



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

**CHIPPEWA CREEK BRIDGE
LANSDOWNE AVENUE EXTENSION
FUTURE HIGHWAY 11/17
CITY OF NORTH BAY
GWP 5748-04-00
DISTRICT 54, SUDBURY, ONTARIO**

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TABLE OF CONTENTS

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

1. INTRODUCTION	1
2. SITE DESCRIPTION AND GEOLOGY	1
3. INVESTIGATION PROCEDURES	2
4. SUMMARIZED SUBSURFACE CONDITIONS	3
4.1 Topsoil	4
4.2 Sand / Gravelly Sand	4
4.3 Silt	5
4.4 Clayey Silt to Silty Clay	5
4.5 Bedrock	5
4.6 Groundwater	6
5. MISCELLANEOUS	6

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

6. ENGINEERING RECOMMENDATIONS	7
6.1 General	7
6.2 Foundations	8
6.2.1 Piles	8
6.2.2 Spread Footings	10
6.2.3 Caissons	10
6.3 Approach Embankments	10
6.4 Construction Considerations	11
6.4.1 Excavation	11
6.4.2 Groundwater Control	11
6.5 Lateral Earth Pressures	12



7. ADDITIONAL STUDIES	13
8. DISCUSSION OF FOUNDATION ALTERNATIVES	14
9. CLOSURE	15

Table 1 – List of Standard Specifications Referenced in Report

Figures PC-LA-1 and PC-LA-2 – Plasticity Charts

Figures GS-LA-1 to GS-LA-3 – Results of Grain Size Distribution Analyses

Explanation of Terms Used in Report

Record of Borehole Sheets

Drawing LA-1 – Borehole Locations and Soil Strata

Appendix A – Site Photographs

**PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT**

for
Chippewa Creek Bridge
Lansdowne Avenue Extension
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 a bridge over Chippewa Creek on a proposed extension of Lansdowne Avenue in North Bay, Ontario. The investigation was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The bridge construction is contemplated in connection with 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 bridge foundations and approaches within about 20 m of the abutments.

2. SITE DESCRIPTION AND GEOLOGY

The site is located on the envisaged extension of Lansdowne Avenue in North Bay. The bridge to be erected will carry Lansdowne Avenue traffic over Chippewa Creek. The alignment of the bridge is considered to be west-east.

Land use in the vicinity of the site comprises residential properties along Lansdowne Avenue. Photographs of the site are provided in Appendix A.



The topography on both sides of Chippewa Creek is relatively flat but sloping towards the creek valley. The inclination of the ground surface east of the creek is gentler and about 3 to 4 m lower than the west bank. The ground is covered with grass, trees and brushes. The native soils are typically represented by sand/soil and clayey deposits.

Chippewa Creek is about 5 m in width at the proposed bridge site and has eroded through the overburden deposits to form 2 to 5 m high banks. The creek flows southerly through the site.

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, about 10 m in the vicinity of the site.

3. INVESTIGATION PROCEDURES

The field subsurface investigation was carried out on April 26 and 27, 2007. Two sampled boreholes numbered LA1 and LA2 were put down at the site. The boreholes were drilled to respective depths of 6.6 and 7.3 m at the locations shown on Drawing LA-1, appended.

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). Due to the restricted site access and steep valley slopes, the boreholes were advanced using tripod equipment and BW washboring techniques under the full-time supervision of a member of our engineering staff. The modified penetration test resistance values were adjusted from a portable hammer (70 lb) to the standard penetration test N values.



Del Bosco Surveying Ltd. laid out and surveyed the borehole locations. Peto MacCallum Ltd. (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 from the boreholes continuously or at frequent depth intervals. The samples were obtained using a split spoon sampler in conjunction with standard penetration tests. The boreholes were backfilled in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

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. The water level observations are noted on the attached borehole logs.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, soil classification and laboratory testing. The laboratory test program comprised the following tests:

- Natural moisture content determinations (10)
- Grain size distribution analyses (5)
- Atterberg limits testing (3)

The results of laboratory natural moisture content determinations, grain size distribution analyses and Atterberg limits testing are shown on the Record of Borehole sheets. In addition, the results of Atterberg limits testing and grain size distribution analyses are presented in respective Figures PC-LA-1, PC-LA-2 and GS-LA-1 to GS-LA-3.

4. SUMMARIZED SUBSURFACE CONDITIONS

Refer to the Record of Borehole sheets for details of the subsurface conditions including soil classification, inferred stratigraphy, soil boundary levels and groundwater observations.



The borehole locations and the preliminary layout of the Chippewa Creek bridge as well as the longitudinal soil profile are presented on Drawing LA-1.

The soil stratigraphy revealed in the boreholes drilled at the site generally comprised surficial topsoil overlying sand / gravelly sand and clayey silt to silty clay. Cobbles were encountered in one borehole. Bedrock was inferred at depths of 6.6 and 7.3 m below grade. Groundwater was at 0.9 and 1.2 m depths (elevation 210.3 to 210.5) and reflects the water level of the adjacent Chippewa Creek.

4.1 Topsoil

Surficial topsoil was present in both boreholes. With a moisture content of about 38%, the topsoil was 200 to 300 mm thick and penetrated at elevation 211.2.

4.2 Sand / Gravelly Sand

Directly beneath the topsoil at depths of 0.2 and 0.3 m (elevation 211.2) in both boreholes was cohesionless sand / gravelly sand. This stratum was very loose to compact in relative density (SPT-N values of 2 to 12) and 12 to 20% in moisture content. Having a thickness of 2.5 m in borehole LA1 and 2.2 m in borehole LA2, the sand / gravelly sand was penetrated at respective depths of 2.7 and 2.5 m (elevation 208.7 and 209.0). It is noteworthy that cobbles were encountered in the unit in borehole LA1 put down at the west abutment.

A 1.7 m thick layer of sand was also revealed below a deposit of cohesive soils at 4.9 m depth (elevation 206.5) in borehole LA1. This layer extended to the termination depth on probable bedrock of 6.6 m (elevation 204.8).

The results of grain size distribution analyses performed on two samples of the sand and gravelly sand are presented in Figures GS-LA-1 and GS-LA-2 respectively.



4.3 Silt

Non-plastic silt with layers of silty clay was identified below the sand / gravelly sand at a depth of 2.5 m (elevation 209.0) in borehole LA2 advanced at the east abutment. This unit was loose (SPT-N value of 5) and had a moisture content of at least 23%. The silt was 900 mm thick and penetrated at 3.4 m depth (elevation 208.1).

4.4 Clayey Silt to Silty Clay

Underlying the sand at a depth of 2.7 m (elevation 208.7) in borehole LA1 and silt at 3.4 m depth (elevation 208.1) in borehole LA2 was a cohesive deposit of clayey silt to silty clay. This deposit had a thickness of 2.2 m in the former borehole and 3.9 m in the latter. The cohesive soils were very soft in consistency. The results of in-situ vane testing carried out in the deposit yielded an undisturbed shear strength of 8 to 10 kPa (soil sensitivity of 3 to 4). The cohesive soils were penetrated at depths of 4.9 and 7.3 m (elevation 206.5 and 204.2).

The results of Atterberg limits testing and grain size distribution analyses conducted on three cohesive soil samples are presented in respective Figures PC-LA-1, PC-LA-2 and GS-LA-3. The liquid and plastic limits of the clayey silt to silty clay were 29 to 35 and 18 to 21 respectively, thus giving the plasticity index values of 9 to 17. The moisture content of the deposit ranged from 43 to 48%.

4.5 Bedrock

Bedrock was inferred by refusal at a depth of 6.6 m (elevation 204.8) in borehole LA1 and 7.3 m depth (elevation 204.2) in borehole LA2. The bedrock type at the site is likely a metasedimentary gneiss, as indicated previously.



4.6 Groundwater

In the course of the field work, groundwater was observed in both boreholes. During and upon completion of drilling, groundwater was at a depth of 0.9 m (elevation 210.5) in borehole LA1 and 1.2 m (elevation 210.3) in borehole LA2. It is noted that the groundwater levels at the site are subject to seasonal fluctuations, precipitation patterns and the water level of the adjacent Chippewa Creek, which was at approximate elevation 210.4 at the time of the investigation.

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
Chippewa Creek Bridge
Lansdowne Avenue Extension
Highway 11/17
GWP 5748-04-00
North Bay

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 a bridge over Chippewa Creek along the proposed extension of Lansdowne Avenue in North Bay, Ontario.

The bridge will be a single span structure with a span length of 28.0 m and width of about 12.7 m (ref. Preliminary General Arrangement drawing prepared by Stantec in July 2007).

The road grade on Lansdowne Avenue at the bridge location will be near elevation 217.6 at the west abutment and 217.3 at the east abutment. The approach embankments to the structure are envisaged to be approximately 2 m high at the west abutment and 5 m high at the east abutment (interpreted from the road grade and ground surface elevations shown on the Stantec drawing referred to above). The water level in Chippewa Creek was at approximate elevation 210.4.

In summary, the soil stratigraphy revealed in the boreholes drilled at the site generally comprised surficial topsoil overlying very loose to compact sand / gravelly sand and very soft clayey silt to silty clay. Cobbles were encountered in one borehole. Bedrock was inferred at depths of 6.6 and 7.3 m (elevation 204.8 and 204.2) at the west and east abutments respectively. Groundwater was at elevation 210.3 to 210.5, near the water level in the Chippewa Creek.

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



It is noted that the upper sandy/silty soils in both boreholes drilled at the site are typically very loose to loose and the underlying very soft clayey silt / silty clay is highly compressible and extends to depths of about 5 and 7 m below grade. The cohesive deposit overlies either loose sand (west abutment) or probable bedrock (east abutment). Consequently, use of spread footings or caissons to support the foundation loads is not suitable for this structure. End-bearing piles driven to bedrock are considered to be the preferred foundation system for the bridge from a foundation engineering perspective.

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 scheme.

A list of the standard specifications referred in this report is enclosed in Table 1.

6.2 Foundations

6.2.1 Piles

Construction of integral abutments supported on H-piles driven to bedrock is considered to be feasible at this site. The H-piles should be driven to refusal on bedrock anticipated at the depths/elevations indicated in the following table:

LOCATION	BOREHOLE No.	DEPTH TO PROBABLE BEDROCK, m	BEDROCK ELEVATION
West Abutment	LA1	6.6	204.8
East Abutment	LA2	7.3	204.2



The recommended factored axial resistance at ultimate limit states (ULS) for three pile sections is as follows:

Pile Section	Factored Axial Resistance at ULS (kN)
HP 310 x 110	2,000
HP 310 x 132	2,400
HP 310 x 174	3,200

Cobbles were identified in the sand stratum in the borehole put down at the west abutment. Taking into account that the sand is very loose to compact, it is considered that damage to the piles when driving through cobbles/boulders is unlikely and, as a consequence, application of a reduction factor to account for potential damage during driving is not necessary.

The resistance at serviceability limit states (SLS) normally allows for 25 mm compression of the pile and founding medium. Considering the bedrock to be a non-yielding material, 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.

Negative skin friction due to consolidation of the native soils under the new embankment loads should be considered during Detail Design. Refer to further comments in Section 6.3 of this report.

The approach fill embankment as well as any fill placed below grade to deal with the soft compressible soils within the limits of the pile foundation should comprise Granular B Type II to enable driving of the piles and minimize the potential for damage during pile installation.

The need to provide the piles with driving shoes or rock points (OPSD-3000.100, 3000.201 and SP 903S01) to minimize the potential for damage when setting on the bedrock or driving through boulders should be assessed at the Detail Design stage.



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

Supporting the structure on conventional spread footings founded in the native soils is not considered feasible at this site due to the low bearing resistance available in the upper portion of the soils revealed at the bridge abutments.

6.2.3 Caissons

Installation of caissons founded on or socketed into bedrock will be difficult due to the high groundwater level, the presence of loose silt and pervious sand strata as well as of cobbles and potential 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 rockfill. The design calls for the embankment to be about 2 m high at the west abutment and 5 m high at the east abutment. The subgrade revealed in the boreholes drilled at the abutments consists of very loose to compact sand/silt underlain by very soft clayey soils.

It is considered that special procedures for construction of the embankment fill adjacent to the structure, similar to those required to construct the embankments over soft and/or compressible soils, are to be implemented to limit post-construction settlement and enhance the stability of the approach embankments. Excavation of the compressible clayey soils extending some 4 to 6 m below the water level in the nearby creek is not considered feasible since use of granular fill and extensive groundwater control measures would be required to backfill the excavation for subsequent pile driving and for adequate compaction of the fill.



We believe that preloading for a period of 6 to 12 months is the most suitable method to deal with the very soft clayey soils at the site. Provided the consolidation is essentially complete before pile installation, negative skin friction should not be a consideration during design of the piles.

The topsoil 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 approach embankment fill.

It is considered that the approach embankments constructed with earth or rockfill at respective inclinations of 2H:1V and 1.25H:1V in accordance with these recommendations will be stable. The factor of safety against a general shear failure of the embankment is computed to be about 1.3. The settlement of the approach fill and native soils induced by the embankment loads is estimated to be in the order of 50 to 100 mm. To minimize the post-construction settlements behind the abutments, the backfill should comprise granular materials.

6.4 Construction Considerations

6.4.1 Excavation

Excavation for construction of the pile caps is expected to extend through the upper very loose sand to a depth of up to 1 m and be relatively straightforward.

The very loose sand is classified as Type 4 material according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes over the full depth of excavation should therefore be inclined at 3H:1V.

6.4.2 Groundwater Control

Groundwater was near the water level in Chippewa Creek (elevation 210.4) during and upon completion of drilling. It is noteworthy that groundwater levels are subject to seasonal fluctuations and precipitation patterns.



It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations.

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 limited number of borehole data and a visual assessment of the bridge site, the recommended scope of work of the foundation investigation for Detail Design is as follows:

- Boreholes should be carried out at each end of the west and east abutments for a total of 4 boreholes. One of the boreholes at each abutment should be advanced to refusal on material with standard penetration N values in excess of 100 blows per 300 mm penetration and the other borehole should be advanced 3 m beyond refusal.
- One borehole should be advanced 20 m back from each of the west and east abutments for a total of 2 boreholes to a depth of twice the embankment fill height or to competent material to support the proposed fill.

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 the advantages and disadvantages of the alternatives is not considered to be required.

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 alternative also depends on other considerations such 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.



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



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



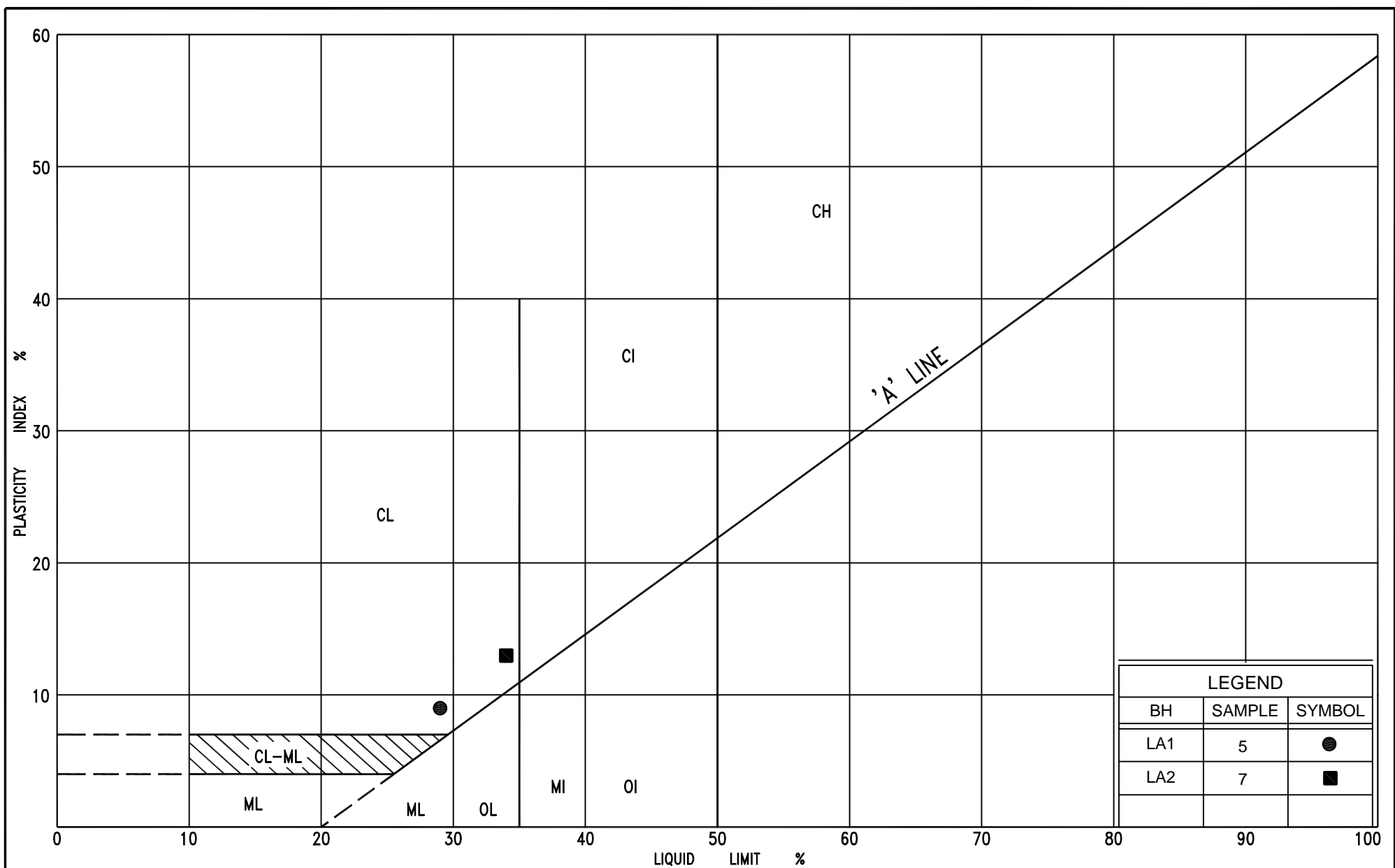
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GD/CN/BRG:gd-mi

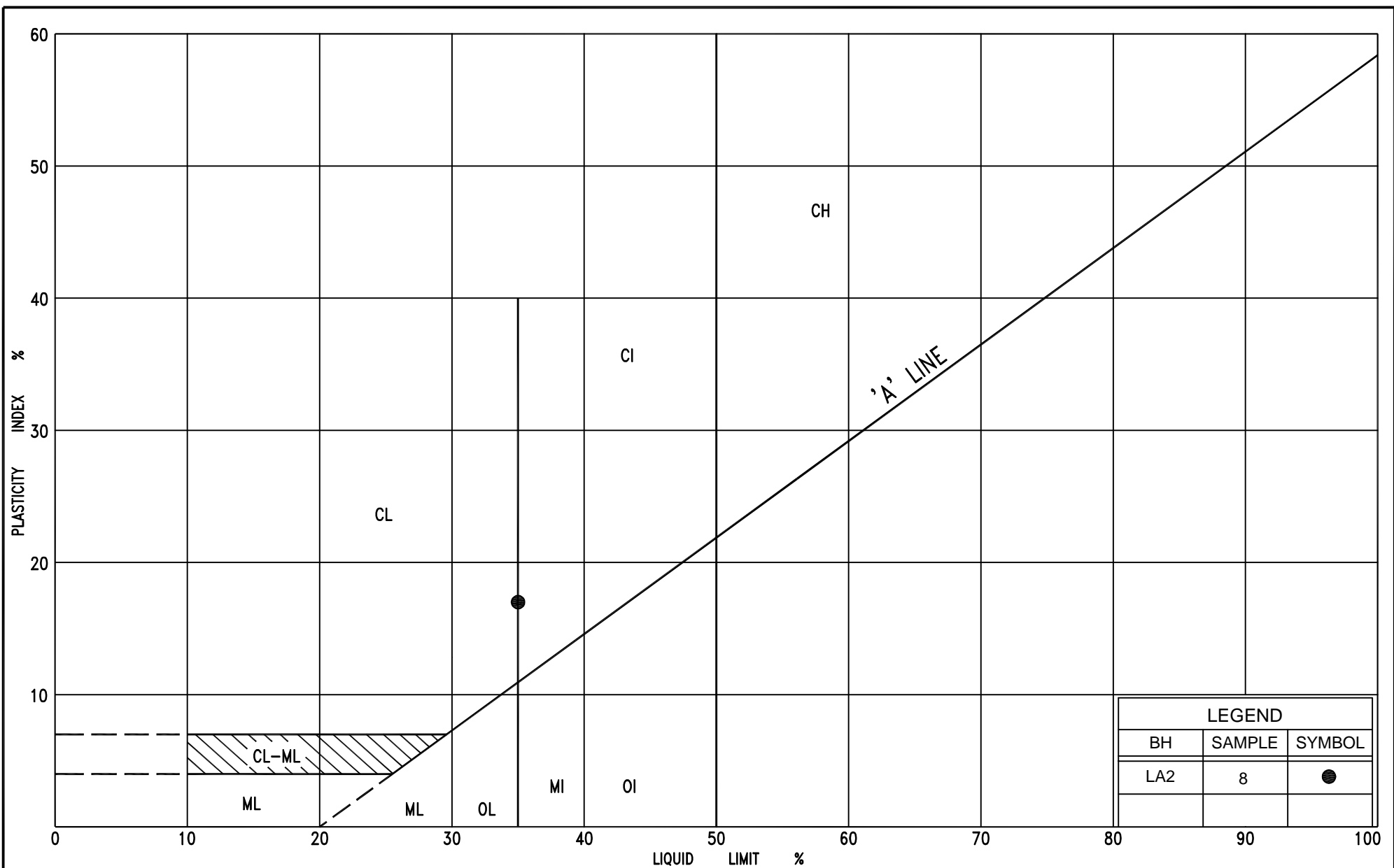




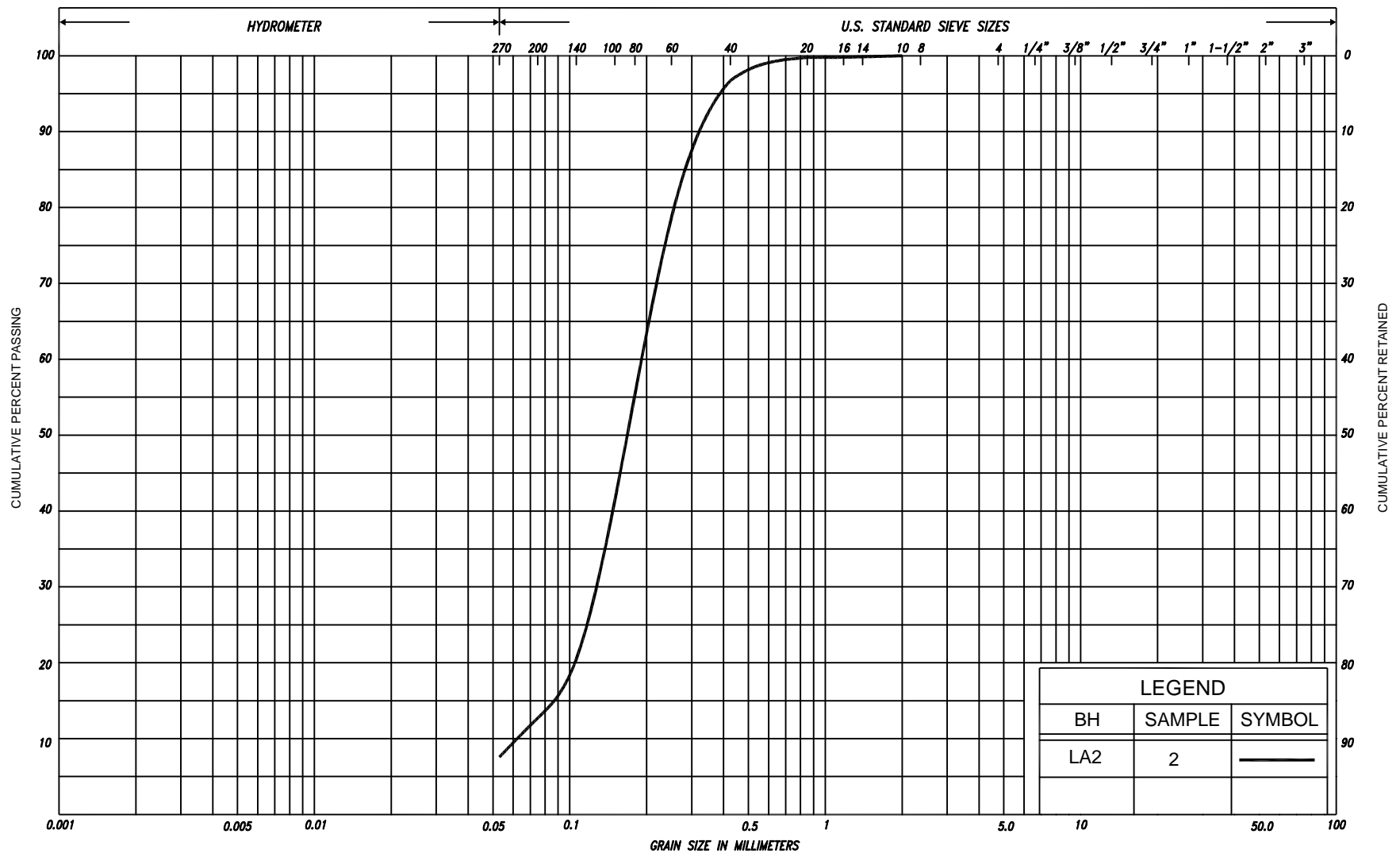
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
SP 405F03	Construction Specification for Pipe Subdrains	November 2006
SP 903S01	Construction Specification for Piling	November 2006
OPSD-3000.100	Foundation Piles Steel H-Pile Driving Shoe	November 2005
OPSD-3000.201	Oslo Points for Foundation, Piles, Steel HP310	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





LEGEND		
BH	SAMPLE	SYMBOL
LA2	8	●



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

GRAIN SIZE DISTRIBUTION

SAND, some silt

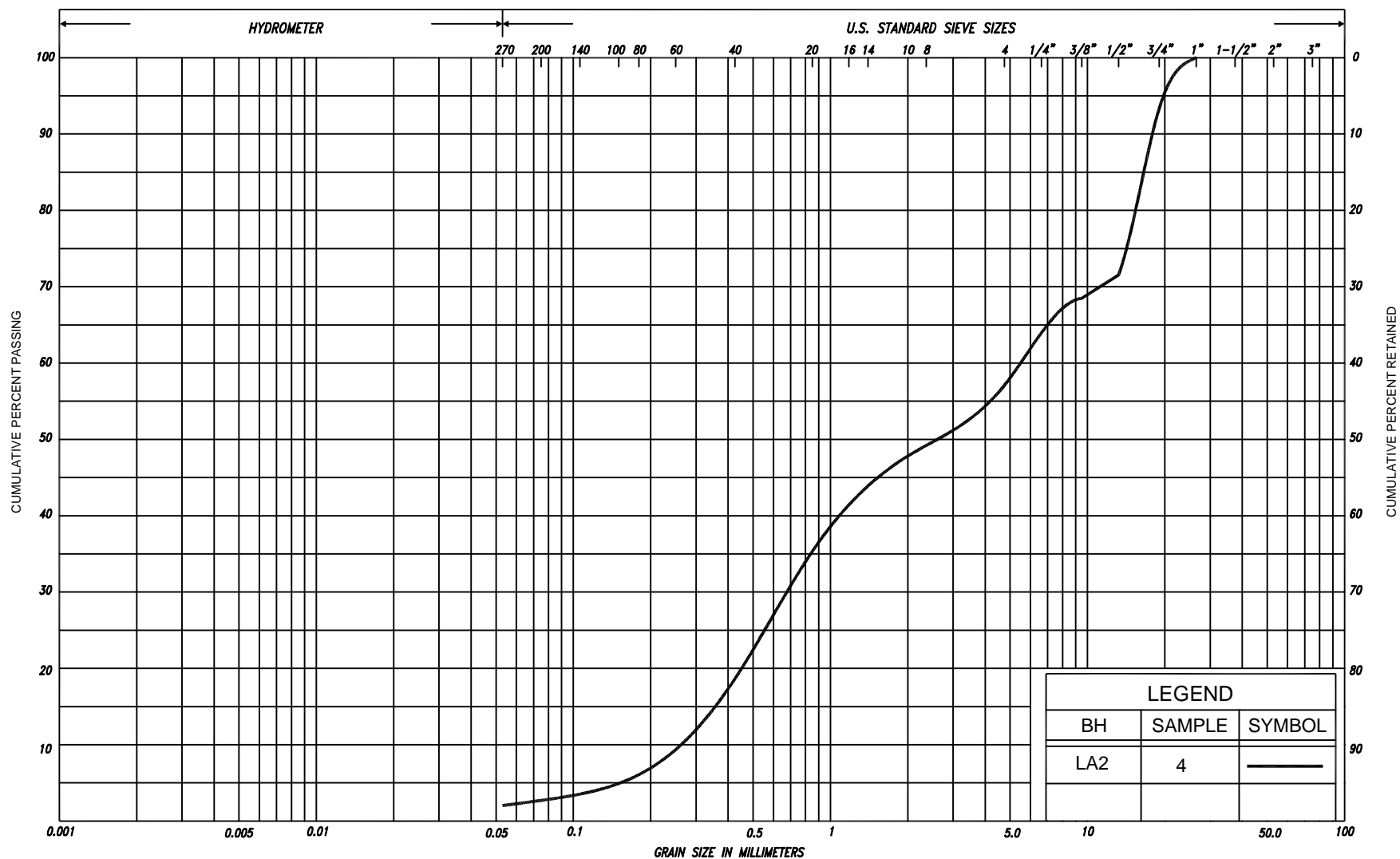
FIG No. GS-LA-1

HWY: 11/17

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



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

GRAIN SIZE DISTRIBUTION

GRAVELLY SAND, trace silt

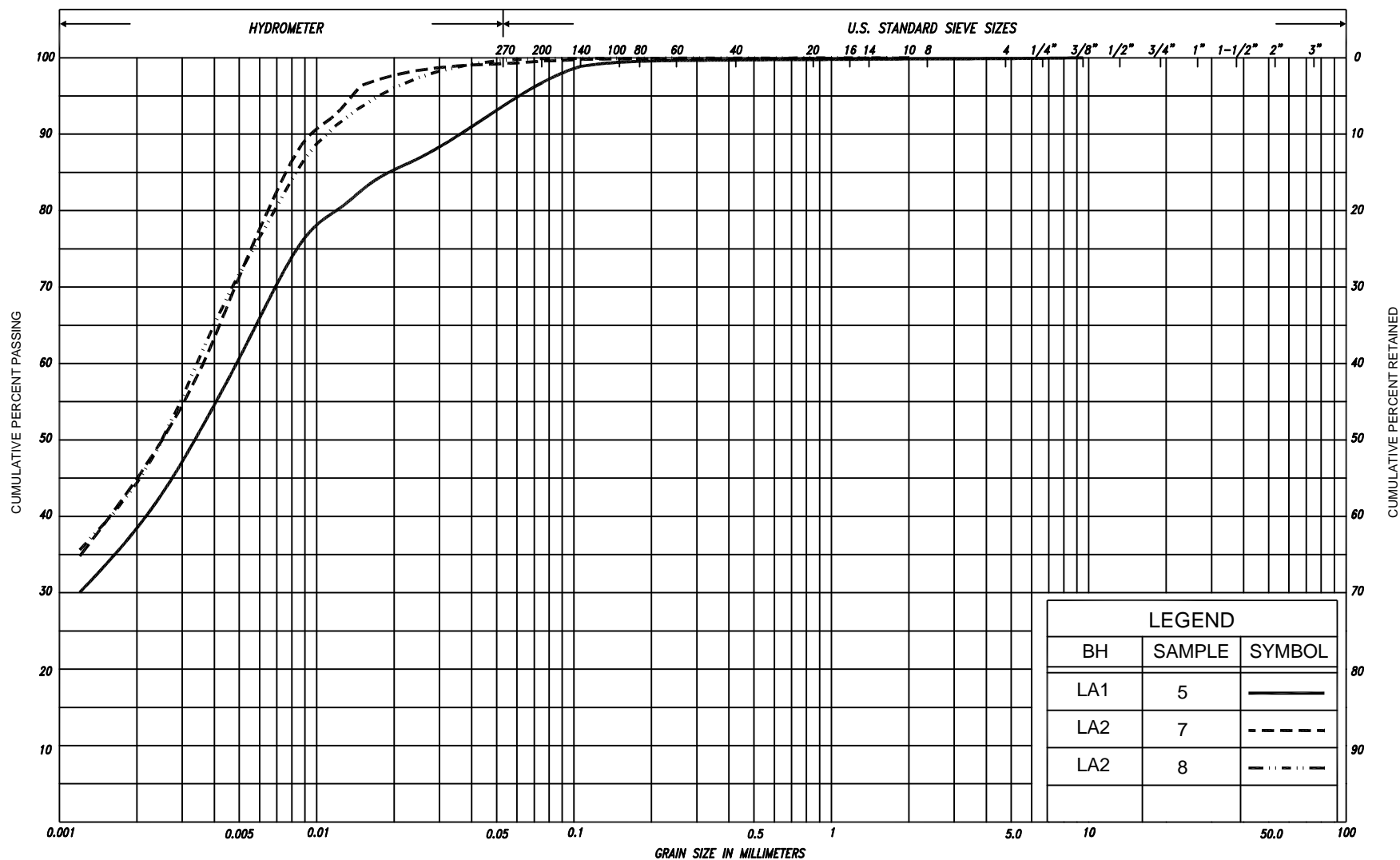


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FIG No. GS-LA-2

HWY: 11/17

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



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

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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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^3	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No LA1

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 131 740 N; 308 359 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE BW Wash Boring and Tripod COMPILED BY T.X.
DATUM Geodetic DATE April 26 and 27, 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									
								20	40	60	80	100					
								20	40	60	80	100					
211.4	Ground Surface																
0.0	Topsoil																
0.2	Sand, trace silt		1	SS	2		211										
	Very loose to Brown Wet compact with gravel		2	SS	12	▽* ▽*											
			3	SS	7		210										
	cobbles																
							209										
208.7																	
2.7	Clayey silt to silty clay trace sand varved		4	SS	3												
	Very soft Grey Wet		5	SS	3		208										0 3 59 38
				FV				+	4								
	with red brown layering						207										
			6	SS	3												
206.5																	
4.9	Sand with gravel, trace silt						206										
	Loose Brown Wet		7	SS	8												
							205										
204.8																	
6.6	End of borehole																
	Refusal on probable bedrock																
	Note: smaller diameter than MTO standard field vane used due to limitations of drilling equipment																
	* 2007 04 26&27																
	▽ Water level observed during drilling																
	▽ Water level measured after drilling																

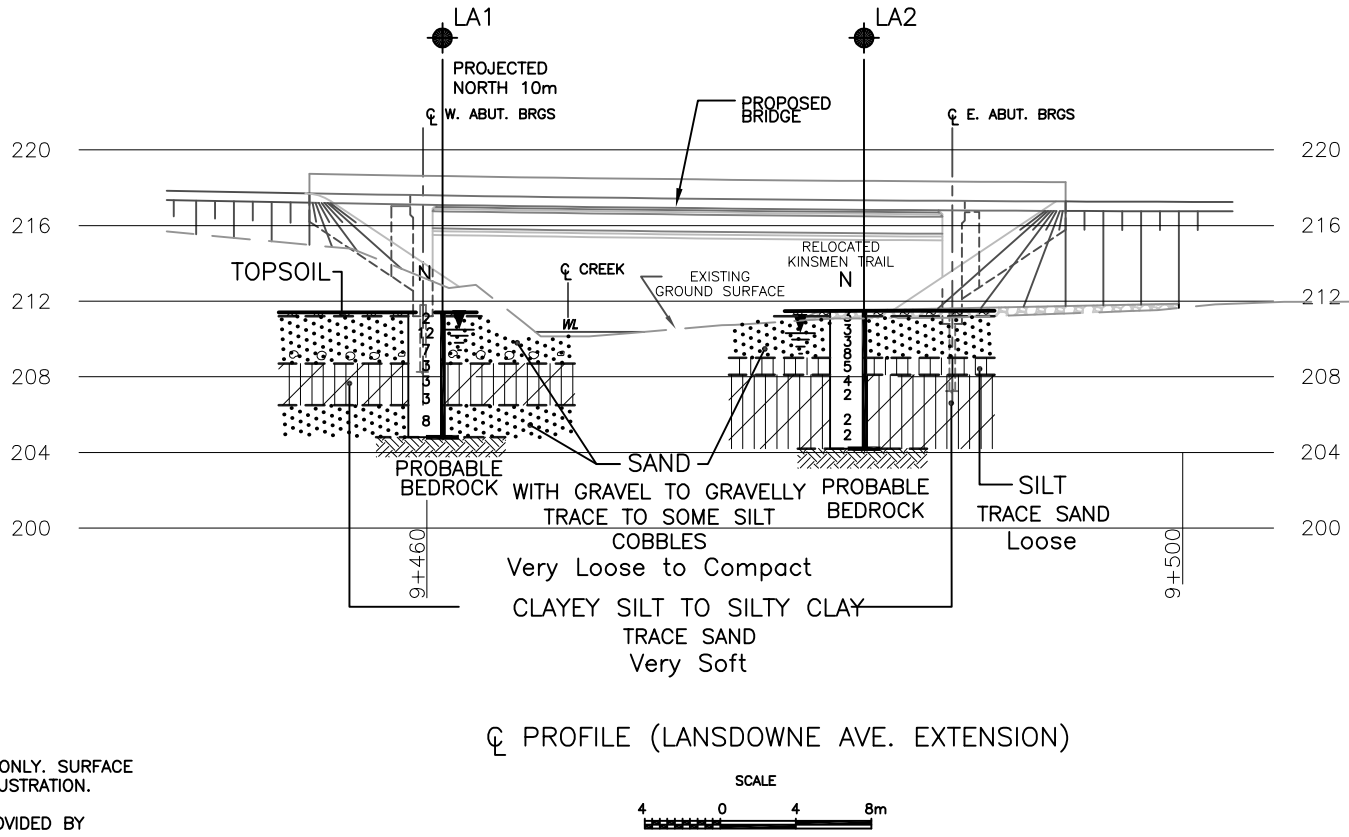
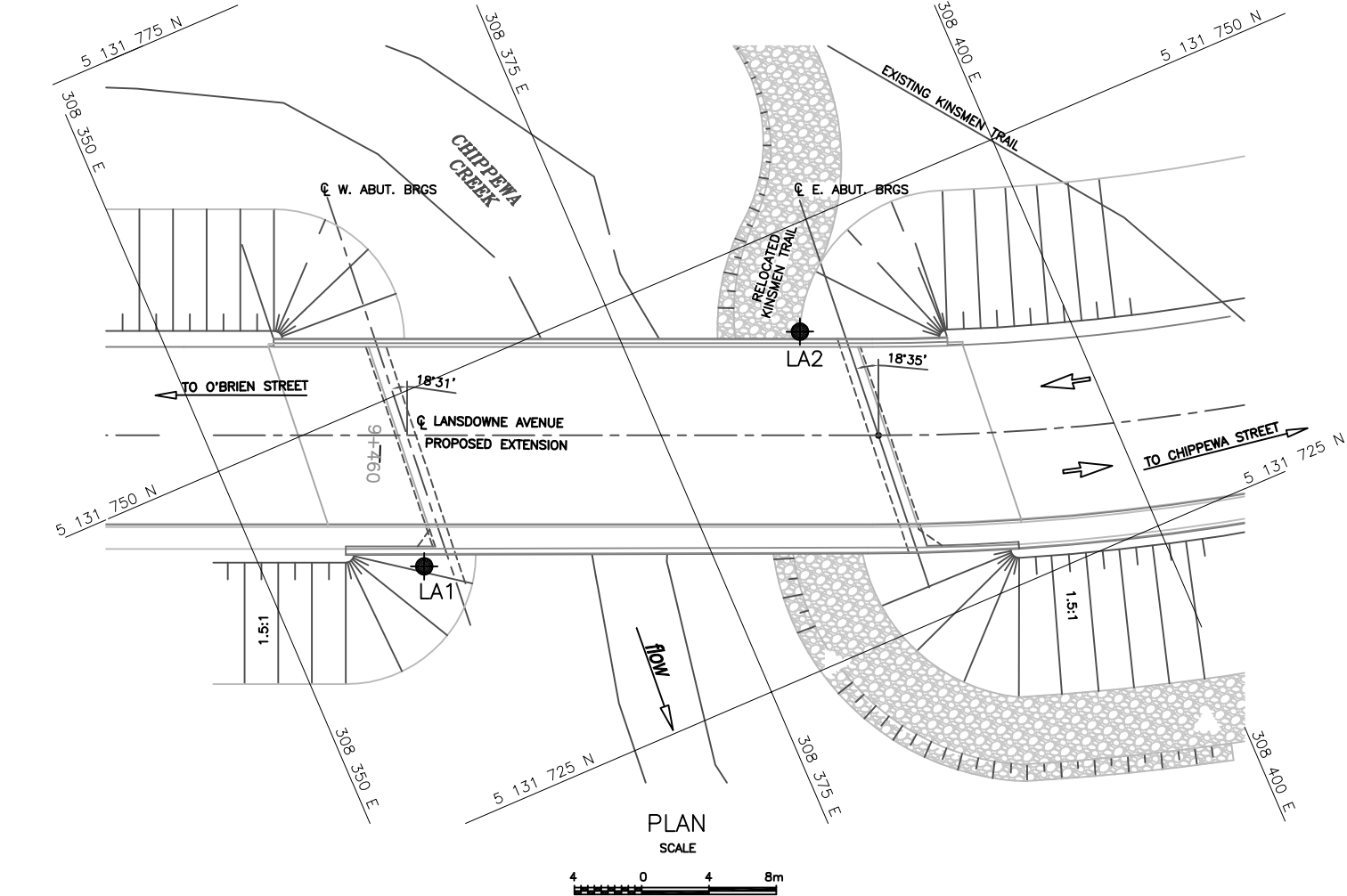
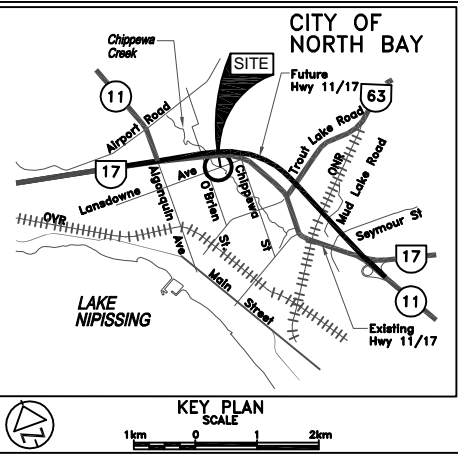
RECORD OF BOREHOLE No LA2

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 131 744 N; 308 385 E. ORIGINATED BY M.R.
 DIST Sudbury HWY 11/17 BOREHOLE TYPE BW Wash Boring and Tripod COMPILED BY T.X.
 DATUM Geodetic DATE April 26, 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					W _p	W	W _L					
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × LAB VANE												
					WATER CONTENT (%)															
211.5	Ground Surface							20	40	60	80	100								
0.0	Topsoil		1	SS	3	▼*▽*	211													
0.3	Sand, some silt		2	SS	3		210													
	Very loose Brown Wet		3	SS	3															0 87 (13)
	organic inclusions		4	SS	8															43 54 (3)
	gravelly, trace silt																			
	Loose																			
209.0	Silt, trace sand		5	SS	5		209													
	layers of silty clay																			
	Loose Grey Wet		6	SS	4															
208.1	Clayey silt to silty clay		7	SS	2		208													
3.4	trace sand																			
	varved																			
	Very soft Grey/ Wet			FV		207													0 1 54 45	
			8	SS	2		206													
				FV															0 1 55 44	
			9	SS	2		205													
204.2	End of borehole																			
7.3	Refusal on probable bedrock																			
	Note: smaller diameter than MTO standard field vane used due to limitations of drilling equipment																			
	* 2007 04 26																			
	▽ Water level observed during drilling																			
	▼ Water level measured after drilling																			

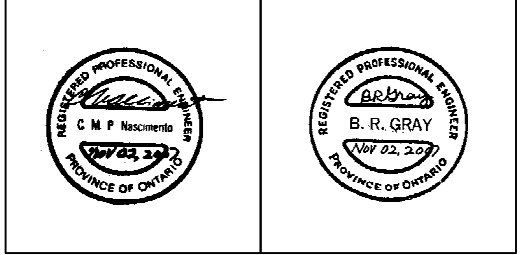


NOTES:
1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
2. COORDINATES AT BOREHOLE LOCATIONS WERE IPROVIDED BY DELBOSCO SURVEYING LTD.

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
LA1	211.4	5 131 740	308 359
LA2	211.5	5 131 744	308 385



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

DATE	BY	DESCRIPTION

Geocres No. 31L-113

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



APPENDIX A

Site Photographs



Photograph 1: Looking north from top of bank at current end of Lansdowne Avenue. Alignment of extension is proposed to the southeast (left of photograph). (June 19, 2007)



Photograph 2: View of existing steep west bank of the Chippewa Creek near the Lansdowne Avenue extension. Photograph was taken at middle of bank looking north. (July 24, 2007)



Photograph 3: Looking south at Chippewa Creek near the proposed bridge site. Note the steeper west bank and shallower east bank and shallow creek with numerous boulders. (July 24, 2007)



Photograph 4: Looking east from the east bank of Chippewa Creek along proposed alignment of Lansdowne Avenue. Note relatively flat terrain sloping up gently to the east. (July 24, 2007)