embankments.

Soil testing and sampling at the proposed location of the center pier in the median of Highway 400 was not within our scope of service in this investigation.

2. INVESTIGATION PROCEDURE

The fieldwork for this investigation was carried out between March 25, and April 6, 1999 and comprised underground services stake-out, the drilling of two boreholes, local survey and reading of water levels in the borehole piezometers. The two sampled boreholes were numbered 400-1 and 400-2 and were drilled to depths of 28.1 and 21.7 m below existing grades at the approximate locations shown on the appended Borehole Location Plan (Drawing No. 1).

Boreholes 400-1 and 400-2 were drilled within the proposed west and east abutment of the structure, respectively.

The boreholes were advanced using a track-mounted CME 55 track mounted drill rig equipped with continuous flight solid stem augers supplied and operated by a specialist drilling contractor under the full-time supervision of a member of our engineering staff.

Representative samples of the overburden were recovered at frequent depth intervals using a conventional split soon sampler in conjunction with standard penetration tests. In addition, dynamic cone penetration tests were carried out in borehole 400-1 to further assess the relative densities of the saturated sand subsoils encountered.

The groundwater conditions in the boreholes were closely monitored during and upon completion of drilling. A piezometer was installed in each of the two boreholes for subsequent water level readings.

The locations and ground surface elevations at the boreholes were established in the field by Peto MacCallum Ltd. Ground surface elevations at the borehole locations were referred to the following temporary benchmark:

T.B.M.: Ground surface elevation of borehole 214 drilled
by Peto MacCallum Ltd. in November, 1998

Elevation: 212.61 (metric, geodetic)

The existing borehole elevation was provided by Bennett Young Limited, Reference W.O. 9828751, RW, D; dated November 5, 1998.

The recovered soil samples were brought to Peto MacCallum Ltd. laboratory for detailed visual examination and water content determination. Selected samples were subjected to Atterberg limits tests, grain size distribution analyses and hydrometer tests.

3. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the Log of Borehole sheets for details of the field work including soil classifications, inferred stratigraphy, standard penetration test "N" values, dynamic cone penetration test values, the results of laboratory water content determinations, together with groundwater observations in the open boreholes and installed piezometers.

The results of grain size distribution and hydrometer tests conducted on selected samples recovered during drilling are presented on Figures 1 and 2, and noted on the Log of the Boreholes sheets.

The results of the Atterberg limit test conducted on a representative sample of the cohesive deposit is provided on the plasticity chart (Figure 3), and noted on the Log of Borehole sheet.

3.1 Soil Stratigraphy

The subsurface stratigraphy revealed a surficial topsoil layer overlying a discontinuous fill layer which was in turn underlain by silty clay till, clayey silt till and sand deposits.

A 200 and 350 mm thick surficial topsoil layer was encountered in boreholes 400-1 and 400-2, respectively. The topsoil comprised dark brown clayey silt with rootlets and organic inclusions.

Fill, about 0.9 m thick, was contacted to a 1.2 m depth below the topsoil at the east abutment in borehole 400-2. The fill comprised sandy silt and clayey silt with rootlets and topsoil inclusions. The fill was loose, based on standard penetration test "N" values of 10 and 4 blows per 300 mm penetration. The water contents of the fill samples were 20 and 23%.

An upper silty clay till deposit was encountered below the topsoil in the west borehole 400-1 at a depth of 0.2 m (elevation 210.9) and the fill in the east borehole 400-2 at a depth of 1.2 m (elevation 208.6) and extended to a depth of 8.4 and 3.8 m (elevations 202.7 and 205.9), respectively. The silty clay till comprises grey silty clay with trace sand and trace gravel. It was firm to hard, based on standard penetration test "N" values of 7 to 32 blows per 300 mm penetration. The water content of the silty clay till samples typically varies from 22 to 12%, indicating drier to wetter than the estimated plastic limit.

Very stiff to hard clayey silt till was encountered below the silty clay till in boreholes 400-1 and 400-2 extending to depths of 11.4 and 13.1 m (elevation 199.7 and 196.7), respectively. Standard penetration test "N" values ranged from 24 to 46 blows per 300 mm penetration. The water content of the clayey silt till samples typically varied from 10 to 20%, indicating drier than to at the estimated plastic limit.

Sand was encountered below the clayey silt till in boreholes 400-1 and 400-2 at depths of 11.4 and 13.1 m (elevations 199.7 and 196.7) and extended to a depth of 25.0 and 18.3 m (elevations 186.0 and 191.5), respectively. The sand was not fully penetrated in borehole 400-1 in this investigation but was inferred and investigated to about 28.0 m, elevation 183.0, by means of a dynamic cone penetration test.

Results of the grain size distribution analysis of a sand sample is presented on Figure 1.

The sand in the east borehole 400-2 was compact, based on standard penetration test "N" values of 14 to 26 blows per 300 mm penetration. The sand in the west borehole 400-1 was compact to a depth of 17.5 m, dense to very dense below. Standard penetration test "N" values of 24 to 66 blows per 300 mm penetration were recorded at a depth intervals between 11.4 and 19.5 m.

Low "N" values of 17 and 19 blows recorded at a depth of 21 and 24 m below existing grade were due to groundwater disturbance. Dynamic cone penetration tests were carried out from 25.0 m in the same borehole and from 21.5 to 24.5 m in a supplementary borehole drilled 2 m to the south of borehole 400-1 to further assess the relative density of the sand. The dynamic cone test penetration resistance was in the range of 8 to 65 blows per 300 mm penetration between 21.5 and 27.4 m, to greater than 125 blows per 300 mm penetration below a depth of 27.4 m (elevation 183.7). Low cone test "N" values less than 30 blows recorded between 21.5 and 27.4 m were also due to groundwater disturbance. Allowing for groundwater disturbance, it is considered that the sand was in dense to very dense condition at depth. The dynamic cone test was terminated by practical refusal at a depth of 28.1 m below existing grade.

A lower hard silty clay till deposit was encountered below the sand in east borehole 400-2 at a depth of 18.3 m (elevation 191.4) and extended to the termination depth of the borehole at 21.7 m (elevation 188.1). Standard penetration test "N" values were 43 blows per 300 mm penetration to greater than 60 blows per 150 mm penetration below a depth of 19.8 m (elevation 189.9).

Results of the grain size distribution analysis of a silty clay till sample is presented on Figure 2. The liquid and plastic limit of silty clay till was 20 and 12, based on the result of Atterberg limit test. The computed plasticity index was 8. Based on the results of the Atterberg limit test, the silty clay till can be classified as a "CL" soil. The water content varied from 10 to 14%, indicating drier than the estimated plastic limit.

3.2 Groundwater

During drilling, free water was encountered in the sand deposit at a depth of 11.4 m (elevation 199.7) in borehole 400-1. After sample 16 (at a depth of 19.8 m, elevation 201.3), the sand was back up to 9.8 m. After sample 17 (at a depth of 21.3 m, elevation 189.8), the sand was back up to 18.3 m. No free water was encountered in borehole 400-2, during drilling.

On completion of drilling, water level was measured at a depth of 1.2 m (elevations 209.9 and 208.6) in the piezometers installed in both boreholes. A piezometer was installed in the sand deposit in each borehole for subsequent water level readings. Water level was measured at depths of 5.6 and 7.4 m (elevations 205.5 and 202.4) in the piezometer installed in the east and west boreholes 400-1 and 400-2, respectively, 5 and 6 days after the completion of drilling. The water level readings indicated that the sand in borehole 400-1 and 400-2 at a depth of 11.4 and 13.1 m, respectively, was under artesian pressure.

The groundwater levels at the site are subject to seasonal fluctuations and rainfall patterns.

4. **ENGINEERING DISCUSSION AND RECOMMENDATIONS**

The project involves the design and construction of a two span structure over the existing Highway 400 to carry the traffic for the future Vaughan Mills Boulevard. Road grade on the proposed structure will be near elevation 219.7, some 9 to 10 m above the existing ground surfaces and 8 m above the road grades of the existing Highway 400.

The subsurface stratigraphy revealed a surficial topsoil layer overlying a discontinuous fill layer which was in turn underlain by silty clay till, clayey silt till and sand deposits. Hard silty clay till and very dense sand were encountered in the proposed east and west abutment location (boreholes 400-2 and 400-1) at a depth of 18.3 and 17.5 m below existing grade, elevations 191.5 and 193.6, respectively.

The water level readings in the piezometers indicated the sand in the east borehole 400-1 and west borehole 400-2 at a depth of 11.4 and 13.1 m below existing grade, respectively, was under artesian pressure.

4.1 Foundations

It is understood that integral abutment foundations are the preferred foundation alternatives for this project. The subsurface conditions are generally adequate for integral abutments or for footings on engineered fill. These alternatives are discussed on the following paragraphs.

4.1.1 Integral Abutments on Piles

Construction of integral abutments supported on steel H-piles driven through the approach fill is feasible, based on the results of the investigation. The recommended axial resistance for HP 310 x 110 piles driven to an adequate set in the very dense sand/hard silty clay till are as follows:

- Factored Axial Resistance at ULS = 1600 kN (east and west abutments)
- Axial Resistance at SLS = 1100 kN (east and west abutments)

The resistance at the ultimate limit state was computed by applying a geotechnical resistance factor of 0.6 to the factored structural resistance of the pile section. A yield strength of 300 MPa is assumed for the steel. The capacity at serviceability limit states allows for 25 mm of compression of the pile and founding medium at full design load.

It is anticipated that the recommended resistance will be achieved by driving the H-piles to about elevation 190 and 185 in the east and west abutment area, respectively. The axial resistance should be confirmed by the Hiley formula or dynamic analysis assuming an ultimate capacity of 3000 and 3200 kN for piles driven in the east and west abutment area, respectively.

Provision should be made to retap the initial piles to confirm the set after adjacent piles have been driven. If it is found that the set of piles is not altered by retapping then this procedure may be discontinued.

The installation operations should be inspected on a full-time basis by qualified geotechnical personnel to confirm the toe elevation, driving resistance, alignment, plumbness, uniformity of set, and quality of splices.

Although cobbles and boulders were not encountered in the exploratory boreholes, their presence should not be overlooked. Accordingly, driving shoes should be provided to minimize the potential for damage when driving through native deposits.

Pile caps should be provided with at least 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

The soil adjacent to the upper portion of the piles is expected to comprise well compacted approach fill placed directly on proposed subgrade. To accommodate movement of the integral abutments, it is recommended that an annular space be provided around the pile to a depth of 3 m below the bottom of the abutment pile cap. The space should be created by pre-augering and placement of two concentric corrugated steel pipes (CSPs) around the pile, or by incorporating the CSPs in the embankment fill. The inner CSP around the pile should be filled with a cement-bentonite grout; the outer CSP should be 600 mm diameter. The piles must be driven to an adequate depth below the flexible zone to ensure stability.

The coefficient of horizontal subgrade reaction, K_s , for Granular "B" backfill or native sand may be computed using the following equation to evaluate the point of contraflexure:

$$K_s = n_h z/b$$

Where

$$z = \text{depth (m)}$$

$$b = \text{pile depth (m)}$$

$$n_h = \text{constant related to soil density}$$

The recommended values for n_h are as follows:

	n_h (kN/m ³)	
	Above Groundwater	Below Groundwater
Granular "B": Backfill	12,000	8,000
Dense Sand	18,000	11,000

For native silty clay till and clayey silt till, a K_s value of 20 MPa/m may be used.

Resistance to lateral loads may be provided in part of mobilization of passive resistance along the pile below the annular space. The lateral resistances recommended for HP 310 x 110 piles are as follows:

- Factored Axial Resistance at ULS = 130 kN
- Lateral Resistance at SLS = 65 kN

4.1.2 Spread Footings

The results of the investigation indicated that relatively low bearing capacity was available in the upper overburden soils. Supporting the proposed structure on conventional spread footings founded on the native overburden at nominal depth

may be uneconomical. However, the structure may be founded on engineered fill constructed as part of the approach embankment. These alternatives are discussed in the following paragraphs.

The bearing resistance of a 2.5 m wide footing supported on conventional spread footing founded on the native very stiff silty clay till near elevations 209.3 in west borehole 400-1 and 208.3 in east borehole 400-2 may be proportioned using the following bearing resistances:

- Factored Axial Resistance at ULS = 400 kN
- Bearing Resistance at SLS = 200 kN

Alternatively, spread footings could be constructed on structural fill placed in the approaches. The structural fill should comprise OPSS Granular "A" material placed in maximum 200 mm thick lifts, compacted to 100% standard Proctor maximum dry density, and extended laterally to a line inclined outwards at 1:1 (H:V) originating at least 1 m from top of footing. The scheme is illustrated on Figure 4. The thickness of granular material below the footing should be at least 1.5 m.

The bearing resistances for a minimum 2.5 m wide footing constructed on structural fill are as follows:

- Factored Axial Resistance at ULS = 950 kN
- Bearing Resistance at SLS = 350 kN

The recommended capacity at SLS allows for 25 mm of total settlement; differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS capacity.

If spread footings are employed, the horizontal force will be resisted in part by the friction force developed between the underside of footing and the silty clay till or granular fill. Unfactored friction factors of 0.35 and 0.45 are recommended for footings on very stiff to hard silty clay till and granular fill, respectively.

The design capacities should be reviewed when further details regarding founding levels and abutment locations are established.

All footings subject to frost action should be provided with 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Prior to placement of structural concrete, all founding surfaced must be inspected by geotechnical personnel from Peto MacCallum Ltd. to ensure that the founding soils are similar to those identified in the boreholes and are capable of supporting the design soil bearing pressure.

4.2 Abutment Walls

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall.

Providing the backfill adjacent to the walls is free draining OPSS Granular "A" or Granular "B" Type I or equivalent, there is no hydrostatic pressures built-up behind the walls, the backfill surface is level and the retaining structure does not exceed 10 m in effective height, the lateral earth pressures can be calculated utilizing equivalent fluid pressures.

For walls restrained at the top, an equivalent fluid pressure of 12.0 kPa/m may be considered. For unrestrained structures an equivalent fluid pressure of 8.0 kPa/m may be considered.

Alternatively, the lateral earth, P, may be computed using the following equation, assuming a triangular pressure distribution:

$$P = K (\gamma h + q)$$

Where

K	=	coefficient of lateral earth pressure
γ	=	unit weight of free-draining granular material (kN/m ³)
h	=	depth below final grade (m)
q	=	surcharge load (kPa), if present

Free draining granular material should be used as backfill behind the wall. The following parameters are recommended for design:

Parameters	Granular "A"	Granular "B", Type I
Angle of internal Friction (degrees)	35	32
Unit Weight (kN/m ³)	22.8	21.2
Active Earth Pressure Coefficient (K_a)	0.27	0.31
At Rest Earth Pressure Coefficient (K_o)	0.43	0.47
Passive Earth Pressure Coefficient (K_p)	3.69	3.25

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed to design integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures.

A weeping tile system and/or weeping holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

A retained soil system could also be employed. The founding material is expected to complete a granular engineered fill or very stiff to hard silty clay till. The following parameters should be employed for design of the system foundation:

Parameters	Granular "A"	Native Silty Clay Till
Friction Angle (degrees)	35	0
Cohesion (kPa)	0	75
Unit Weight (kN/m ³)	22.8	20.0

The supplier of the retained soil system should be responsible for design of the structure (backfill, reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations performance level, appearance etc.

4.3 Approach Embankments

It is understood that the height of fill to be placed on the approaches on both sides of the proposed structure will be about 9 to 10 m. Based on the information revealed in the boreholes, the anticipated subgrade comprises shallow fill and native firm to hard silty clay till.

No settlement or bearing capacity problems due to placing approach fill on the inorganic native soils are expected, providing the soft and deleterious materials are excavated. In this regard, the surficial fill in the area of west borehole 400-1 and east borehole 400-2 should be excavated.

Settlement of the fill embankment caused by the consolidation of the fill material under its own weight should be minimized by strict placement and compaction controls.

Topsoil, the existing fill and other obviously unsuitable material should be stripped and excavated prior to placement of approach fill. The exposed subgrade surfaces should be proof rolled with a heavy vibratory compactor under the full time supervision of geotechnical personnel from Peto MacCallum Ltd. Any soft, wet or deleterious materials that becomes evident during proof rolling should be sub-excavated and replaced with approved backfill compacted to 98% standard Proctor maximum dry density. The area can then be brought up to the design subgrade level with suitable and approved materials. The subgrade fill should be placed in lifts not more than 200 mm thick in the loose state, each lift being compacted to at 98% standard Proctor maximum dry density.

Compaction of backfill behind the abutment walls should be carried out using lightweight equipment to prevent possible damage to the walls. The structures should be checked for the need of bracing or lateral supports during the application of compaction.

All backfilling and compaction operations should be monitored by representatives of Peto MacCallum Ltd. to approve material, to evaluate placement operations, and to verify that the specified degree of compaction is being achieved uniformly throughout the fill.

The proposed embankment slopes up to 10 m high inclined at 2 horizontal to 1 vertical should be stable. The recommendations should be reviewed if the fill heights exceed 10 m, to verify bearing capacities of native soils.

Measures to control surface runoff and minimize erosion of the embankment slope should be established.

4.4 Excavation and Groundwater Control

Excavation for construction of footings or pile caps is expected to be carried out primarily within the approach fill, existing surficial fill and native silty clay till.

Excavation of the fill and overburden should be relatively straightforward using conventional equipment. The presence of cobbles and boulders within the overburden should be expected in the till soils and planned for construction.

All construction work should be carried out in accordance with the Occupational Health and Safety Act (OSHA) and local regulations. With respect to the OSHA, the fill materials and firm to stiff silty clay till are considered as Type 3 soils. The very stiff and hard silty clay till is considered as Type 2 soil. If an excavation contains more than one soil type, trench and excavation slope geometry shall be governed by the highest numbered soil type.

Artesian pressure was noted in the sand in the west borehole 400-1 and east borehole 400-2 at a depth of 11.4 and 13.1 m below existing grade, respectively. For shallow depth of excavation envisaged, the groundwater should not pose any special problems during excavation.

No major groundwater problems are anticipated during construction. Any seepage, which may occur from precipitation, from water which may be perched in the fill, or from more pervious seams in the native deposits, can be controlled by pumping from temporary sumps.

5. ANCILLARY CONSIDERATIONS

The recommendations in this report have been based on the findings in the boreholes. Soils conditions may vary between and beyond the boreholes, especially with respect to fill and topsoil conditions. Variations in conditions identified during construction may necessitate design modifications.

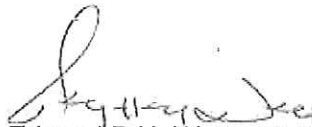
In view of variable depths of pile founding soils at the proposed east and west abutment locations (20 and 26 m below existing grade, respectively), consideration should be given to carry out a supplementary investigation to determine the subsurface condition at the proposed location of center pier in the median of Highway 400.

Environmental considerations, such as fill material chemical quality, were not within the scope of our services related to this investigation. We would be pleased to provide geo-environmental ✓ services regarding the aforementioned issues, if required, and provide the necessary recommendations in compliance with the current regulatory requirements.

We trust you will find this report completed within our terms of reference. If you have any questions regarding the information presented, or when we may be of service during the construction phase of the project, please do not hesitate to contact this office.



Peto MacCallum Ltd.

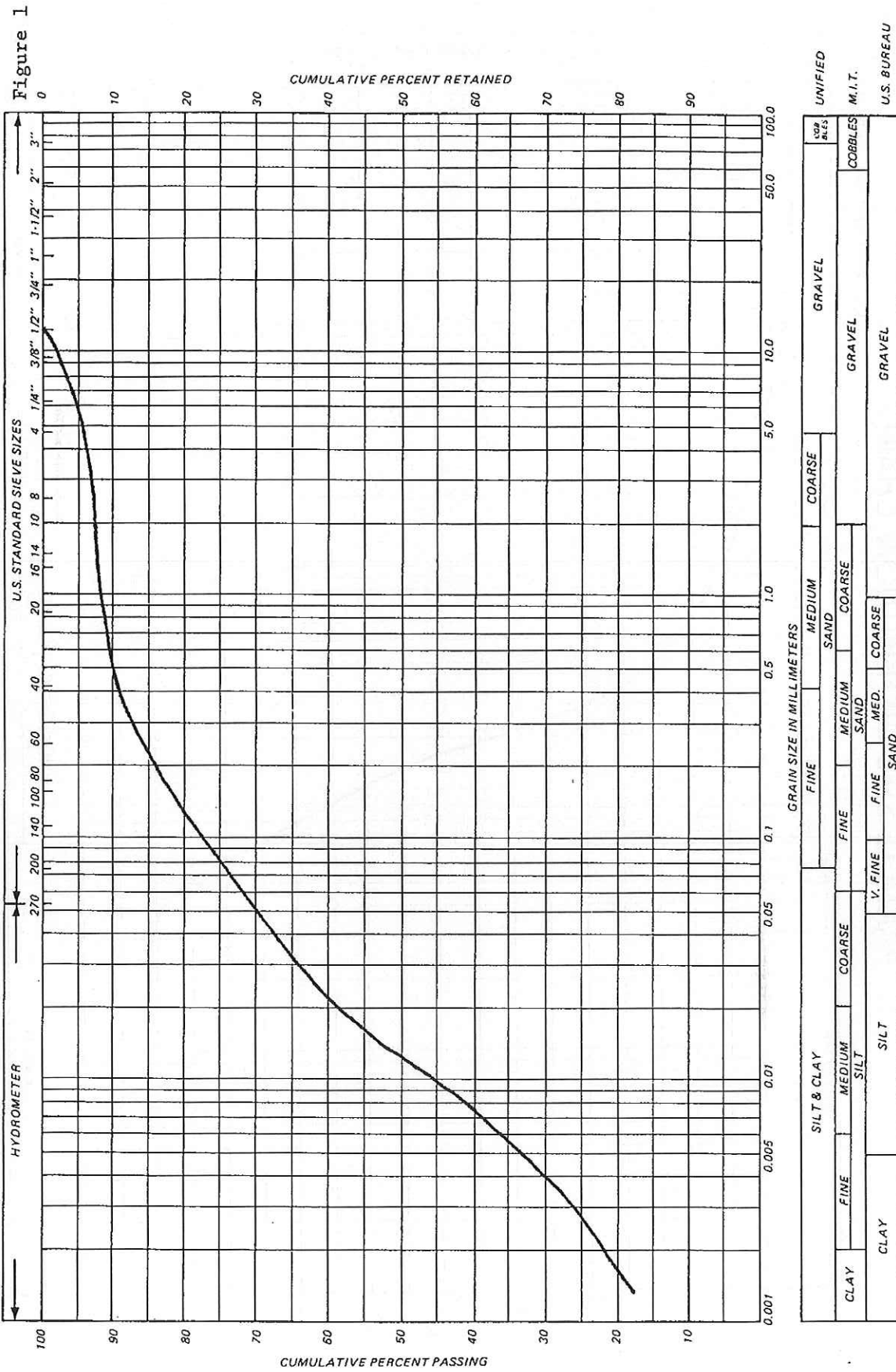

Edward B.H. Wong, M.Eng., P.Eng.
Project Engineer


Carlos M.P. Nascimento, P.Eng.
Senior Consultant

EW/CN:lr

PARTICLE SIZE DISTRIBUTION CHART

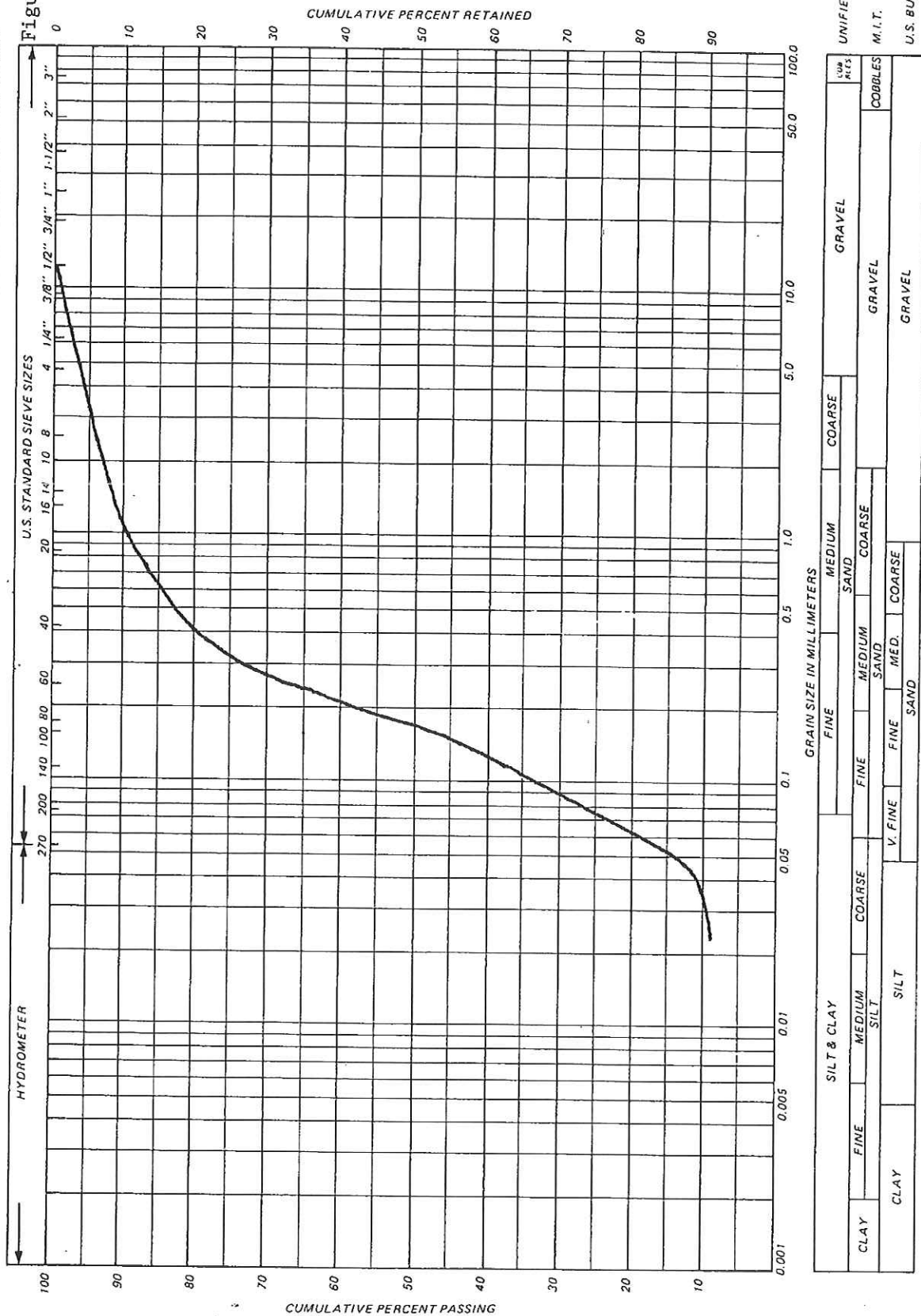
OUR PROJECT NO. 99TF023



REMARKS SILTY CLAY TILL: silty clay with sand, trace gravel CL

Borehole 400-2, Sample 15, Depth 10.3-18.8 m, $W_L = 20$, $W_L = 12$, $PI = 8$

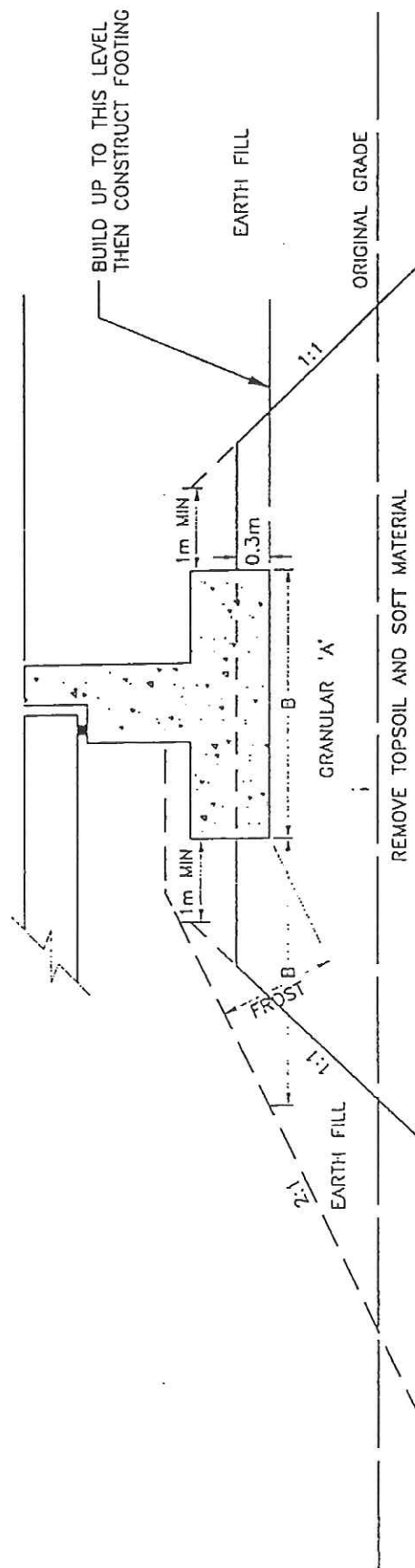
figure 2



REMARKS SAND: sand with silt, trace gravel

Borehole 400-1, Sample 15, Depth 18.3-18.8 m

CROSS SECTION



LONGITUDINAL SECTION

1. REMOVE TOPSOIL AND/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
3. CONSTRUCT CONCRETE FOOTING
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED
5. REFER TO TEXT OF REPORT FOR FROST DEPTH

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DATE	SCALE	JOB NO.	FIGURE NO.
May, 1999	N.T.S.	99TF023	4

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL. DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE: - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS. 0.3m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475J PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS:-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3m</u>
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4
SOFT	2 - 4	12 - 25	LOOSE	4 - 10
FIRM	4 - 8	25 - 50	COMPACT	10 - 30
STIFF	8 - 15	50 - 100	DENSE	30 - 50
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50
HARD	> 30	> 200		
W.T.P.L. WETTER THAN PLASTIC LIMIT			D.T.P.L. DRIER THAN PLASTIC LIMIT	
		A.P.L. ABOUT PLASTIC LIMIT		

TYPE OF SAMPLE

S.S. SPLIT SPOON	T.W. THINWALL OPEN
W.S. WASHED SAMPLE	T.P. THINWALL PISTON
S.B. SCRAPER BUCKET SAMPLE	O.S. OESTERBERG SAMPLE
A.S. AUGER SAMPLE	F.S. FOIL SAMPLE
C.S. CHUNK SAMPLE	R.C. ROCK CORE
S.T. SLOTTED TUBE SAMPLE	
P.H. SAMPLE ADVANCED HYDRAULICALLY	
P.M. SAMPLE ADVANCED MANUALLY	

SOIL TESTS

Qu UNCONFINED COMPRESSION	L.V. LABORATORY VANE
Q UNDRAINED TRIAXIAL	F.V. FIELD VANE
Qcu CONSOLIDATED UNDRAINED TRIAXIAL	C CONSOLIDATION
Qd DRAINED TRIAXIAL	

LOG OF BOREHOLE NO. 400-1

PROJECT: HIGHWAY 400 UNDERPASS

LOCATION: VAUGHAN MILLS BOULEVARD, VAUGHAN, ONTARIO

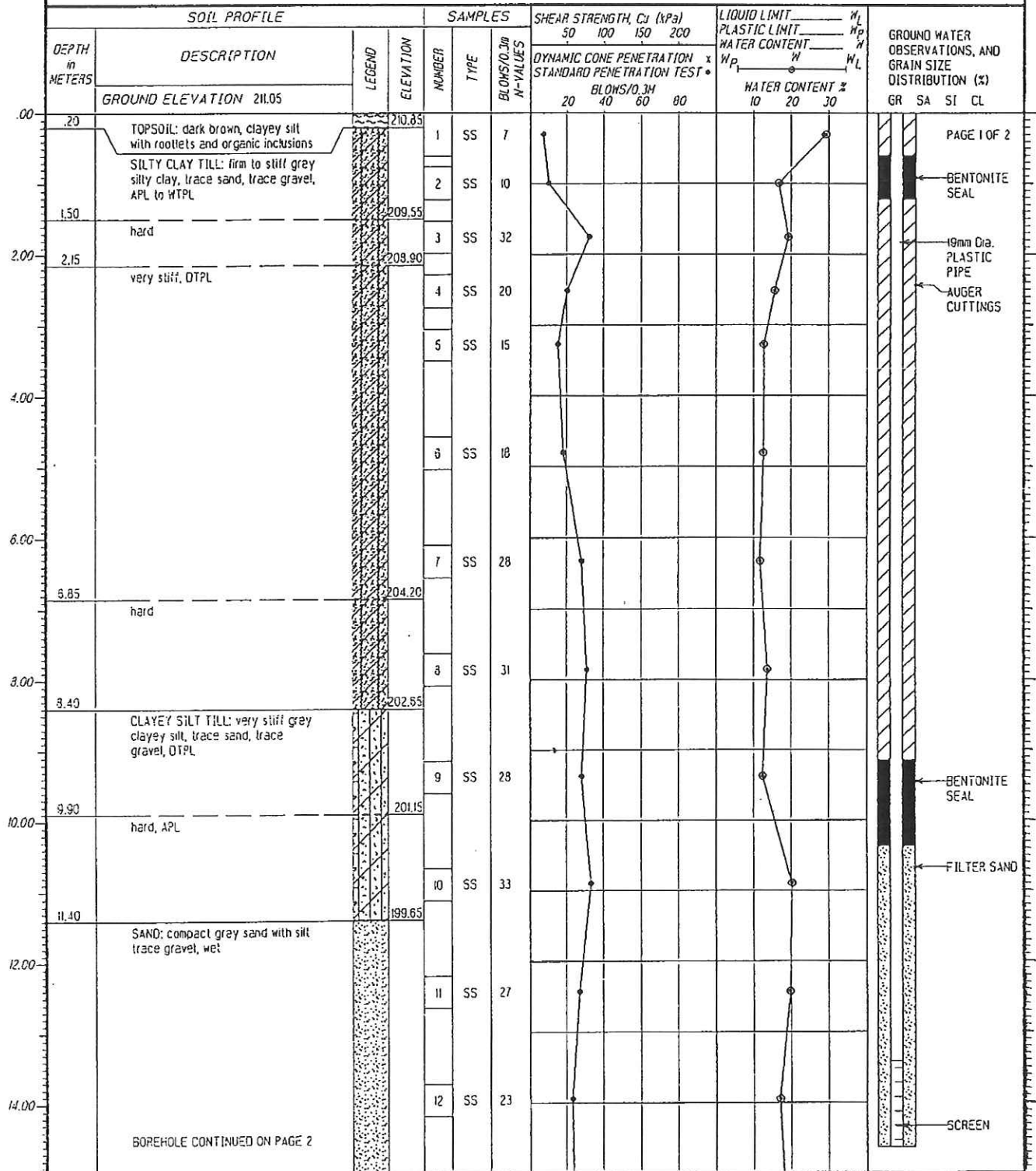
BORING METHOD: CONTINUOUS FLIGHT SOLID STEM AUGERS

BORING DATE: MARCH 31, 1999

OUR PROJECT NO.: 99TF023

ENGINEER: E.W.

TECHNICIAN: F.P.



NOTES:

+ --- UNDISTURBED FIELD VANE
 ⊙ --- REMOLDED FIELD VANE
 ● --- LAB SHEAR TEST
 ▲ --- POCKET PENETROMETER

CHECKED BY: E.W.

LOG OF BOREHOLE NO. 400-1

PROJECT: HIGHWAY 400 UNDERPASS

LOCATION: VAUGHAN MILLS BOULEVARD, VAUGHAN, ONTARIO

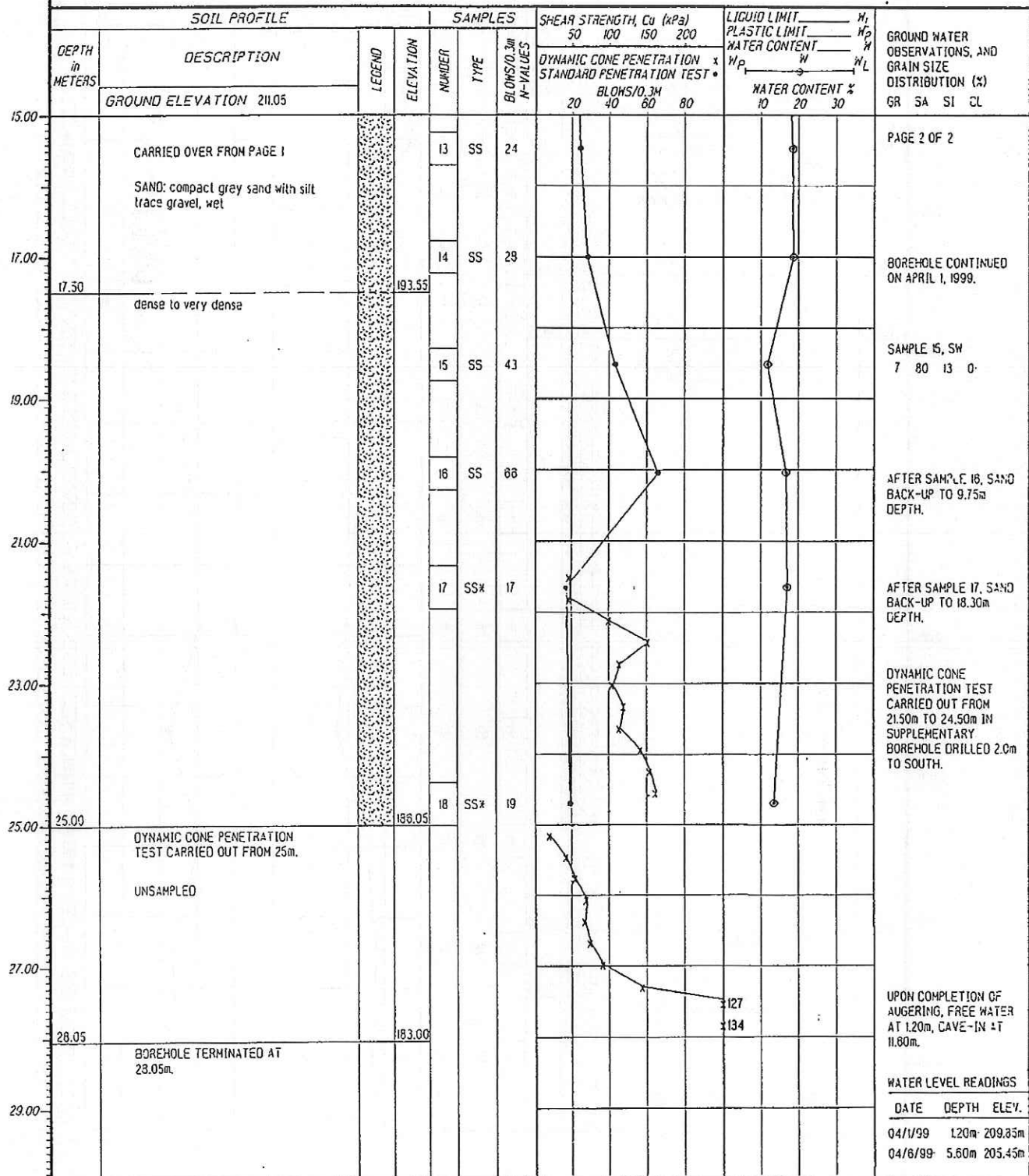
BORING METHOD: CONTINUOUS FLIGHT SOLID STEM AUGERS

BORING DATE: MARCH 31, 1999

OUR PROJECT NO.: 99TF023

ENGINEER: E.W.

TECHNICIAN: F.P.



NOTES: 1. * LOW 'N' VALUE DUE TO GROUND WATER DISTURBANCE.

2. LOW CONE 'N' VALUE BETWEEN 21.5 AND 27.4m DUE TO GROUND WATER DISTURBANCE.

+ --- UNDISTURBED FIELD VANE
 ⊕ --- REMOLDED FIELD VANE
 ⊙ --- LAB SHEAR TEST
 ▲ --- POCKET PENETROMETER

CHECKED BY: E.W.

LOG OF BOREHOLE NO. 400-2

PROJECT: HIGHWAY 400 UNDERPASS

LOCATION: VAUGHAN MILLS BOULEVARD, VAUGHAN, ONTARIO

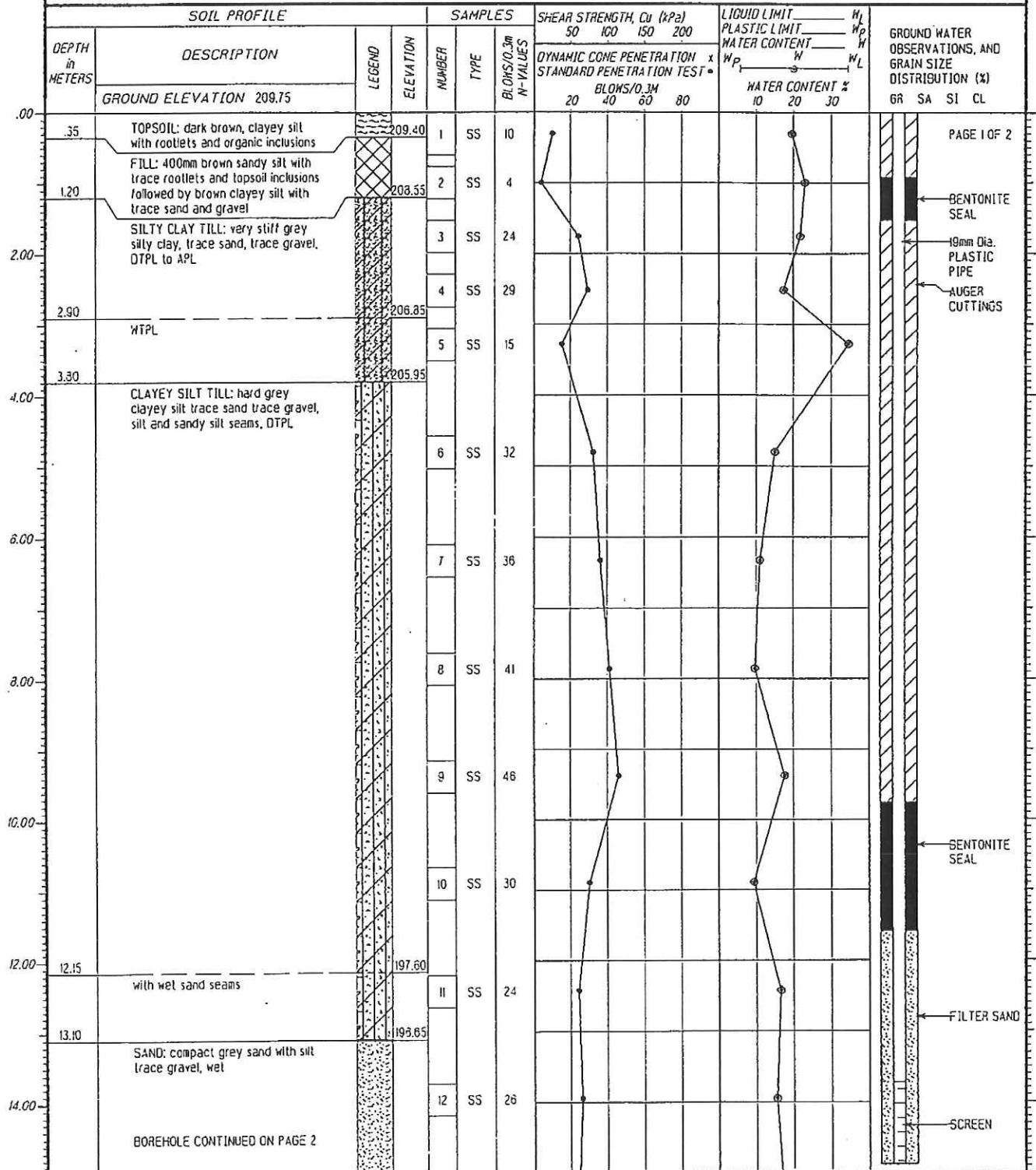
BORING METHOD: CONTINUOUS FLIGHT SOLID STEM AUGERS

BORING DATE: MARCH 31, 1999

OUR PROJECT NO.: 99TF023

ENGINEER: E.W.

TECHNICIAN: F.P.



NOTES:

+ --- UNDISTURBED FIELD VANE
 @ --- REMOLDED FIELD VANE
 ● --- LAB SHEAR TEST
 ▲ --- POCKET PENETROMETER

CHECKED BY: E.W.

LOG OF BOREHOLE NO. 400-2

PROJECT: HIGHWAY 400 UNDERPASS

LOCATION: VAUGHAN MILLS BOULEVARD, VAUGHAN, ONTARIO

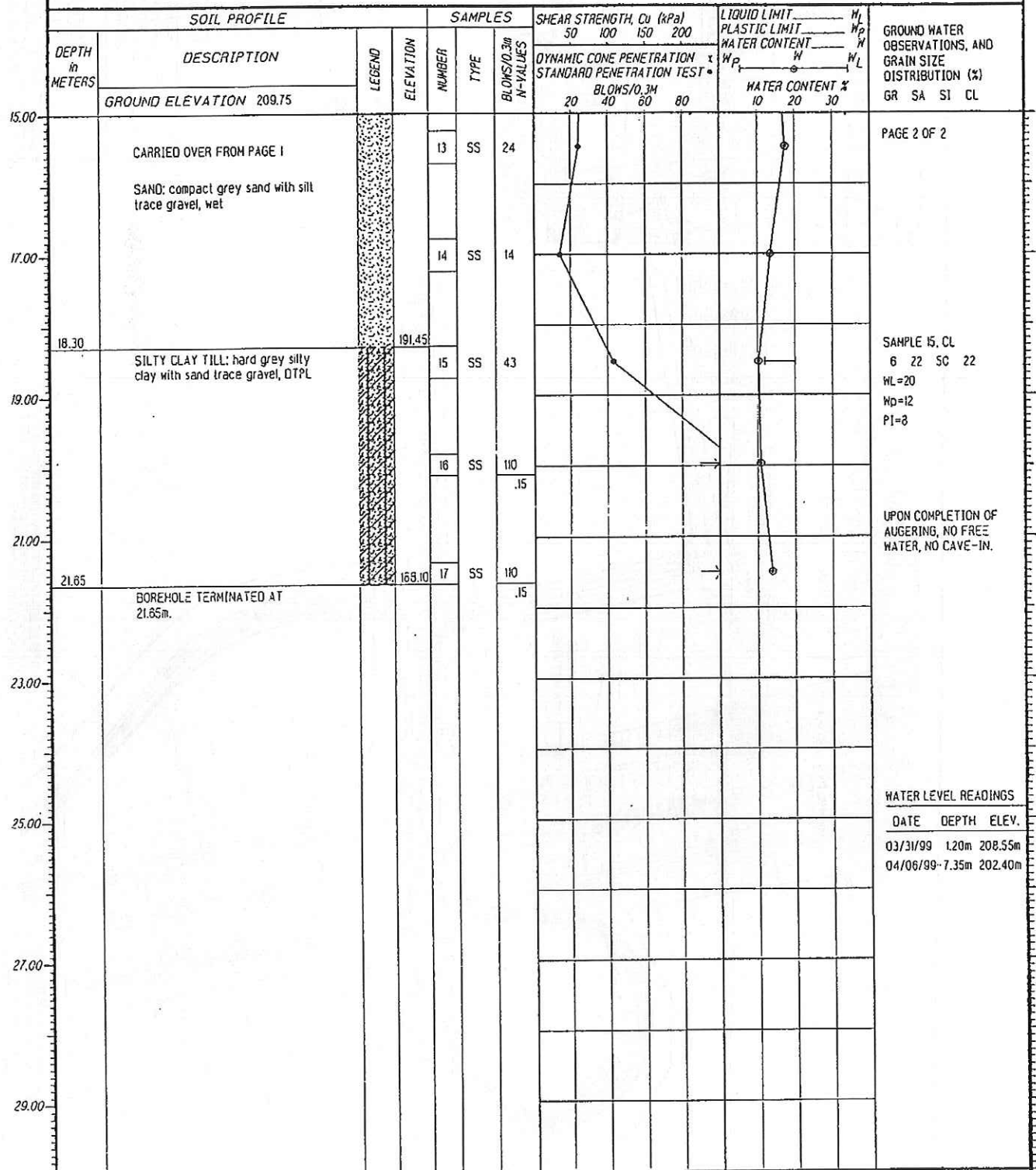
BORING METHOD: CONTINUOUS FLIGHT SOLID STEM AUGERS

BORING DATE: MARCH 31, 1999

OUR PROJECT NO.: 99TF023

ENGINEER: E.W.

TECHNICIAN: F.P.



NOTES:

+ --- UNDISTURBED FIELD VANE
⊕ --- REMOLDED FIELD VANE
⊗ --- LAB SHEAR TEST
▲ --- POCKET PENETROMETER

CHECKED BY: E.W.

