



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

O'BRIEN STREET OVERPASS

FUTURE HIGHWAY 11/17

CITY OF NORTH BAY

GWP 5748-04-00

DISTRICT 54, SUDBURY, ONTARIO

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**PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT**

for
O'Brien Street Overpass
Future Highway 11/17
City of North Bay
GWP 5748-04-00
District 54, Sudbury, Ontario

1. INTRODUCTION

This report summarizes the results of a preliminary foundation investigation carried out for construction of an overpass on the alignment of O'Brien Street at Highway 11/17 in North Bay, Ontario. The investigation was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The overpass construction is contemplated for the Future Highway 11/17 fully controlled access freeway from 1.5 km south of the South Junction of Highway 17 on Highway 11 northerly for 5.4 km within the City of North Bay.

The report provides preliminary subsurface information pertaining to the proposed overpass structure foundations and approaches within about 20 m of the abutments.

2. SITE DESCRIPTION AND GEOLOGY

The site is located on the O'Brien Street alignment at the intersection with the existing Highway 11/17 in North Bay. The overpass will carry the Future Highway 11/17 traffic over O'Brien Street. The alignment of the overpass is considered to be north-south.

Land use in the vicinity of the site comprises residential and commercial properties and community facilities. Photographs of the site are provided in Appendix A.



The local topography is relatively flat, gently sloping down to the east. The highway crosses Chippewa Creek about 150 m east of the existing O'Brien Street intersection. The ground beyond the right-of-way is occupied by residences, restaurants and a hotel. A stand of trees and bushes occurs to the north-south along the Chippewa Creek bank. The native soils are typically represented by sand/silt and clayey deposits.

The project site is situated in the Algonquin Highlands physiography region within the Canadian Shield. The typical rock type in the project area is highly metamorphosed metasedimentary migmatitic biotite gneiss of the Precambrian. The bedrock is at various depths, over 20 m in the vicinity of the site.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out on April 16 and 17, 2007, comprising 4 boreholes and one dynamic cone penetration test advanced to depths of 18.4 to 21.6 m at the locations shown on Drawing OS-1, appended. The boreholes were terminated on soils exhibiting SPT-N values over 100, according to the Terms of Reference for the project, and bedrock was not contacted in the boreholes.

The conditions within 20 m of the abutments were only inspected visually and inferred subsurface changes noted, since boreholes were not requested by MTO for preliminary design within these limits.

The locations of and ground surface elevations at the boreholes were determined by Peto MacCallum Ltd. (PML). The boreholes were advanced using continuous flight hollow stem augers, powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

Del Bosco Surveying Ltd. laid out and surveyed the borehole locations. PML cleared the locations of the boreholes for the presence of underground services and utilities. All elevations in this report are expressed in metres.



Representative soil samples were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted to assess the strength characteristics of the substrata. In situ vane shear testing using the MTO 'N' vane was also performed to further assess the shear strength of the cohesive soils encountered.

Groundwater conditions at the borehole locations were assessed during drilling by visual examination of soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes. All the boreholes were backfilled with a bentonite/cement mixture in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment procedures.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. The recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determination. In addition, one Atterberg limits test and 13 grain size distribution analyses were carried out on selected soil samples, their results being presented in respective Figures PC-OS-1 and GS-OS-1 to GS-OS-5 as well as on the corresponding Record of Borehole sheets.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classification, inferred stratigraphy, boundary elevations, standard and dynamic cone penetration resistance as well as in situ vane shear test data, groundwater observations, the results of the Atterberg limits test, grain size distribution analyses and moisture content determinations.

The borehole locations and stratigraphic cross-sections prepared from the borehole data are shown on Drawing OS-1.

The subsurface stratigraphy revealed in the boreholes drilled at the site comprised a surficial fill over sands and silts interlayered with clayey silt and underlain by silt till / sandy silt till. Boulders were encountered in one borehole. The strata encountered are summarized below.



4.1 Fill

Surficial fill was present in boreholes OS1, OS3 and OS4. Composed of sand with variable silt and gravel, the fill was 0.5 to 4.4 m thick and very loose to compact in relative density (SPT-N values of 3 to 12). The upper 0.5 and 1.2 m thick sand fill in boreholes OS1 and OS3 comprised the shoulder pavement fill of the existing highway. A 150 mm thick layer of concrete was penetrated at 2.7 m depth in borehole OS4. The fill had a moisture content of 11 to 19% and was penetrated at elevation 217.6 to 222.2.

The results of grain size distribution analysis performed on a representative sample of the sand fill are presented in Figure GS-OS-1.

4.2 Sand / Gravelly Sand

Present at surface in borehole OS2 and directly beneath the fill at depths of 0.5 to 4.4 m (elevation 217.6 to 222.2) in the remaining boreholes was cohesionless sand. This stratum ranged in thickness from 4.4 to 10.8 m and was very loose to dense (SPT-N values of 3 to 47), typically compact. The moisture content of the sand varied between 4 and 23%. The stratum extended to depths of 5.5 to 10.8 m (elevation 212.3 to 217.2).

A 1.5 m thick layer of sand was also encountered below silt at 16.2 m depth (elevation 206.5) in borehole OS1. This layer was dense (SPT-N value of 32), with a moisture content of 11%, and penetrated at a depth of 17.7 m (elevation 205.0).

Gravelly sand was identified below silt at 17.6 m depth (elevation 204.4) in borehole OS4. This unit was 2.2 m thick and very dense (SPT-N value of 69). The gravelly sand was underlain by very dense sand at a depth of 19.8 m (elevation 202.2). Borehole OS4 was terminated in the sand at 21.6 m depth (elevation 200.4).

The grain size distribution charts of eight samples of the sand are presented in Figure GS-OS-2.



4.3 Silty Sand

Overlain by the sand in borehole OS1 and silt in borehole OS3 at respective depths of 5.5 and 15.0 m (elevation 217.2 and 207.0) was silty sand. The relative density of the silty sand was loose in the former borehole and compact in the latter (SPT-N values of 4 and 13). Having a moisture content of 19% in borehole OS3 and a thickness of 1.5 m, the silty sand extended to depths of 7.0 and 16.5 m (elevation 215.7 and 205.5).

4.4 Silt

Non-plastic silt was revealed below the sand / silty sand or clayey silt at depths of 7.0 to 14.0 m (elevation 209.1 to 215.7) in all the boreholes. Interlayered with clayey silt in three boreholes, this stratum was very loose to compact in relative density and 11 to 27% in moisture content. With a total thickness varying between 3.5 and 9.2 m, the silt was penetrated at depths of 15.0 to 17.6 m (elevation 204.4 to 207.0).

The grain size distribution charts of two samples of the silt are presented in Figure GS-OS-3.

4.5 Clayey Silt

Cohesive clayey silt was encountered below the sand at 10.8 m depth (elevation 212.3) in borehole OS2 or within the silt at depths of 10.0 to 10.2 m (elevation 211.8 to 212.5) in the remaining boreholes. This deposit was 1.5 to 3.2 m thick and soft to firm in consistency. The results of in situ vane testing carried out within the clayey silt yielded undisturbed shear strength values of 46 and over 100 kPa (soil sensitivity of 4). The deposit extended to depths of 11.7 to 14.0 m (elevation 209.1 to 211.0).

The results of Atterberg limits testing and grain size distribution analysis conducted on a cohesive soil sample are presented in respective Figures PC-OS-1 and GS-OS-4. The liquid and plastic limits of the clayey silt were 33 and 20 respectively, thus giving a plasticity index of 13. The moisture content of the deposit ranged from 31 to 44%.



4.6 Silt Till / Sandy Silt Till

Underlying the sand, silty sand or silt at depths of 16.5 to 17.7 m (elevation 205.0 to 205.6) in boreholes OS1 to OS3 was silt till / sandy silt till. This unit was compact to 17.5 to 18.2 m depth (elevation 203.8 to 205.6) and very dense below these levels, with a moisture content of about 16%. The silt till / sandy silt till was not penetrated upon termination of the boreholes (on probable boulder in borehole OS2) at depths of 17.5 to 18.6 m (elevation 204.1 to 204.7), likely extending to 19.4 m depth (elevation 202.6) at which a dynamic cone penetration test was completed in borehole OS3.

The results of grain size distribution analysis performed on a sample of the silt till are presented in Figure GS-OS-5.

4.7 Groundwater

In the course of the field work, groundwater was observed in all the boreholes. During the augering, water was detected at depths of 3.1 to 4.5 m (elevation 218.6 to 218.9). Upon completion of drilling, groundwater was measured in boreholes OS1, OS3 and OS4 at depths of 5.0 to 5.1 m (elevation 217.0 to 217.6). The groundwater levels at the site are subject to seasonal fluctuations and precipitation patterns.

5. MISCELLANEOUS

The field work was carried out under the supervision of Mr. M. Rapsey, Senior Technician, and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer. Walker Drilling Co. Ltd. supplied the drilling equipment. The laboratory work was carried out in the PML laboratory in Toronto.

This Preliminary Foundation Investigation Report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

**PART B
PRELIMINARY FOUNDATION DESIGN REPORT**

for
O'Brien Street Overpass
Future Highway 11/17
City of North Bay
GWP 5748-04-00
District 54, Sudbury, Ontario

6. ENGINEERING RECOMMENDATIONS

6.1 General

This report provides preliminary foundation engineering comments and recommendations regarding design and construction of foundations, abutments and approach fill embankments for the proposed grade separation overpass to be located at O'Brien Street and the existing / Future Highway 11/17 in North Bay, Ontario.

The overpass is envisaged to be a single span structure with a span length of 36.0 m and width of about 29.8 m (ref. Preliminary General Arrangement (GA) drawing prepared by Stantec in July 2007).

The road grade on the Future Highway 11/17 at the overpass location will be near elevation 227.2 at the north abutment and 226.9 at the south abutment. The approach embankments to the structure are envisaged to be approximately 4.0 to 4.5 m high at the north abutment and 4.9 m high at the south abutment (interpreted from the road grade and ground surface elevations shown on the preliminary GA drawing). Existing grade along the alignment of O'Brien Street will be cut about 2.0 to 2.5 m to the proposed grade near elevation 220.0.

In summary, the subsurface stratigraphy revealed in the boreholes drilled at the site comprised a surficial fill over very loose to dense sands and silts interlayered with soft to firm clayey silt and underlain by compact to very dense silt till / sandy silt till. Boulders were encountered in one borehole. The boreholes were terminated on soils exhibiting SPT-N values over 100, according to the Terms of Reference for the project, and bedrock was not contacted in the boreholes. Groundwater was at elevation 217.0 to 217.6 upon completion of drilling.



Based on the preliminary data and subject to results of a detailed field investigation, design and construction of foundations to support the proposed overpass is considered feasible at this site.

It is noted that the relative density of the upper sandy/silty soils decreases with depth from compact to loose / very loose and the cohesionless soils are interlayered at 10 m depth by a 1.5 to 3.2 m thick deposit of compressible soft to firm clayey silt. The native soils become dense to very dense at a depth of about 18 m below grade (elevation 205 at the north abutment and elevation 204 at the south abutment). Consequently, the use of spread footings or caissons to support this structure is not recommended due to potentially large settlements and/or installation difficulties. H-piles driven to refusal on the very dense glacial till (or on the bedrock likely underlying the site) are considered to be the preferred foundation system for the overpass from a foundation engineering perspective. However, a detailed field investigation is needed to define the thickness of the very dense till and/or to determine the bedrock surface elevations within the limits of the structure foundations.

The presence of compressible clayey soils necessitates special construction procedures to enable pile installation, limit the development of negative skin friction on the piles and overstressing of the clayey soils when subjected to the embankment loading which could result in settlement of the embankment adjacent to the abutment and/or increased lateral loads on the piles.

The foundation frost penetration depth at this site is 2.0 m according to OPSD-3090.100. The seismic site coefficient is 1.0 – Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC), 2006 Edition, for the anticipated foundation system.

6.2 Foundations

6.2.1 Piles

Construction of conventional or integral abutments supported on steel H-piles driven to the very dense silt till / sandy silt till (or underlying bedrock) should be feasible at this site. Assuming a pile penetration of 1.5 m into the very dense glacial till, the piles would be about 19 to 20 m long and founded at approximate elevation 202 to 204. We note that the detailed subsurface investigation



may not determine the bedrock surface if the overlying very dense glacial till or sand and gravel strata with SPT-N values over 100 exceed 3 m in thickness, because these deposits would be considered adequate for pile foundations.

The piles will be driven through the existing fill containing localized concrete rubble and one of the boreholes put down at the north abutment was terminated on probable boulders. Consequently, there is a risk that the piles may be damaged during driving through rubble or cobbles/boulders expected at the site and/or may reach refusal in the very dense till above the bedrock surface. For preliminary purposes and to account for the above uncertainties, a reduction of about 200 kN was applied to the axial resistance at ultimate limit states (ULS) for the following pile sections driven to refusal on very dense glacial till or sand and gravel at the levels indicated previously:

Pile Section	Factored Axial Resistance at ULS (kN)
HP 310 x 110	1,600
HP 310 x 132	1,900

The resistance at serviceability limit states (SLS) normally allows for 25 mm compression of the pile and founding medium. Considering the very dense glacial till (or bedrock) to be non-yielding materials and the expected length of the piles, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm axial deformation of the pile is larger than the factored resistance at ULS.

Pile caps should be provided with at least 2.0 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.

6.2.2 Spread Footings

The sand near the probable founding elevation 218 is marginally compact and underlain by loose to very loose sandy/silty soils interlayered with soft to firm clayey silt within the zone of influence of spread footings. Therefore, it is not recommended to support the structure on conventional



spread footings founded in the sand due to the relatively low bearing resistance of the native soils and potentially excessive settlements of the structure abutments.

6.2.3 Caissons

Installation of caissons will be impractical due to the high groundwater level, the presence of typically loose silt and pervious sand strata as well as of potential cobbles and boulders at the site. Consequently, it is considered that use of caissons to support the foundation loads is not suitable for this structure.

6.3 Approach Embankments

It is anticipated that the approach embankments will be constructed with earth borrow, granular material and/or rock fill. The design calls for the embankment to be about 4.0 to 4.5 m high at the north abutment and 4.9 m high at the south abutment. The subgrade revealed in the boreholes drilled at the abutments consists of the existing embankment fill over compact sandy/silty soils becoming loose to very loose and interlayered at 10 m depth with soft to firm clayey silt. Subject to further investigation during Detail Design, the existing fill and native soils are considered marginally competent to support the proposed additional embankment height.

Any topsoil or deleterious materials at the abutment locations and along the alignment of the approach fill within 20 m of the abutments should be stripped prior to placement of the new embankment fill.

The embankments should be constructed following conventional MTO procedures (OPSD-200.010, 201.010, 202.010 and SP 206S03) and have side slopes inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rock fill.

The settlement of the approach fill and native soils induced by the new embankment loads is anticipated to be in the order of 50 mm. The backfill to the abutments should comprise granular materials to minimize long-term settlements.



6.4 Construction Considerations

6.4.1 Excavation

Excavation for construction of the O'Brien Street pavement to the lowest level at about elevation 220.0 is expected to extend through the fill and compact sand to a depth of about 2 to 3 m. Excavation of these soils should be relatively straightforward.

The fill and compact sand are classified as Type 3 material according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Therefore, temporary cut slopes over the full depth of excavation should be inclined at an angle of 45° to the horizontal. The need to excavate flatter sideslopes if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction should not be overlooked.

6.4.2 Groundwater Control

The highest groundwater level observed in the course of the field work was below elevation 219. This level is about 1.0 m lower than the planned lower level of the O'Brien Street pavement, therefore it is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations.

It is noteworthy that groundwater levels are subject to seasonal fluctuations and precipitation patterns.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.



6.5 Lateral Earth Pressures

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, p (kPa) may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation.

$$p = K (\gamma h + q) + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)
 γ = unit weight of free-draining granular material, kN/m^3
 h = depth below final grade, m
 q = surcharge load, kPa, if present
 C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
Where \emptyset = angle of internal friction of retained soil (35° for Granular A or Granular B Type II or Type III)
 δ = angle of friction between the soil and wall (23.5° for Granular A or Granular B Type II or Type III)

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

PARAMETERS	GRANULAR A or GRANULAR B TYPE II or TYPE III
Internal Friction Angle, \emptyset (degrees)	35
Unit weight, γ (kN/m^3)	22.8
Coefficient of Active Earth Pressure, K_a	0.27
Coefficient of Earth Pressure At Rest, K_o	0.43
Coefficient of Passive Earth Pressure, K_p	3.69

The assigned geotechnical parameter values are the same for all granular materials in view of their similar physical characteristics.



Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed in design of 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.

The magnitude of the passive resistance is dependent on the actual lateral movement of the structure toward the retained soil. We refer to Figure C6.16 of the CHBDC for this computation. The subsoil/backfill should be considered as medium dense sand for the project.

A subdrain system (SP 405F03) and/or weep holes (OPSD-3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the wall. The subdrains 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 installed on a positive grade and lead to a frost-free outlet.

7. ADDITIONAL STUDIES

The recommendations in this report are preliminary and based on PML's interpretation of the factual information obtained from a limited number of boreholes and a visual site assessment. Detailed foundation investigation will be required at the structure location during the Detail Design phase of the project. The interpretation and recommendations are only provided for planning purposes and feasibility studies.

Based on the about 30 m wide structure, the limited data and visual site assessment of the overpass structure, the recommended scope of work for the foundation investigation is as follows:

- Three boreholes should be carried out at the centre and at each end of the south and north abutments for a total of 6 boreholes. Three boreholes should extend 3 m beyond refusal defined as bedrock or material with standard penetration resistance N values over 100.



- Two boreholes should be carried out for each approach embankment and extend to at least 15 m depth to investigate the condition of the native cohesive soils.
- Four boreholes should be drilled in the section of cut for the O'Brien Street platform. These boreholes should also extend to a minimum depth of 15 m.

8. DISCUSSION OF FOUNDATION ALTERNATIVES

In view of the site conditions described previously, it is considered that spread footings or caissons are not feasible or practical at this site. Consequently, the foundation alternatives were limited to the use of driven piles and a discussion of their advantages and disadvantages is not considered to be necessary.

From the foundation perspective, conventional, semi-integral or integral abutments founded on driven piles are considered to be feasible. The integral abutments are considered to be the most economic in the long term in view of the lower maintenance costs.

The selected foundation scheme also depends on such considerations as structural design and road grades which are being evaluated separately by Stantec.



9. CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

A handwritten signature in blue ink, appearing to read 'G. Degil'.

Grigory O. Degil, PhD, P.Eng.
Senior Foundation Engineer



A handwritten signature in blue ink, appearing to read 'C. M. P. Nascimento'.

C. M. P. Nascimento, P.Eng.
Senior Project Engineer



A handwritten signature in blue ink, appearing to read 'Brian R. Gray'.

Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

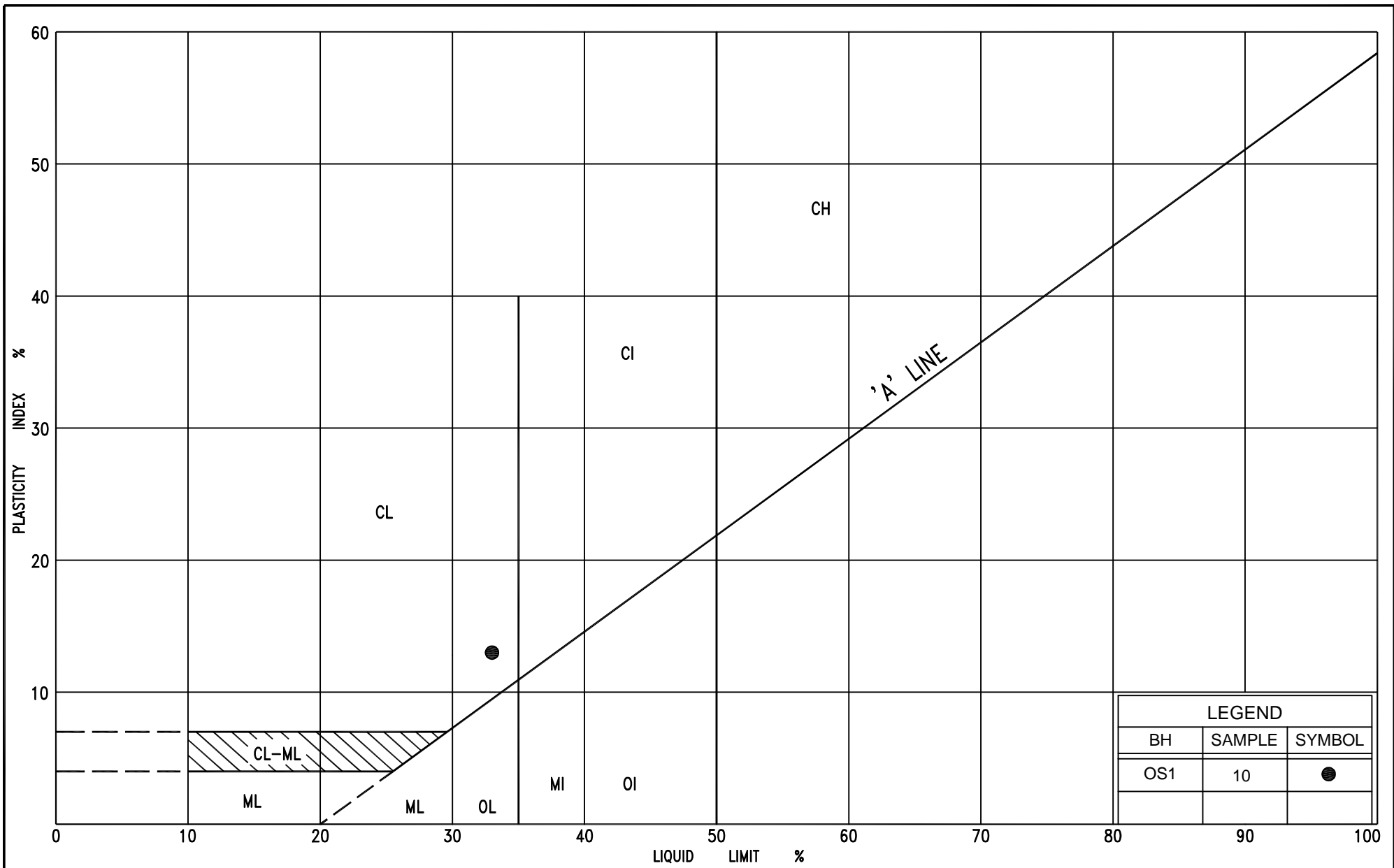


GD/CN/BRG:gd-mi



TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
SP 206S03	Construction Specification for Grading	November 2006
SP 405F03	Construction Specification for Pipe Subdrains	November 2006
OPSD-200.010	Earth/Shale Grading – Undivided Rural	November 2005
OPSD-201.010	Rock Grading-Undivided Rural	November 2005
OPSD-202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment	November 2005
OPSD-3090.100	Foundation Frost Depth for Northern Ontario	November 2005
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail	November 2005



Ministry of
Transportation
Ontario

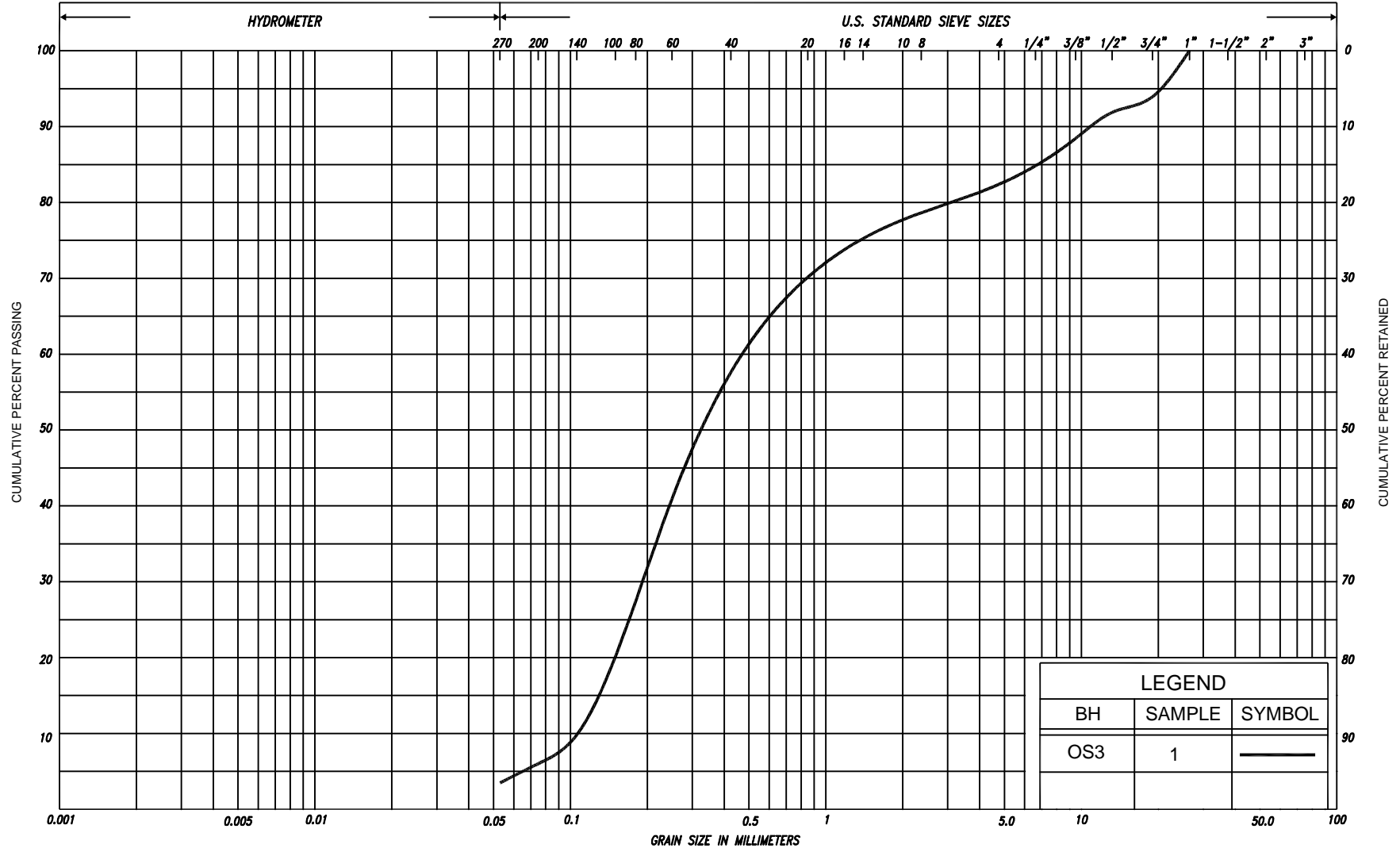
PLASTICITY CHART

CLAYEY SILT, trace sand

FIG No. PC-OS-1

HWY: 11/17

G.W.P. No. 5748-04-00



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL				COBBLES	UNIFIED		
					SAND													
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE			GRAVEL				COBBLES
	SILT																	
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL								U.S. BUREAU
					SAND													

GRAIN SIZE DISTRIBUTION

SAND, some gravel, trace silt
(PAVEMENT FILL)

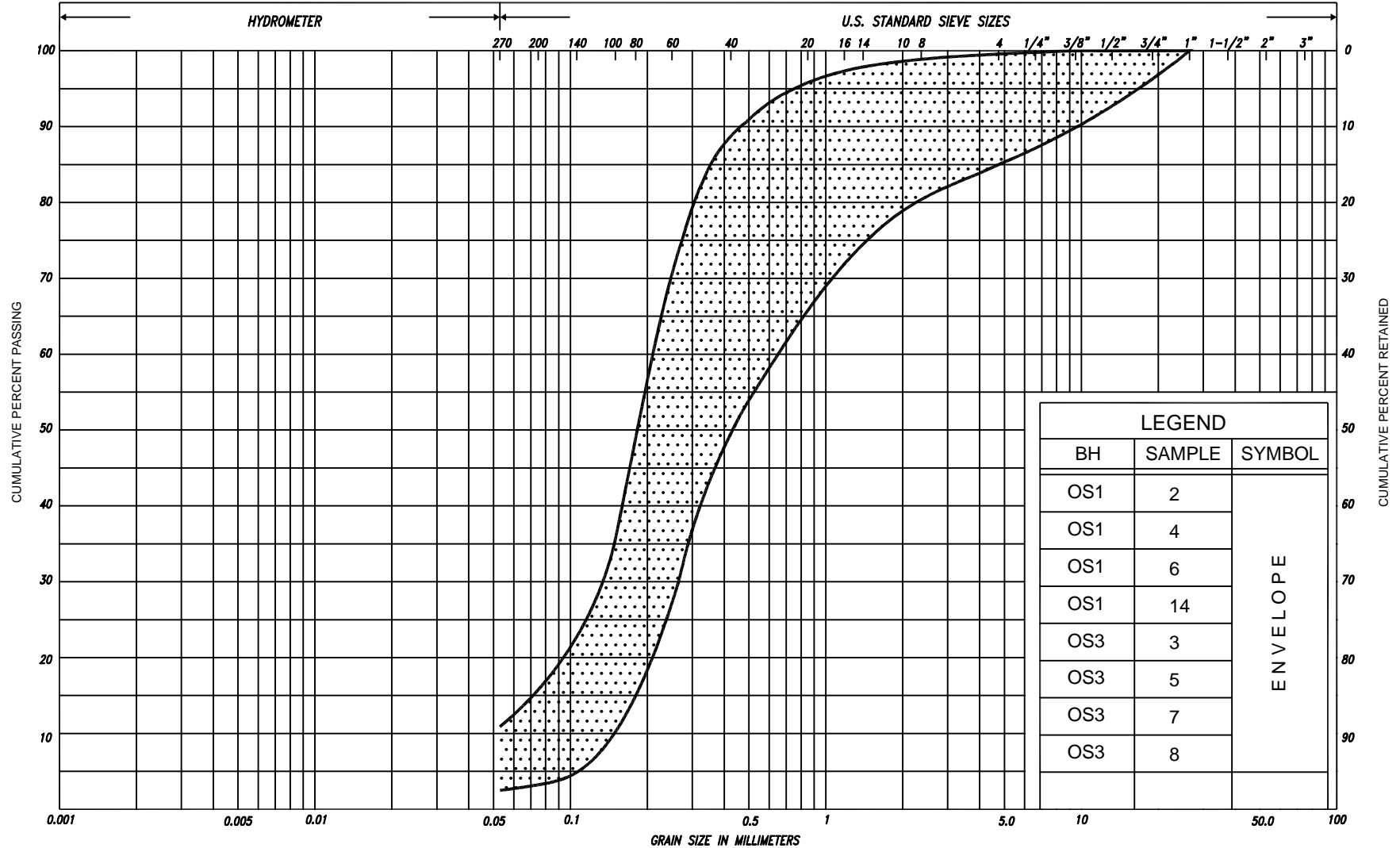
FIG No. GS-OS-1

HWY: 11/17

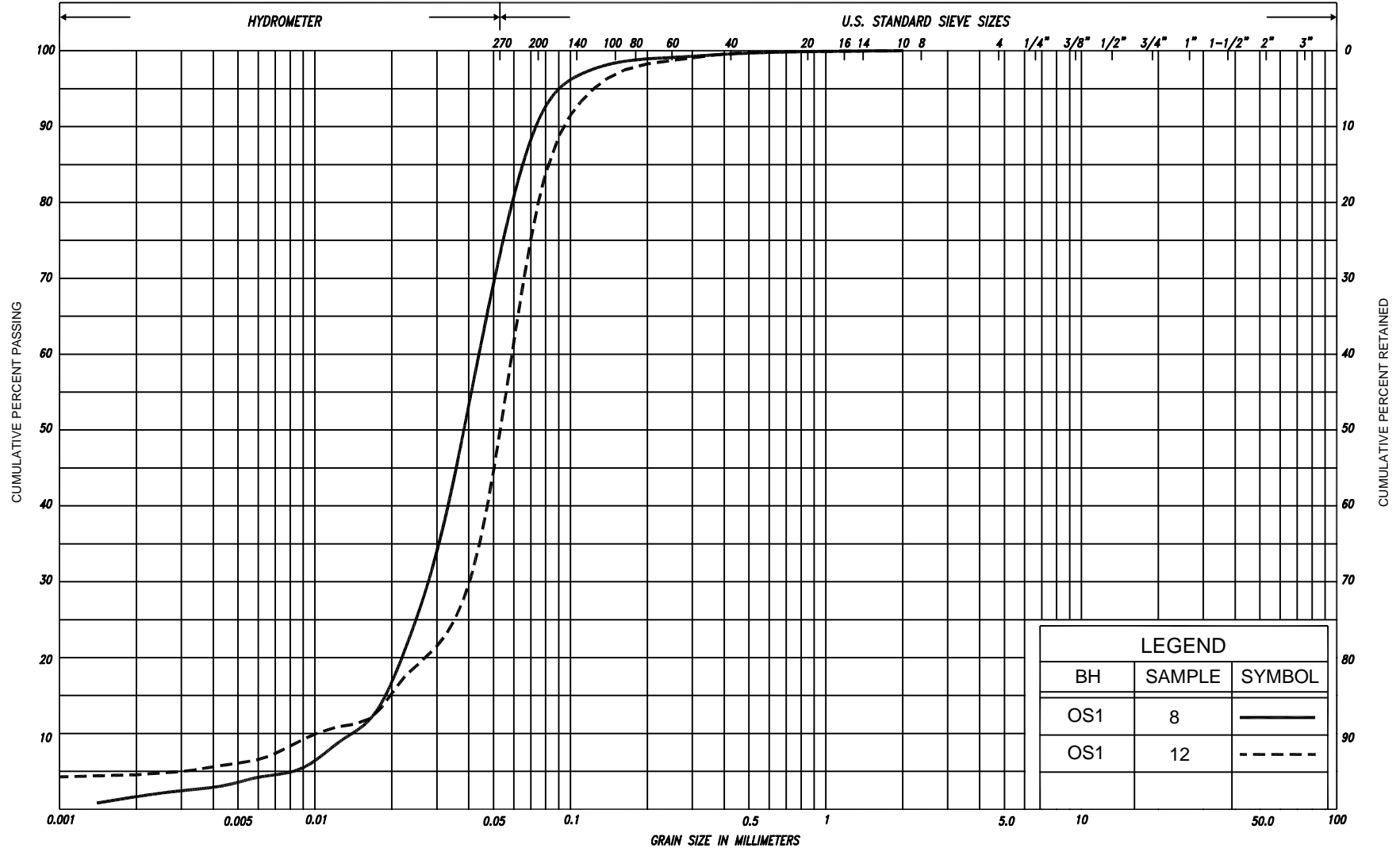
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SILT & CLAY				FINE		MEDIUM		COARSE		GRAVEL				COBBLES	UNIFIED		
				SAND													
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT																
CLAY			SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU
						SAND											



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED			
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.	
	SILT					SAND							GRAVEL			COBBLES	M.I.T.	
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL							COBBLES	U.S. BUREAU

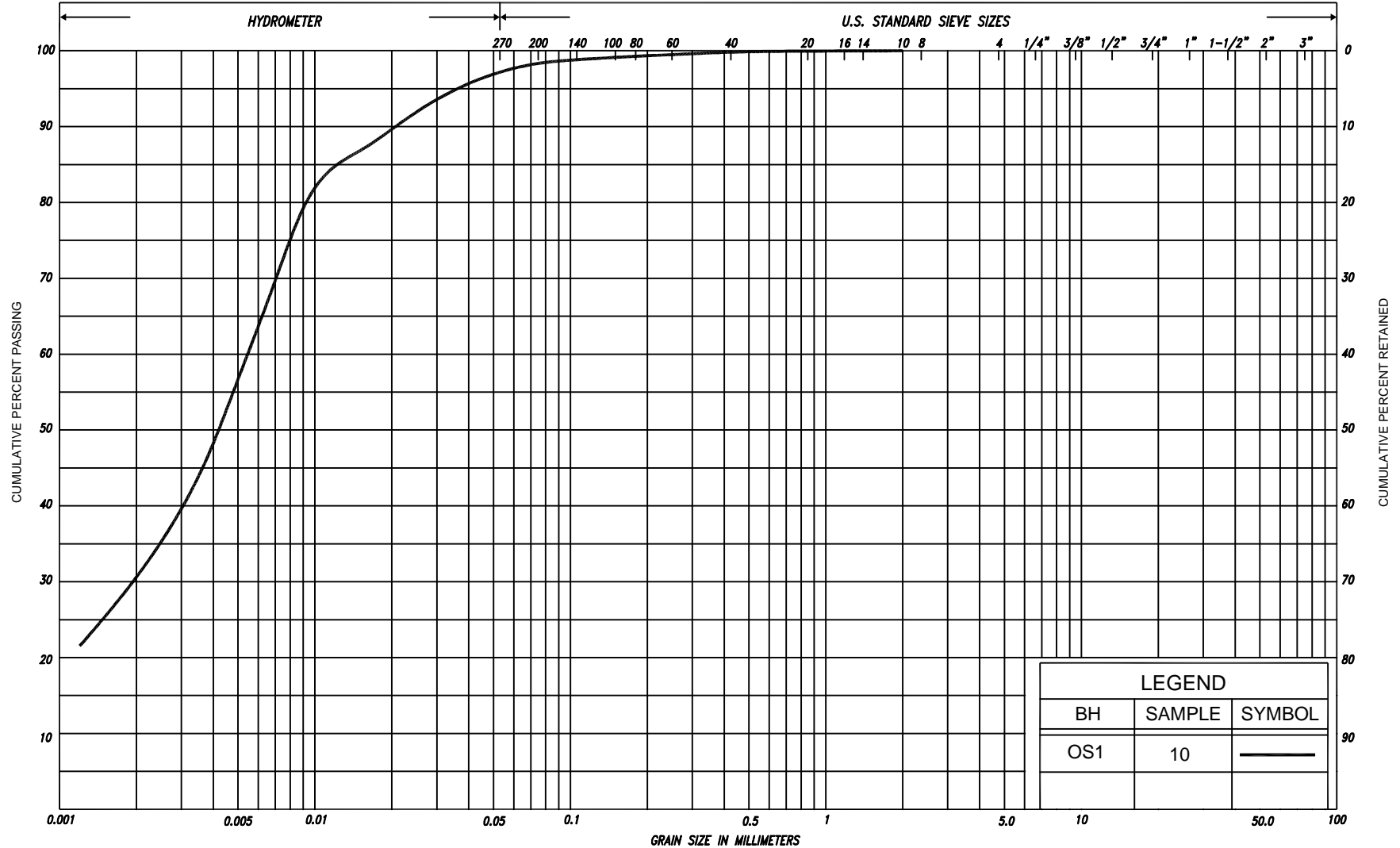
GRAIN SIZE DISTRIBUTION

SILT, trace to some sand, trace clay

FIG No. GS-OS-3

HWY: 11/17

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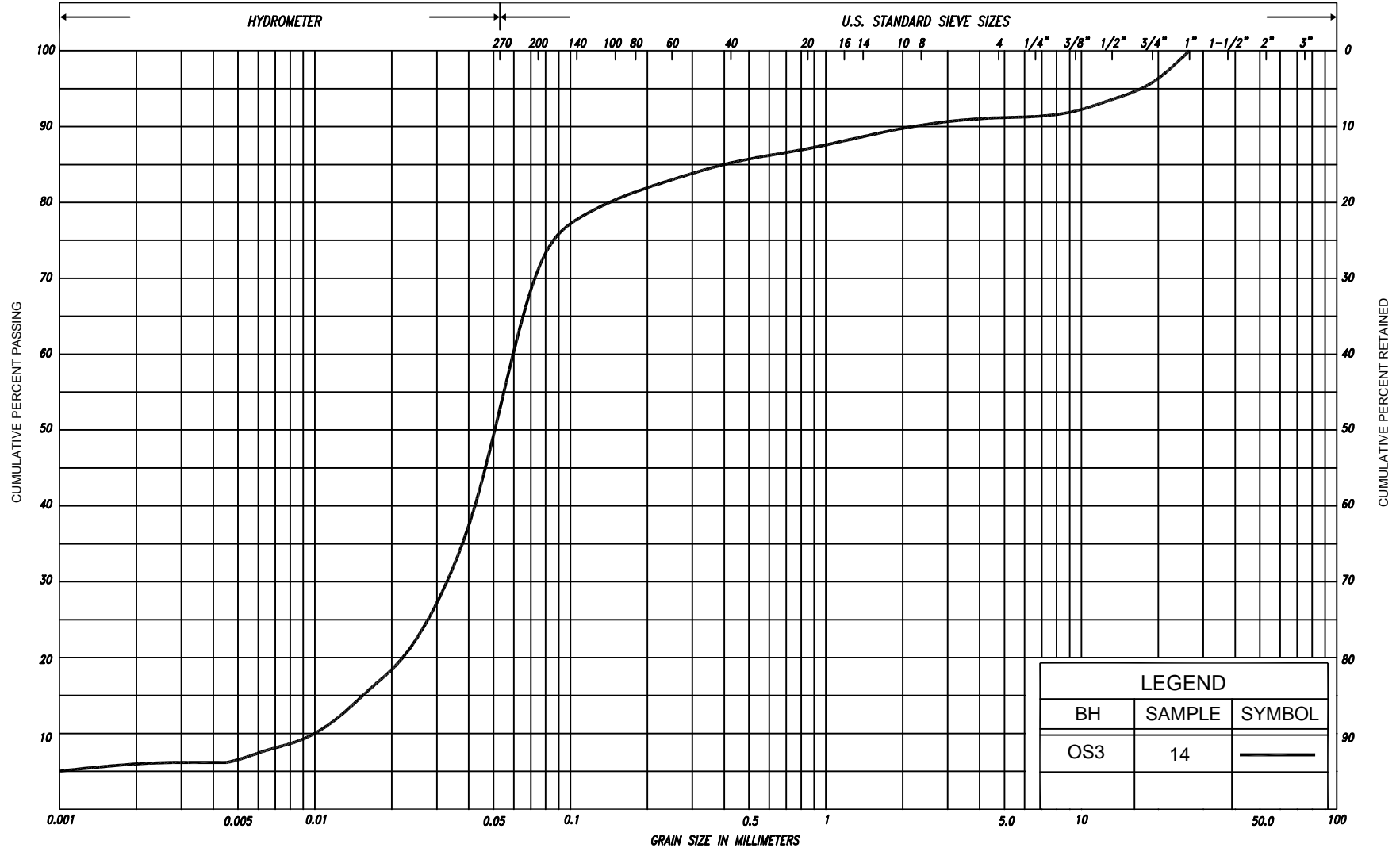
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand

FIG No. GS-OS-4

HWY: 11/17

G.W.P. No. 5748-04-00



SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COB BLES	UNIFIED			
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL		COBBLES	M.I.T.		
	SILT					FINE		SAND							U.S. BUREAU		
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL							
					SAND												

GRAIN SIZE DISTRIBUTION

SILT, some sand, trace gravel, trace clay
(TILL)

FIG No. GS-OS-5

HWY: 11/17

G.W.P. No. 5748-04-00



Ministry of
Transportation
Ontario

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m^3	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m^3	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m^3/s	RATE OF DISCHARGE
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kN/m^2	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No OS1

1 of 2

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 132 071 N; 308 067 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
DATUM Geodetic DATE April 17, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w _p	w	w _L					
								○ UNCONFINED		+ FIELD VANE										
								● QUICK TRIAXIAL		× LAB VANE										
								WATER CONTENT (%)												
							20	40	60	80	100	20	40	60						
222.7	Ground Surface																			
0.0	Sand, trace silt																			
222.2	Brown/ Dry grey																			
0.5	(PAVEMENT FILL)																			
	Sand trace silt, trace gravel		1	SS	14															
	Compact Brown Moist		2	SS	16								○					1	89 (10)	
			3	SS	19															
			4	SS	16								○					5	89 (6)	
	Wet																			
			5	SS	19															
			6	SS	12								○					3	94 (3)	
217.2	Silty sand																			
5.5	Loose Brown Wet		7	SS	4															
215.7	Silt trace sand, trace clay																			
7.0	Loose Brown Wet		8	SS	7								○					0	9 89 2	
	Grey		9	SS	7															
212.5	Clayey silt, trace sand thin layers of silt																			
10.2	Firm Grey Wet		10	SS	4								+					0	2 67 31	
				FV																
211.0	Silt, some sand layers of silty clay																			
11.7	Very loose Grey Wet		11	SS	2															
	trace clay		12	SS	4								○					0	20 75 5	
	Loose																			
207.7	Cont'd																			

RECORD OF BOREHOLE No OS1

2 of 2

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 132 071 N; 308 067 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
DATUM Geodetic DATE April 17, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER	CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa											W _P
207.7 15.0	Compact							207											GR SA SI CL	
			13	SS	20															
206.5 16.2	Sand some silt, some gravel Dense Grey Wet							206										15 69 (16)		
			14	SS	32															
205.0 17.7	Silt, with sand some gravel, some clay Very dense Grey Wet							205												
204.1 18.6	(TILL)																			
18.6	End of borehole																			
<div><div>*</div><div>2007 04 17</div></div> <div><div>▽</div><div>Water level observed during drilling</div></div> <div><div>▼</div><div>Water level measured after drilling</div></div>																				

RECORD OF BOREHOLE No OS2

1 of 2

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 132 033 N; 308 069 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
DATUM Geodetic DATE April 16, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)								
								○ UNCONFINED + FIELD VANE													
								● QUICK TRIAXIAL × LAB VANE													
						20	40	60	80	100	W _P	W	W _L								
223.1	Ground Surface																				
0.0	Sand trace silt, trace gravel																				
	Dense to Brown Dry to compact wet		1	SS	47																
			2	SS	23																
	Grey		3	SS	25																
			4	SS	10																
			5	SS	15																
			6	SS	10																
	Loose to very loose		7	SS	4																
	with silt																				
			8	SS	6																
			9	SS	3																
212.3	Clayey silt		10	SS	2																
10.8	thin layers of silt																				
	Soft Grey Wet																				
	reddish brown layers		11	SS	WH**																
				FV																	
			12	SS	4																
209.1	Silt, trace sand																				
14.0	Loose Grey Wet																				

Cont'd

ON MOT VER3 05TF058-ST.GPJ ON MOT.GDT 11/6/2007 8:02:06 AM

$+^7, \times^5$: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No OS3

1 of 2

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 132 062 N; 308 118 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE C.F.H.S.A + Dynamic Cone Penetration Test COMPILED BY T.X.
DATUM Geodetic DATE April 17, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED		+ FIELD VANE							
								● QUICK TRIAXIAL		× LAB VANE							
							WATER CONTENT (%)										
							20	40	60	80	100	20	40	60			
222.0	Ground Surface																GR SA SI CL
0.0	Sand some gravel, trace silt Compact Brown Moist																
220.8	(PAVEMENT FILL)		1	SS	11		221						○				18 76 (6)
1.2	with lenses of sandy topsoil, organics																
219.8	(FILL)		2	SS	12		220						○				
2.2	Sand trace gravel, trace silt Compact to Brown Moist very loose Wet		3	SS	13		219						○				2 89 (9)
			4	SS	7		218						○				7 88 (5)
			5	SS	15		217						○				
			6	SS	3		216						○				2 95 (3)
	Mottled rusty brown						215										
214.1	Silt, trace sand		8	SS	9		214						○				1 94 (5)
7.9	Loose to Grey Wet compact						213						○				
212.0	Clayey silt occ. layers of silt						212										
10.0	Soft to Grey Wet firm		10	SS	3		211						○				
210.3	Silt, trace sand						210										
11.7	Compact Grey Wet to loose		11	SS	11		209						○				
							208						○				
207.0	Cont'd																

RECORD OF BOREHOLE No OS3

2 of 2

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 132 062 N; 308 118 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE C.F.H.S.A + Dynamic Cone Penetration Test COMPILED BY T.X.
DATUM Geodetic DATE April 17, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER	CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa										WATER CONTENT (%)		
									○ UNCONFINED		+ FIELD VANE								● QUICK TRIAXIAL		
207.0								20	40	60	80	100									
15.0	Silty sand Compact Grey Wet		13	SS	13																
205.5								206													
16.5	Silt, some sand trace gravel, trace clay																				
204.5	Compact Grey Wet to dense		14	SS	17			205									9 19 66 6				
17.5	(TILL) End of borehole Probable silt Compact to dense (TILL)							204													
202.6								203													
19.4	End of dynamic cone penetration test																				
<div>* 2007 04 17</div> <div>▽ Water level observed during drilling</div> <div>▼ Water level measured after drilling</div> <div>C.F.H.S.A - Denotes Continous Flight Hollow Stem Augers</div> <div>WH** Denotes penetration due to weight of rods and hammer</div>																					

RECORD OF BOREHOLE No OS4

1 of 2

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 132 030 N; 308 114 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
DATUM Geodetic DATE April 17, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)					
								○ UNCONFINED		+ FIELD VANE													
								● QUICK TRIAXIAL		× LAB VANE													
222.0	Ground Surface						20	40	60	80	100												
0.0	Sand trace silt, trace gravel occ. silty clay																						
	Compact to Brown Moist very loose to wet		1	SS	10																		
			2	SS	4																		
	concrete		3	SS	3																		
	Mottled grey		4	SS	12																		
	some gravel																						
	(FILL)		5	SS	12																		
217.6	Sand, trace silt																						
4.4	Compact Brown Wet		6	SS	11																		
			7	SS	12																		
			8	SS	15																		
	silty		9	SS	11																		
	Rusty brown																						
213.3	Silt, trace sand																						
8.7	occ. layers of clayey silt																						
	Very Brown Wet loose		10	SS	3																		
211.8	Clayey silt																						
10.2	with layers of silt																						
	Firm Grey Wet		11	SS	4																		
209.7	Silt																						
12.3	Loose to Grey Wet very loose		12	SS	5																		
			13	SS	1																		

Cont'd

<div style="display: flex; justify-content: space-between;"> RECORD OF BOREHOLE No OS4 2 of 2 METRIC </div>																		
G.W.P. 5748-04-00		LOCATION Co-ords. 5 132 030 N; 308 114 E.				ORIGINATED BY M.R.												
DIST Sudbury HWY 11/17		BOREHOLE TYPE Continuous Flight Hollow Stem Augers				COMPILED BY T.X.												
DATUM Geodetic		DATE April 17, 2007				CHECKED BY C.N.												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa											
207.0							20	40	60	80	100							
	trace sand																	
	Compact		14	SS	11													
			15	SS	12													
204.4																		
17.6	Gravelly sand, with silt																	
	Very Grey Wet dense																	
			16	SS	69													
202.2																		
19.8	Sand with gravel, trace silt																	
	Very Grey Wet dense																	
200.4			17	SS	100/ 20cm													
21.6	End of borehole																	
<div style="display: flex; justify-content: space-between;"> <div> <p>* 2007 04 17</p> <p>▽ Water level observed during drilling</p> <p>▼ Water level measured after drilling</p> </div> <div> <p>20</p> <p>15 — 5</p> <p>10</p> </div> <div> <p>(%) STRAIN AT FAILURE</p> </div> </div>																		

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

CONT No
GWP No 5748-04-00

O'BRIEN STREET OVERPASS

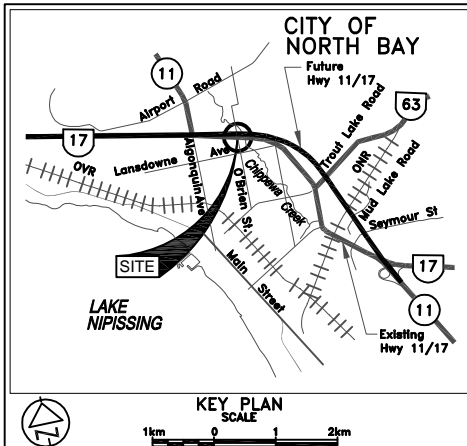
FUTURE HIGHWAY 11/17

BOREHOLE LOCATIONS AND SOIL STRATA



SHEET

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (Cone)		
	Borehole & Cone		
N	Blows/0.3m (Std. Pen Test, 475 J / blow)		
CONE	Blows/0.3m (60° Cone, 475 J / blow)		
	W L at time of investigation April 2007		
	Head		
	ARTESIAN WATER Encountered		
	PIEZOMETER		

BH No	ELEVATION	COORDINATES	
		NORTHINGS	EASTINGS
OS1	222.7	5 132 071	308 067
OS2	223.1	5 132 033	308 069
OS3	222.0	5 132 062	308 118
OS4	222.0	5 132 030	308 114

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

REVISIONS	DATE	BY	DESCRIPTION

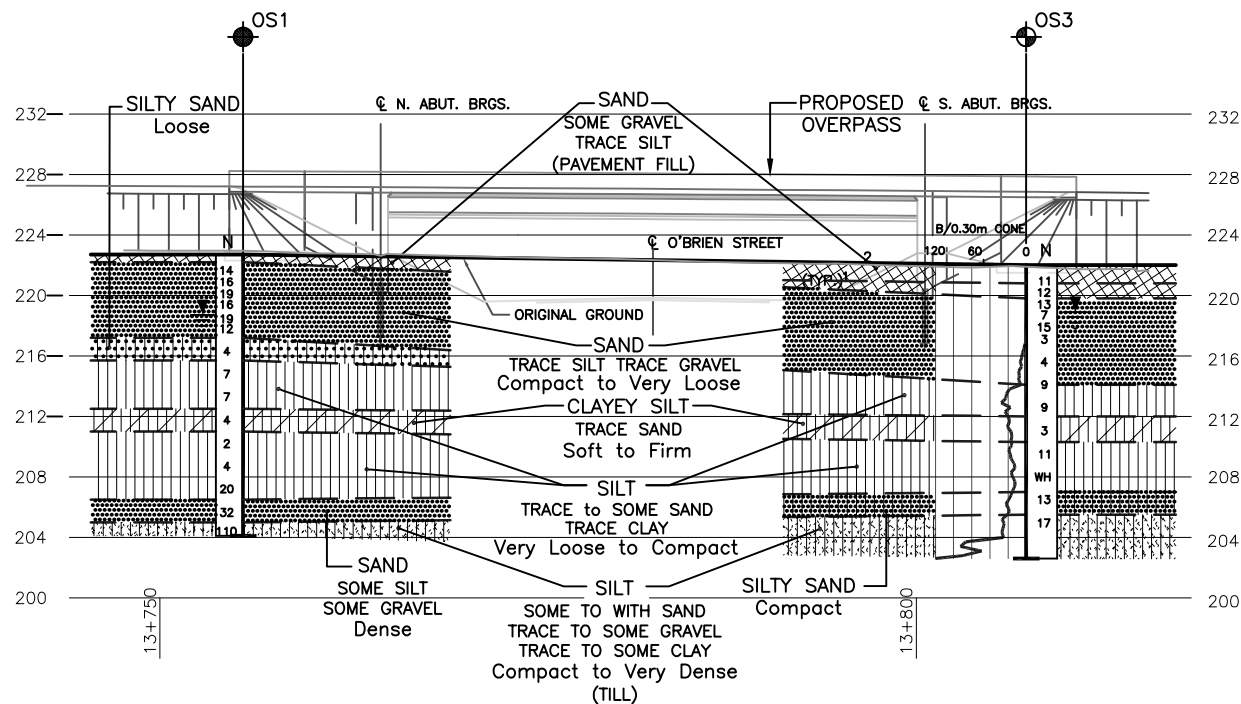
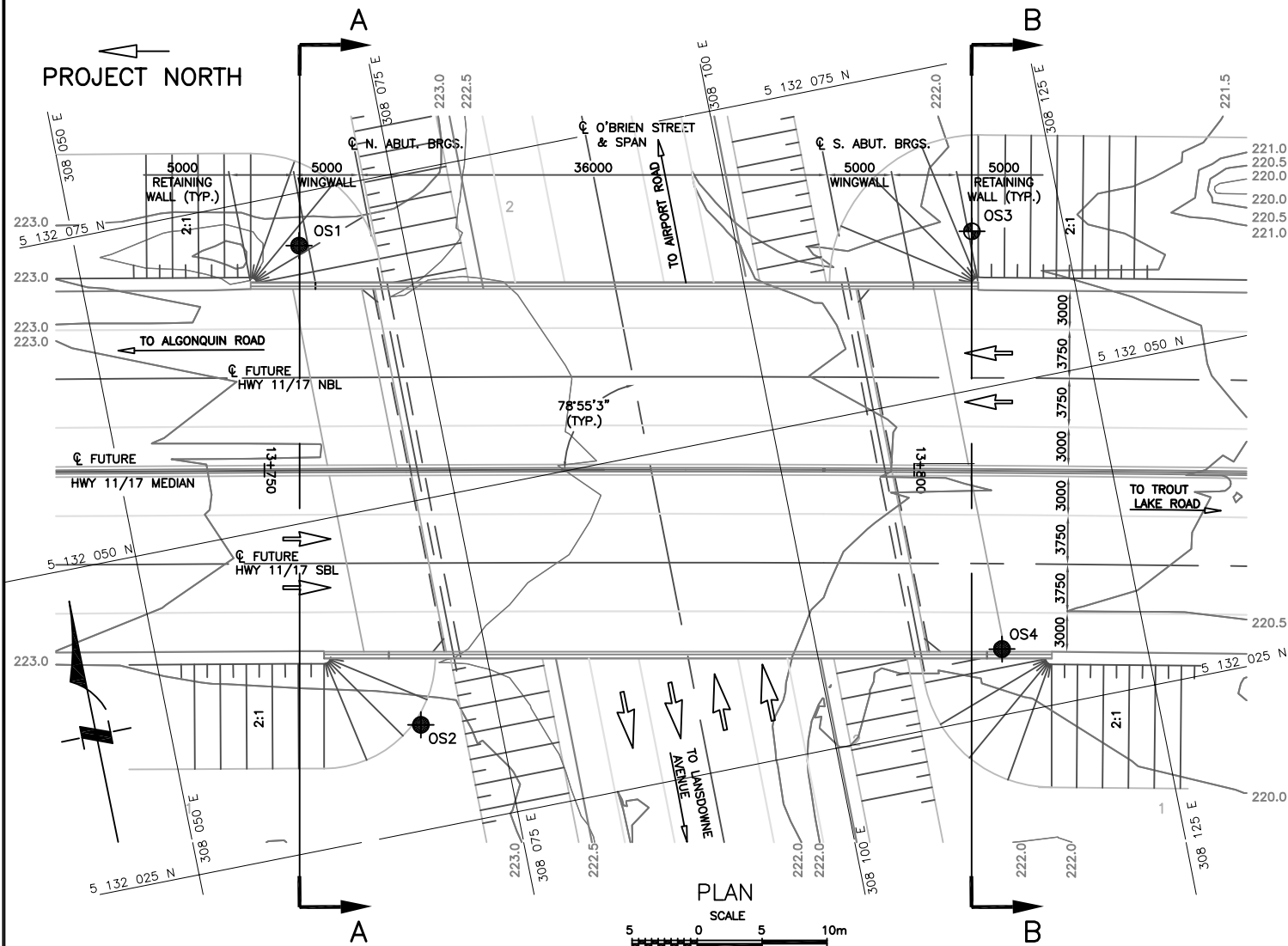
Geocres No. 31L-112

HWY No	11/17	DIST	54
SUBM'D	FP	CHECKED	GD
DRAWN	NA	CHECKED	CN
DATE	NOV. 02, 2007	APPROVED	BRG
SITE	---	DWG	OS-1



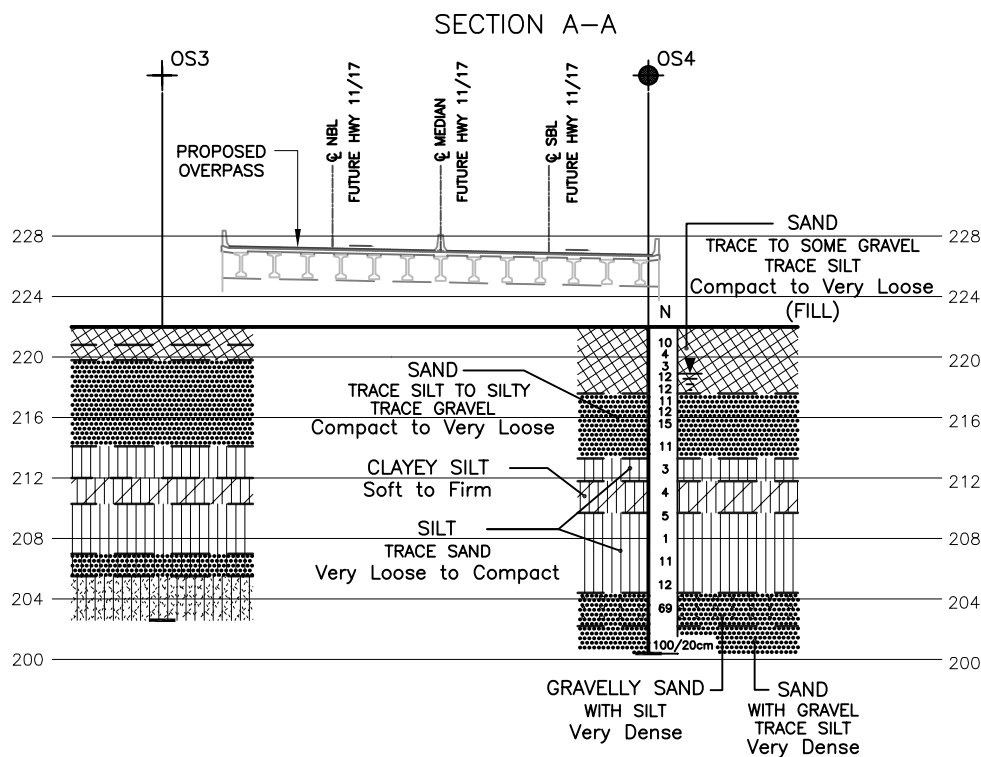
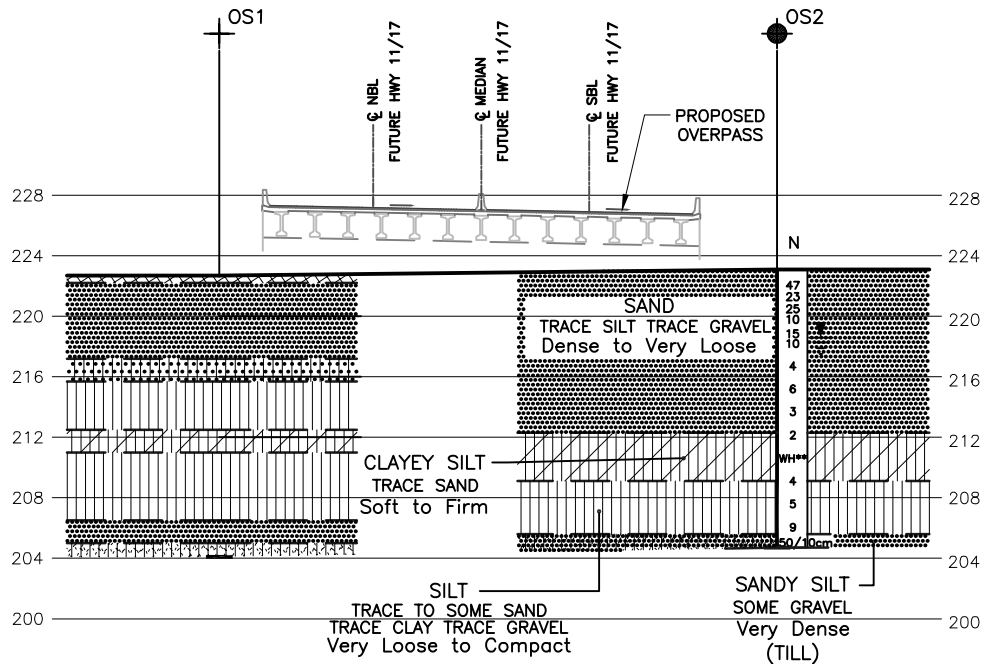
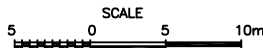
REF No.: STANTEC DRAWING: 65000580_O_BRIAN_B1.dwg
dated JULY 2007

PROJECT NORTH



NOTES:

- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- COORDINATES AT BOREHOLE LOCATIONS WERE BY DEL BOSCO SURVEYING LTD.





APPENDIX A

Site Photographs



Photograph 1: Looking north at existing Highway 11/17 from southeast corner of intersection with O'Brien Street. Note institutional land use (church). North abutment location is behind white pickup truck. [Note Project North direction.] (June 19, 2007)



Photograph 2: Looking southwest from southeast corner of existing Highway 11/17 and O'Brien Street. Note commercial land use. South abutment location is behind stopped cars. [Note Project North direction.] (June 19, 2007)



Photograph 3: Looking east at O'Brien Street from southwest corner of intersection with existing Highway 11/17. Dip in O'Brien Street (white car beside pole) is location of Chippewa Creek crossing. [Note Project North direction.] (June 19, 2007)



Photograph 4: Looking south at existing Highway 11/17 from southwest corner of intersection with O'Brien Street. Ground slopes down to Chippewa Creek valley beyond tree line. [Note Project North direction.] (June 19, 2007)