PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

# FOUNDATION INVESTIGATION AND DESIGN REPORT
for
INTERCHANGE CROSSING ROAD OVERPASS
HIGHWAY 11 NORTHBOUND LANES
SITE NO. 44-505/1
TOWNSHIP OF SOUTH HIMSWORTH
NORTH BAY AREA, ONTARIO
G.W.P. 323-00-00

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PML Ref.: 10TF013A-S2
Index No.: 315FIR and 316FDR
GEOCRES No.:  31L-161
January 18, 2013

**Peto MacCallum Ltd.**
CONSULTING ENGINEERS

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# for
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CONSULTING ENGINEERS

# FOUNDATION INVESTIGATION REPORT
for
Interchange Crossing Road Overpass, Highway 11 Northbound Lanes
Site No. 44-505/1
Township of South Himsworth
North Bay Area, Ontario
GWP 323-00-00

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## 1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed Highway 11 Northbound Lanes (NBL) Interchange Crossing Road Overpass at the existing Highway 11 NBL. The overpass is part of the new interchange at the south entrance to Powassan project extending from 5.7 km south of the Highway 534 northerly 5.0 km. The study was carried out by Peto MacCallum Ltd. (PML) for AECOM Canada Ltd (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed overpass over the proposed Interchange Crossing Road will be constructed along the existing Northbound Lanes of Highway 11 approximately between Station 20+840.2 and 20+872.2, Highway 11 NBL chainage, in the Township of South Himsworth (refer to AECOM Drawing No. 1, dated April 2011, Highway 11 NBL Interchange Crossing Road Overpass, General Arrangement –ALT 2A).

The purpose of this report was to summarize the subsurface stratigraphy encountered at the proposed structure and approaches within about 20 m of the abutments.

## 2. SITE DESCRIPTION AND GEOLOGY

The contemplated structure is proposed for the Highway 11 NBL about 1.2 km south of the existing Purdon Line / Main Street and Highway 11 NBL at-grade intersection in the Municipality of Powassan. The site is about 35 km south of the City of North Bay in the Geographic Township of South Himsworth.

Land use in the vicinity of the site includes the existing Highway 11 transportation corridor and farming activity east and west of the Highway 11 and scattered residential houses east of Highway 11 NBL. Gravel pits are present both east and west of the highway. The existing Highway 11 SBL is located west of Highway 11 NBL with wider than typical median of approximately 50 m. The local topography of the site is generally flat to the east and west of

Highway 11 NBL. The existing Highway 11 NBL embankment is about 2 to 3 m high. A TransCanada Pipe Lines Ltd. facility crosses Highway 11 approximately 300 m south of the proposed overpass location. Overhead and buried utilities are also present at the proposed bridge location. The ground cover includes grasses in the Highway 11 area and bushes and stands of trees elsewhere. Site Photographs of the structure location are attached in Appendix A.

The project site is located within the physiographic region known as the Number 11 Strip. The soil cover at the project site is from glaciofluvial outwash deposits of kame formation comprising sand and gravel soils locally with cobbles and boulders which overlies Precambrian age monzonitic (granitic) rock formation.

## 3. INVESTIGATION PROCEDURES

The field work for this study was carried out during the period of November 21 to 28, 2011. A total of six boreholes (ICN-1 to ICN-6) were drilled to 6.1 to 12.8 m at the locations shown on Drawing IN-1, appended. Two boreholes, ICN-2 and ICN-4 were added to investigate sloping bedrock conditions for the construction of deep foundations.

The structure control points were staked in the field by exp Geomatics according to the GA drawing dated April 2011 prepared by AECOM. The positions of the boreholes relative to the structure control points were selected at each foundation unit by PML allowing for drill rig accessibility and underground utilities and to minimize interference with the Highway 11 traffic lanes. Consequently, the boreholes ICN-1, ICN-2 and ICN-5 were drilled approximately 10 m away from intended borehole locations. Although the results of the investigations are considered representative, allowances should be made for variations in subsurface stratigraphy.

The ground surface elevations at the borehole locations were established by PML using the existing ground surface elevation at the structure control points as provided by Exp Geomatics. All elevations in this report are expressed in metres.

The boreholes were advanced using continuous flight hollow stem augers and ‘N’ casing through the soil cover with a track-mounted (Mooroka) D-120 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a PML field supervisor. Two boreholes ICN-3 and ICN-5 were extended 3.0 m into bedrock to 12.3 and 15.0 m depth using NQ diamond rock coring equipment supplemented by wash boring techniques. In addition, boulder coring was carried out in borehole ICN-3 from depth of 5.6 to 6.1 m using NQ diamond rock coring equipment.

Soil samples were recovered from the boreholes at regular 0.75 and 1.5 m depth intervals using the standard penetration test method. Standard penetration tests and field vane tests were conducted to assess the strength characteristics of the substrata. Because the presence of silt seams in the clayey deposits exceeded the field vane capacity, only a limited number of field vane tests were carried out. Pocket penetrometer tests were carried out in the clayey soil seams to obtain representative test results on the in-situ shear strength. Soils were identified in accordance with the MTO soil classification manual procedures.

The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, where encountered, by measuring the groundwater level in the open holes.

The boreholes were backfilled with a bentonite/cement mixture where required in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment.

The recovered soil samples were returned to our laboratory in Toronto for detailed visual examination, laboratory testing and classification. The laboratory testing program included the following tests:

- Natural moisture content determinations (61)
- Atterberg Limits (8)
- Grain size distribution analyses (16)

The laboratory grain size distribution charts are presented in Figures ICN-GS-1 to ICN-GS-5 and Atterberg Limits results are presented in Figures ICN-PC-1 to ICN-PC-3. All of the test results are summarized on the Record of Borehole sheets.

We also refer to the results of consolidation testing carried out on representative samples of cohesive clayey soils obtained for the design of sections of the embankments of the proposed

N-E/W and E/W-S ramps. These results were reported in the Foundation Investigation and Design report prepared by PML, Reference No. 10TF013A-H1.

## 4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole Sheets for details of the subsurface conditions including soil classifications, bedrock description, inferred stratigraphy, standard penetration test results and groundwater observations. The results of laboratory particle size distributions, Atterberg limits and moisture content determinations are also shown on the Record of Borehole Sheets.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawings IN-1 and IN-2. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised fill or surficial peat / topsoil overlying cohesionless silt to sand underlain by cohesive silty clay to clayey silt over cohesionless silt which in turn was underlain by silty sand till / sand till extending to bedrock. Cobbles and boulders were encountered within the till deposit. The bedrock / probable bedrock surface was contacted at 9.2 to 12.8 m (elevation 267.0 to 270.8) in four boreholes (ICN-2 to ICN-5). The remaining two boreholes (ICN-1 and ICN-6) drilled at the approaches were terminated by refusal on probable boulders at 6.1 and 12.0 m (elevation 267.1 and 274.1).

A summary of the findings is given below.

### 4.1 Fill

A 600 mm to 2.0 m thick fill was encountered surficially in boreholes ICN-2 to ICN-4. The unit extended to 0.6 to 2.0 m (elevation 278.2 to 279.2). The fill layer includes sand and gravel / silty sand over sand containing organics / topsoil and glass inclusions. N values varied from 3 to 11 indicating very loose to compact relative density.

The results of grain size distribution analysis for a sand fill sample are included in Figure ICN-GS-1. The moisture content results were 21 to 25%.

## 4.2 Peat / Topsoil

A 900 mm thick peat layer was encountered surficially in borehole ICN-5 extending to 0.9 m (elevation 279.1). The peat was fine fibrous.

A 300 and 500 mm thick topsoil was present at surface in boreholes ICN-1 and ICN-6 extending to 0.3 and 0.5 m (elevation 278.6 and 279.9). The moisture content results were 23 and 26%.

## 4.3 Silt to Sand

A cohesionless deposit with varying granular composition was encountered below the fill and topsoil / peat at 0.3 to 1.4 m (elevation 278.2 to 279.9) in all of the boreholes except borehole ICN-4. The deposit was 0.3 to 1.2 m thick extending to 1.4 to 2.1 m (elevation 277.7 to 278.8). N values ranged from 5 to 11, locally 1 in borehole ICN-5 indicating loose to compact relative density with local very loose conditions. The moisture content determinations ranged from 22 to 29%.

## 4.4 Silty Clay to Clayey Silt

A cohesive deposit of silty clay to clayey silt containing silt layers was encountered below the cohesionless silt to sand deposits at 1.4 to 2.1 m (elevation 277.7 to 278.8) in all of the boreholes. The stratum was 3.5 to 5.5 m thick extending to 5.3 to 6.9 m (elevation 272.2 to 274.9).

N values typically ranged from 5 to 11. Lower N values of 2 to 4 may be due to layers of wet silt in the clayey soils and were not considered to be representative of the actual soil consistency. In-situ vane testing conducted in boreholes within the cohesive deposit indicated shear strength values 88 to greater than 100 kPa, with sensitivity values of 7. In-situ vane test results are influenced by silt layers within the clayey soils stratum. Penetrometer test results varied from 50 to 138 kPa and confirmed the range of insitu vane test results. The stratum was considered to be typically stiff to very stiff consistency with local firm layers.

The results of grain size distribution analysis for silty clay and clayey silt samples are included in Figures ICN-GS-2 and ICN-GS-3. The plasticity charts are presented in Figures ICN-PC-1 and ICN-PC-2. The liquid limit of silty clay samples ranged from 35 to 41 and the plastic limit ranged from 21 to 23 with plasticity index values of 13 to 18. The liquid limit of clayey silt samples was 28 and 31 and the plastic limit was 19 and 22 with plasticity index values of 6 to 12. The moisture content determinations ranged from 26 to 44%.

Based on consolidation test results carried out for the N-E/W and E/W-S ramps of the interchange on cohesive soils with similar characteristics (Reference PML Report No. 10TF013A-H1), it is inferred that the underlying native cohesive clayey soils were subjected to preconsolidation pressures of 570 to 1,000 kPa. The measured initial void ratios ($e_o$) were 0.82 and 1.3, compression index ($C_c$) were 0.4 and 0.7 and coefficient of consolidation ($C_v$) were 1.7 and 1.9 $m^2$/month.

## 4.5 Silt

A cohesionless deposit of silt was encountered below the cohesive silty clay/clayey silt deposit at 5.3 to 6.9 m (elevation 272.2 to 274.9) in all of the boreholes except borehole ICN-3. The deposit was 0.7 to 2.7 m thick extending to 6.1 to 9.6 m (elevation 269.5 to 274.1). N values ranged from 8 to 17 indicating loose to compact relative density.

The results of grain size distribution analysis for a silt sample are included in Figure ICN-GS-4. The plasticity chart is presented in Figure ICN-PC-3. The liquid and plastic limits of a silt sample were 22 and 19, respectively with plasticity index value of 3. The moisture content determinations ranged from 27 to 41%.

## 4.6 Silty Sand Till / Sand Till

A till deposit with varying granular composition was encountered below the silt in boreholes ICN-1, ICN-2, ICN-4 and ICN-5 at 6.2 to 9.6 m (elevation 269.5 to 274.0) and below the clayey silt in borehole ICN-3 at 5.6 m (elevation 274.0). The till deposit was 2.4 to 6.1 m thick, extending to 9.2 to 12.8 m (elevation 267.0 to 270.8). Cobbles and boulders were encountered within this deposit. N values ranged from 21 to 54 and 50 to 87 for 10 to 30 cm sampler penetration. The deposit was typically found to be dense with local compact to very dense layers.

The results of grain size distribution analysis for sand till samples are included in Figure ICN-GS-5. The moisture content determinations ranged from 4 to 25%.

## 4.7 Cobbles and Boulders

Two layers of cobbles and boulders were encountered in the boreholes.

An upper 0.5 and 0.6 m thick layer of cobbles and boulders was encountered below the silt and clayey silt deposit within the upper zone of the sand till stratum at 7.0 and 5.6 m (elevation 272.8, 274.0 and 274.1) in boreholes ICN-2, ICN-3 and ICN-6, respectively. Boulder coring was also carried out in borehole ICN-3 from of 5.6 to 6.1 m (elevation 274.0 to 273.5). The layer extended to 7.6 and 6.1 m (elevation 272.2 and 273.5) in boreholes ICN-2 and ICN-3, respectively and borehole ICN-6 was terminated on a boulder of this deposit at 6.1 m (elevation 274.1).

A lower layer of cobbles and boulders was also encountered within the till deposit in boreholes ICN-1, ICN-2, ICN-4 and ICN-5 at 8.2 to 11.3 m (elevation 267.8 to 271.8). Borehole ICN-1 was terminated by refusal on boulders at 12.0 m (elevation 267.1). The lower layer of cobbles and boulders was at least 0.7 to 2.4 m thick extended at least to 9.2 to 12.8 m depths (elevation 267.0 to 270.8) in the remaining boreholes.

## 4.8 Bedrock

Granite bedrock surface was contacted / inferred at 11.7 and 12.8 m (elevation 267.0 and 267.9) at the south abutment boreholes ICN-2 and ICN-3. At the north abutment boreholes ICN-4 and ICN-5, the bedrock was contacted / inferred at 9.2 and 11.1 m (elevation 269.1 and 270.8).

The bedrock surface between borehole locations slopes at angles of 1.7° (boreholes ICN-2 and ICN-3) and 3.1° (boreholes ICN-4 and ICN-5) on the transverse direction at the abutments. On a north-south direction, the bedrock slopes at approximately 6.8° (boreholes ICN-2 and ICN-5) and 2.1° (boreholes ICN-3 and ICN-4). Steeper angles may occur along the bedrock surface slopes between borehole locations.

The summary of the depth to bedrock and elevations is provided in following table:

| **Foundation Element** | **Borehole** | **Depth to Bedrock (m)** | **Bedrock Elevation** | |
|---|---|---|---|---|
| | | | **Refusal** | **Cored** |
| South Approach | ICN-1 | >12.0 | <267.1 | – |
| South Abutment | ICN-2 | 12.8 | 267.0 | – |
| | ICN-3 | 11.7 | – | 267.9 |
| North Abutment | ICN-5 | 9.2 | – | 270.8 |
| | ICN-4 | 11.1 | 269.1 | – |
| North Approach | ICN-6 | >6.1 | <274.1 | – |

The measured core recovery was 100%. The RQD determined from the rock cores was in a range of 75 to 100%, thus indicating a good to excellent quality rock.

The granite bedrock exhibited high strength and was typically found to be slightly weathered to unweathered, locally containing highly weathered zones in borehole ICN-3.

A detailed description of the rock cores retrieved from boreholes ICN-3 and ICN-5 is given in Table A, appended. Photographs of the rock cores are shown in Appendix B.

### 4.9 Groundwater

During augering, groundwater was observed at 0.4 to 0.6 m (elevation 278.7 to 279.7) in boreholes ICN-1, ICN-2 and ICN-4. Upon completion of drilling, groundwater was measured at 1.9 m (elevation 278.3) in borehole ICN-4. No water was encountered in borehole ICN-6. The remaining boreholes ICN-3 and ICN-5 were charged with drilling water for rock coring and their water levels would not be representative. Based on the natural water content profile in the boreholes, it is estimated that the water level at the site is in the range of 2.5 to 3.8 m (elevation 276.2 to 277.6). The groundwater level is subject to seasonal fluctuation and rainfall patterns.

## 5. CLOSURE

Mr. F. Portela carried out the field investigation for this study under the supervision of Mrs. N. S. Balakumaran, P. Eng., and Mr. C. M. P. Nascimento, P. Eng., Project Manager. Walker Drilling Ltd. supplied the drill rig for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This Foundation Investigation Report was prepared by Mrs. N. S. Balakumaran, P. Eng., and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C. M. P. Nascimento, P. Eng., Project Manager conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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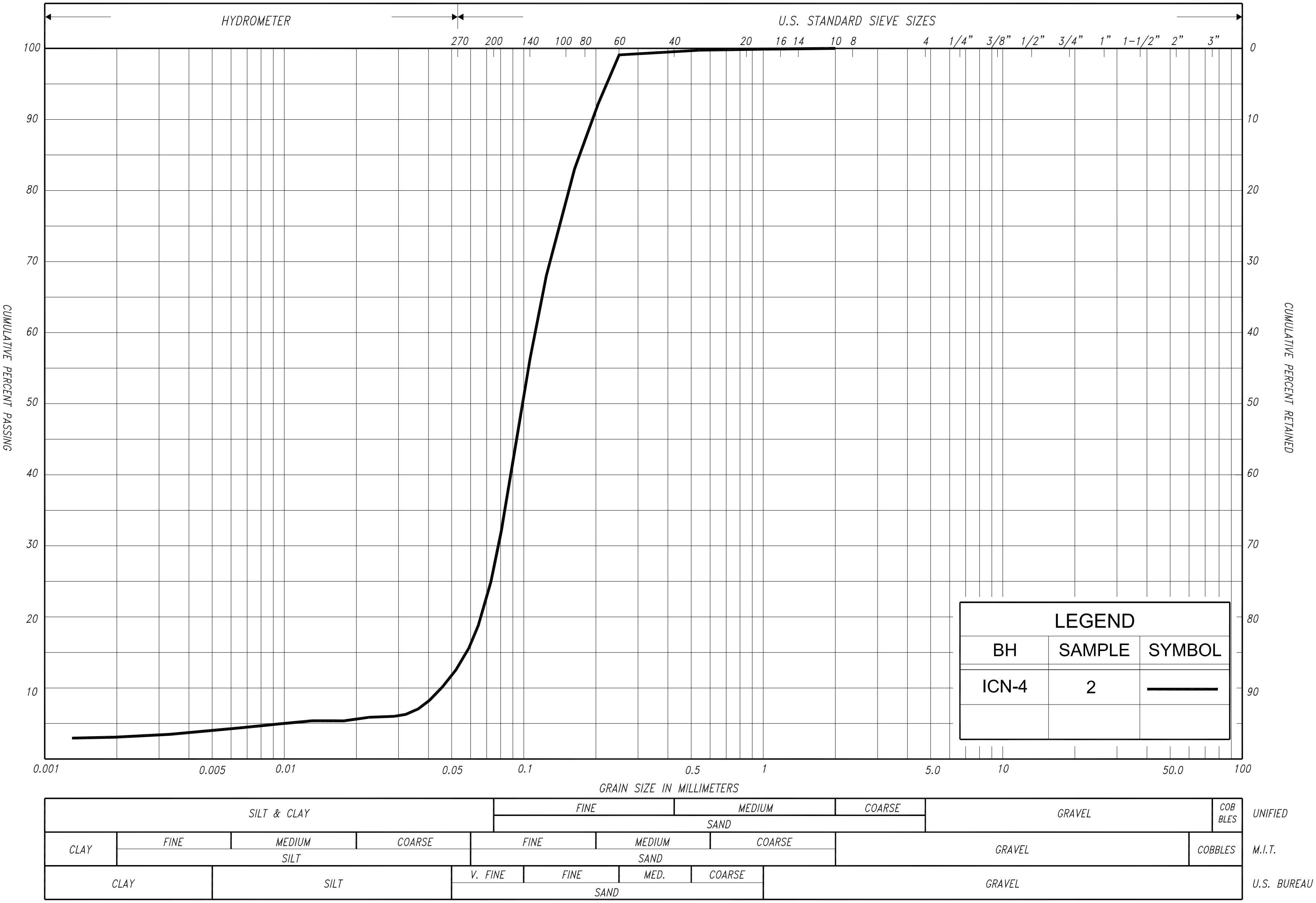
# TABLE A

## ROCK CORE DESCRIPTIONS

| **CORE RECOVERY** | | | | | **CORE DESCRIPTION** | |
|---|---|---|---|---|---|---|
| **HOLE NO.** | **CORE NO.** | **DEPTH (m)** | **RECOVERY (%)** | **RQD (%)** | **DEPTH (m)** | **DESCRIPTION** |
| ICN-3 | 13 | 11.7 – 12.0 | 100 | 75 | 11.7 – 15.0 | GRANITE: Pink. medium crystalline, garnetiferous, high strength, slightly weathered to unweathered with highly weathered zone, close to moderate spaced flat to dipping cross joints, rough planar, tight to sandy, with 200 mm thick layer of pegmatite at 13.0 m, pink, coarse crystalline, dipping lower contact, friable, silty to sandy, good to excellent quality. |
| | 14 | 12.0 – 13.5 | 100 | 82 | | |
| | 15 | 13.5 – 15.0 | 100 | 79 | | |
| ICN-5 | 13 | 9.2 – 10.4 | 100 | 99 | 9.2 – 12.3 | GRANITE: Pink, medium crystalline, garnetiferous, high strength, unweathered to slightly weathered, wide spaced flat cross joints, rough planar, tight to slightly altered with silt on surface, excellent quality. |
| | 14 | 10.4 – 12.0 | 100 | 100 | | |
| | 15 | 12.0 – 12.3 | 100 | 100 | | |

NOTE: RQD = Rock Quality Designation

Originated: JFW
Compiled: FP
Checked: NB / CN

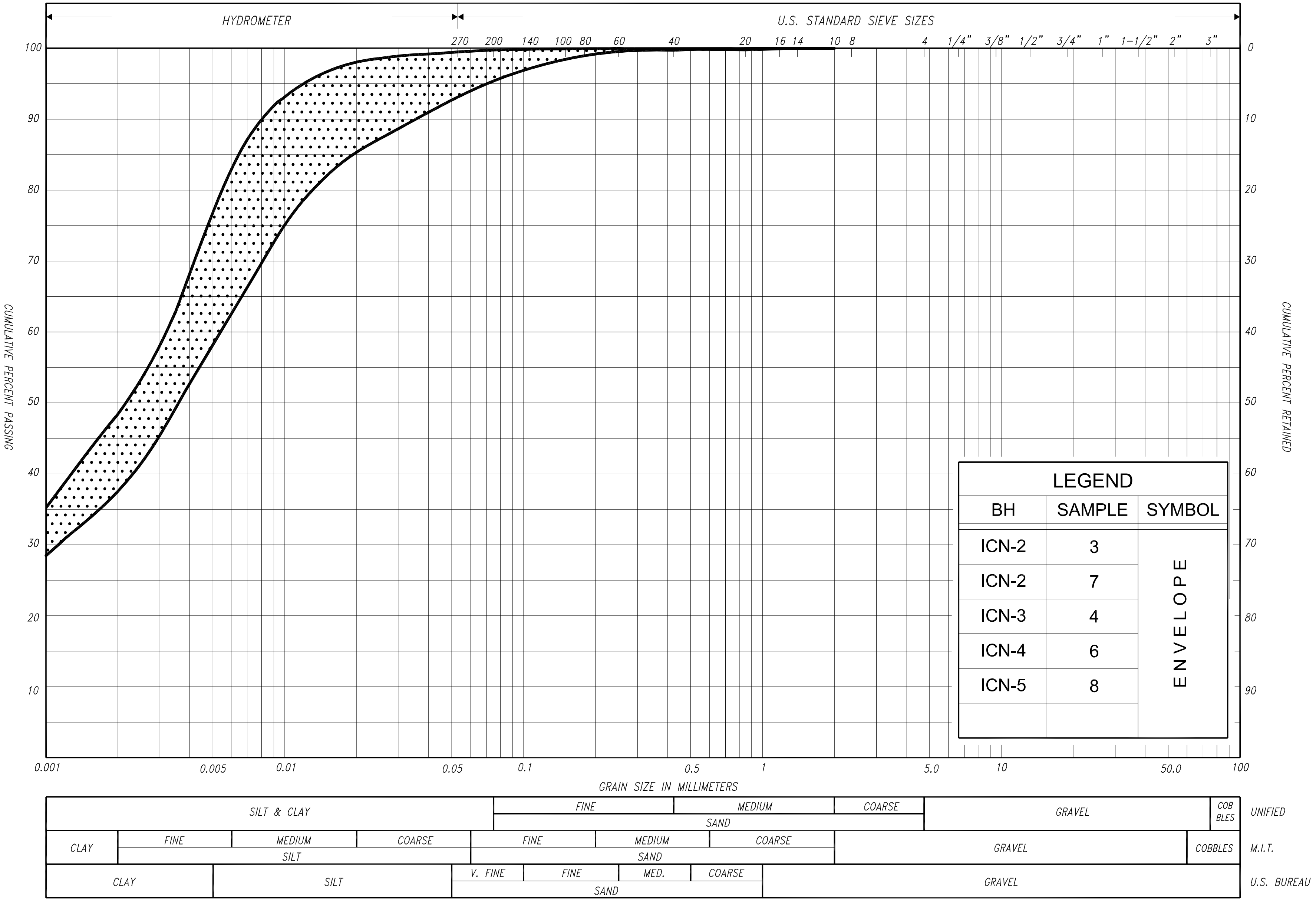


Ontario

# GRAIN SIZE DISTRIBUTION

SAND, with silt, trace clay
(FILL)

FIG No. ICN-GS-1
HWY: 11
G.W.P. No. 323-00-00



LEGEND

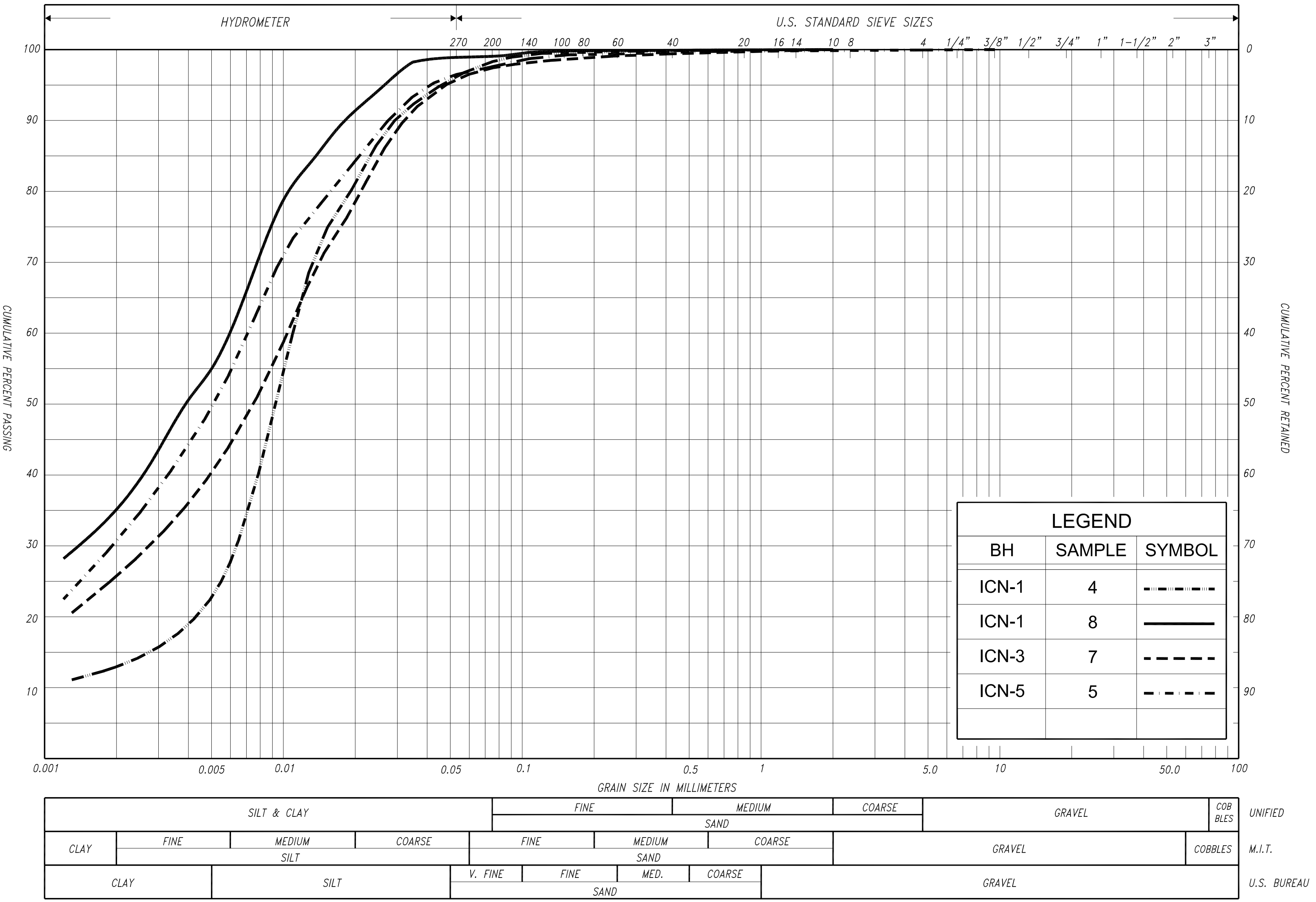
| BH | SAMPLE | SYMBOL |
|---|---|---|
| ICN-2 | 3 | ENVELOPE |
| ICN-2 | 7 | |
| ICN-3 | 4 | |
| ICN-4 | 6 | |
| ICN-5 | 8 | |

Ontario

# GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand (CI)

| | |
|---|---|
| FIG No. | ICN-GS-2 |
| HWY: | 11 |
| G.W.P. No. | 323-00-00 |

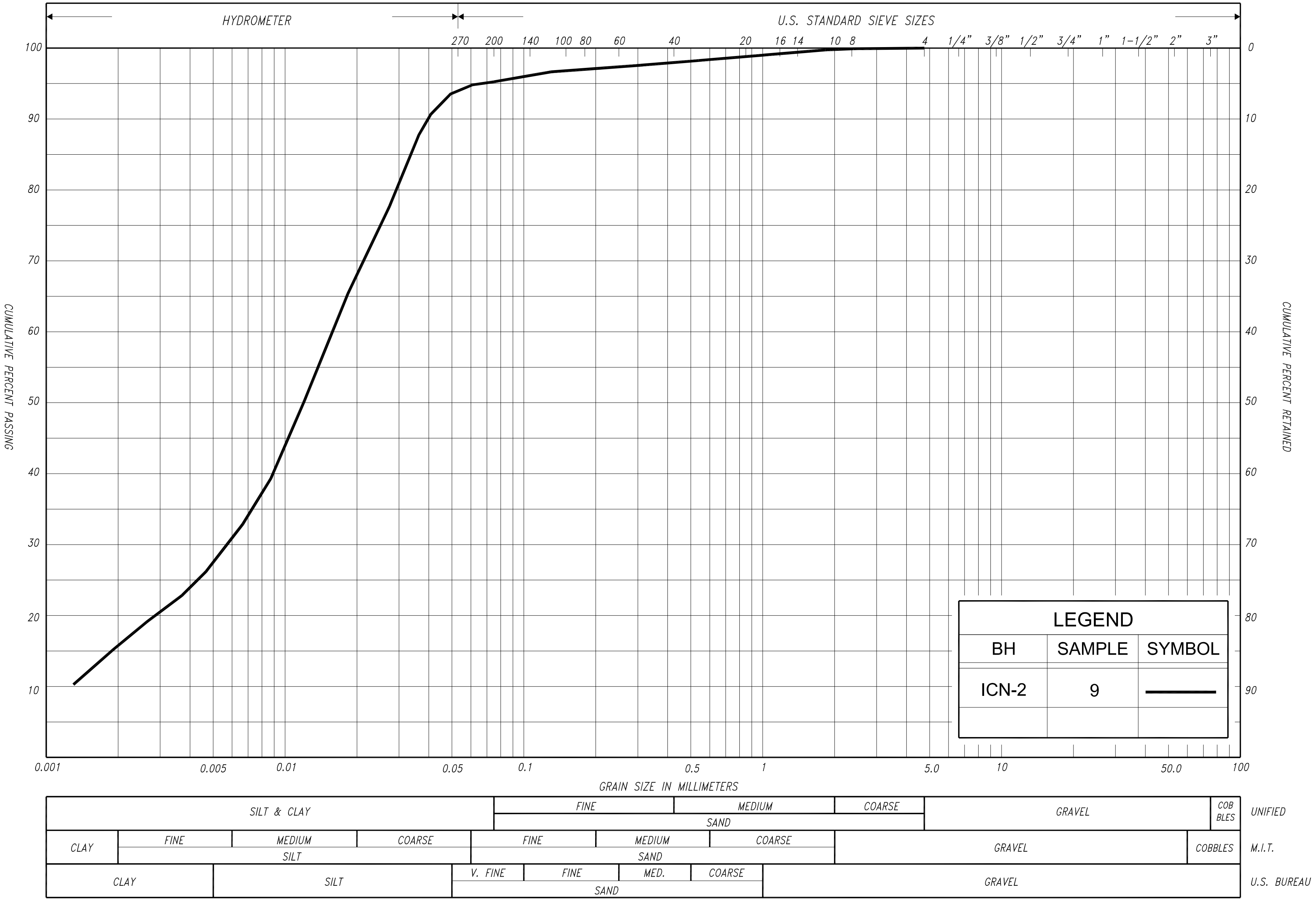


Ontario

# GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand (CL)

FIG No. ICN-GS-3
HWY: 11
G.W.P. No. 323-00-00

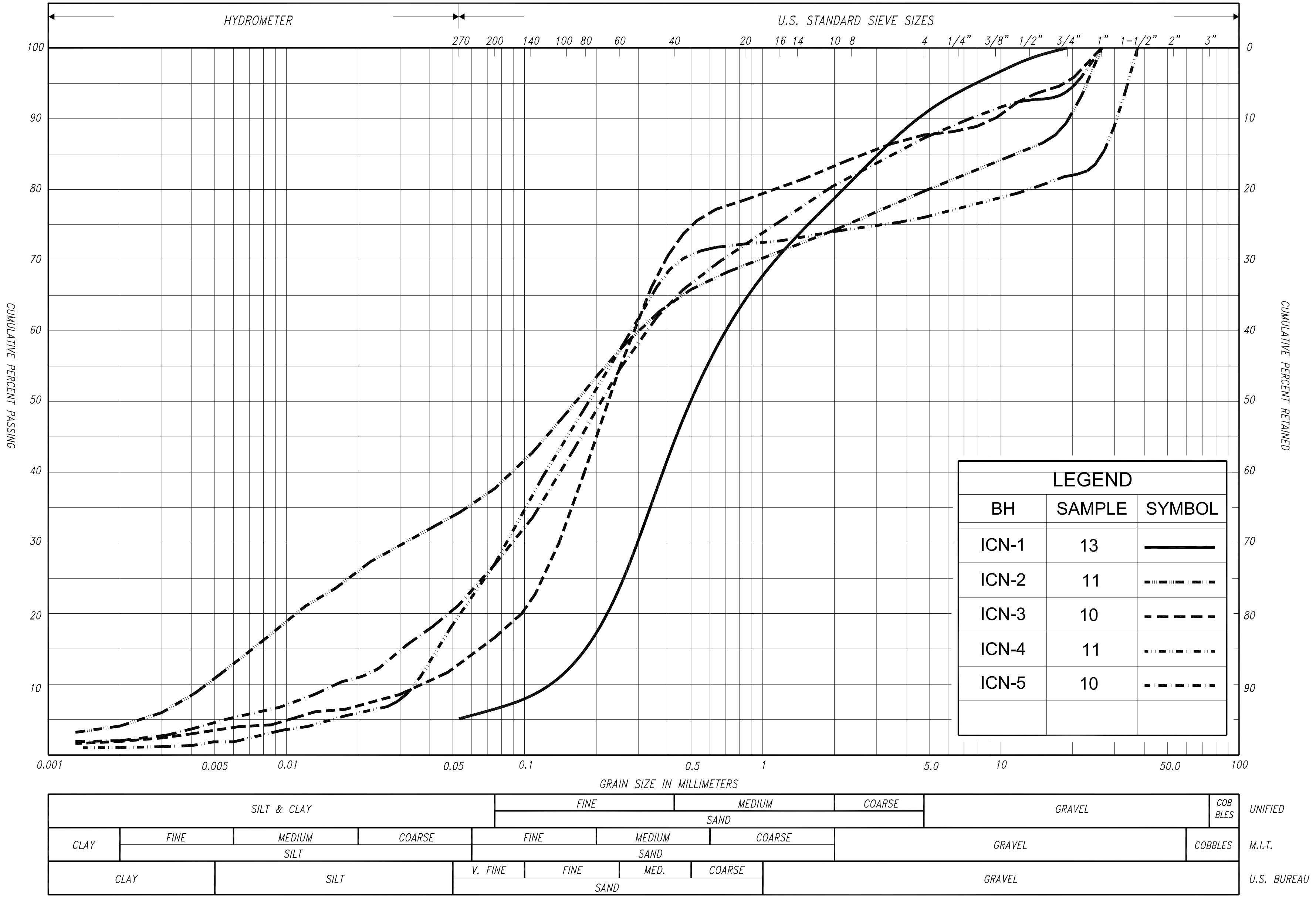


Ontario

# GRAIN SIZE DISTRIBUTION

SILT, some clay, trace sand (ML)

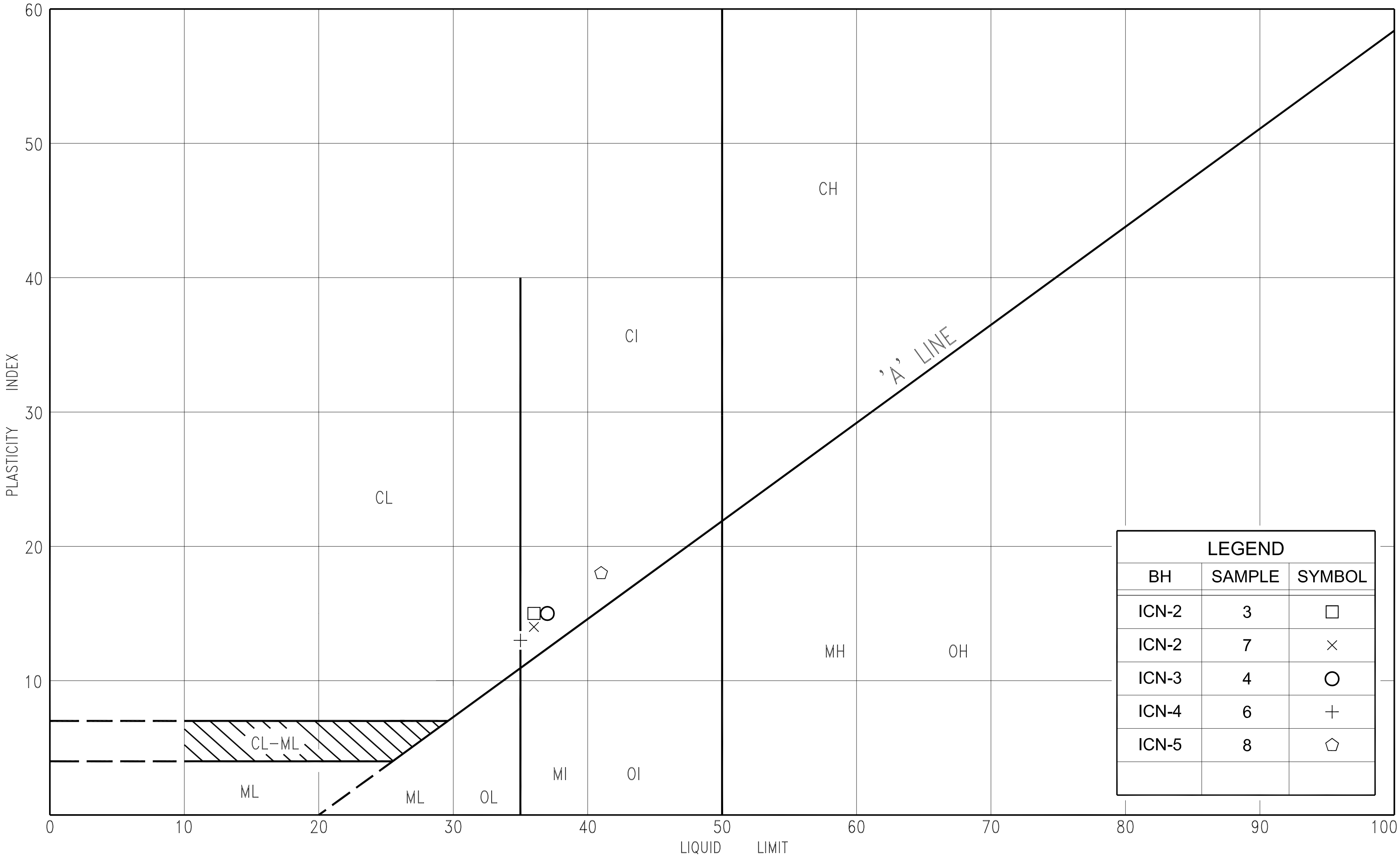
FIG No. ICN-GS-4
HWY: 11
G.W.P. No. 323-00-00



# GRAIN SIZE DISTRIBUTION

SAND, trace to with silt, trace to with gravel, trace clay
(TILL)

FIG No. ICN-GS-5
HWY: 11
G.W.P. No. 323-00-00



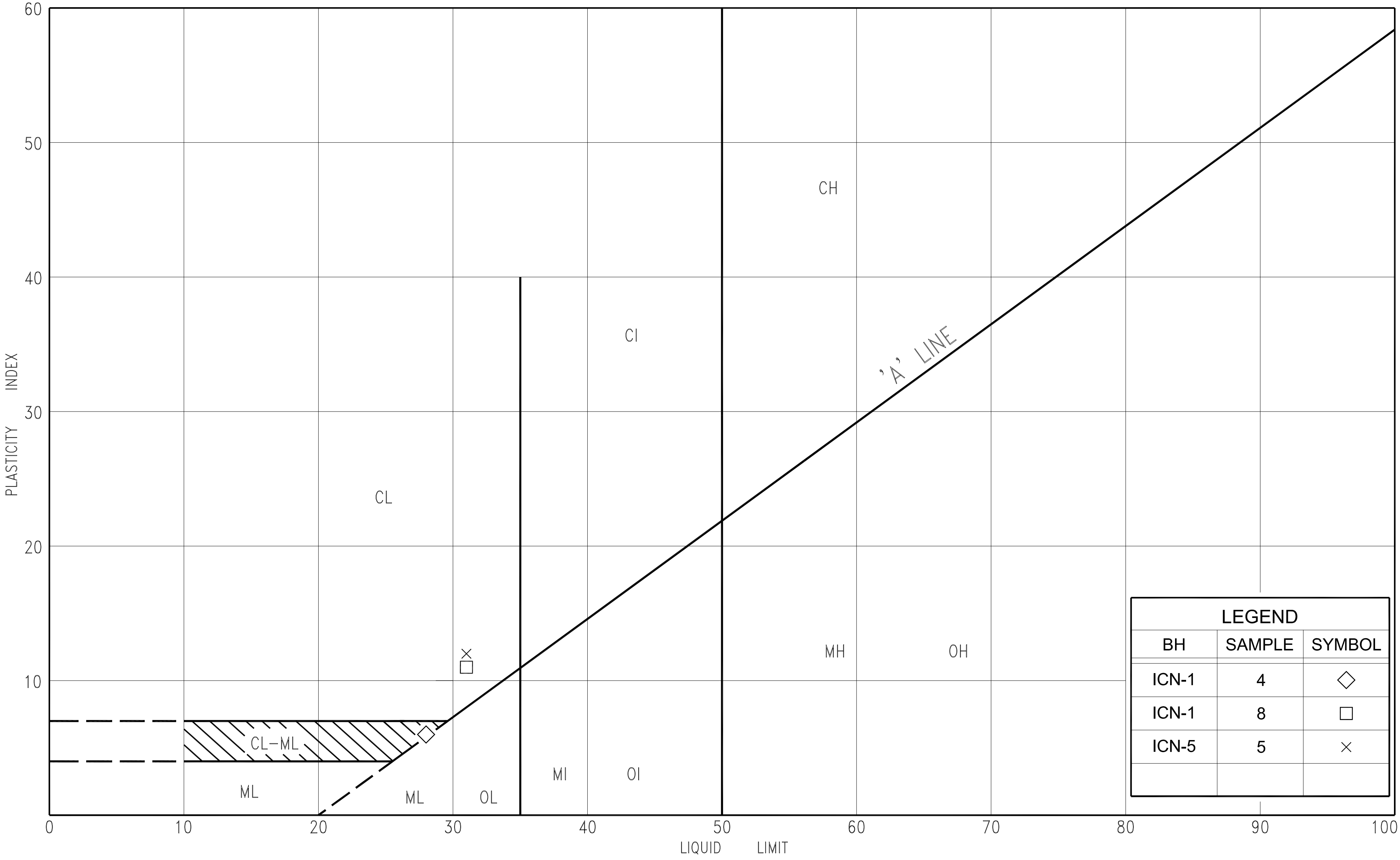
LEGEND

| BH | SAMPLE | SYMBOL |
|---|---|---|
| ICN-2 | 3 | □ |
| ICN-2 | 7 | × |
| ICN-3 | 4 | ○ |
| ICN-4 | 6 | + |
| ICN-5 | 8 | ⬠ |

# PLASTICITY CHART

## SILTY CLAY, trace sand (CI)

| | |
|---|---|
| FIG No. | ICN-PC-1 |
| HWY: | 11 |
| G.W.P. No. | 323-00-00 |



# PLASTICITY CHART

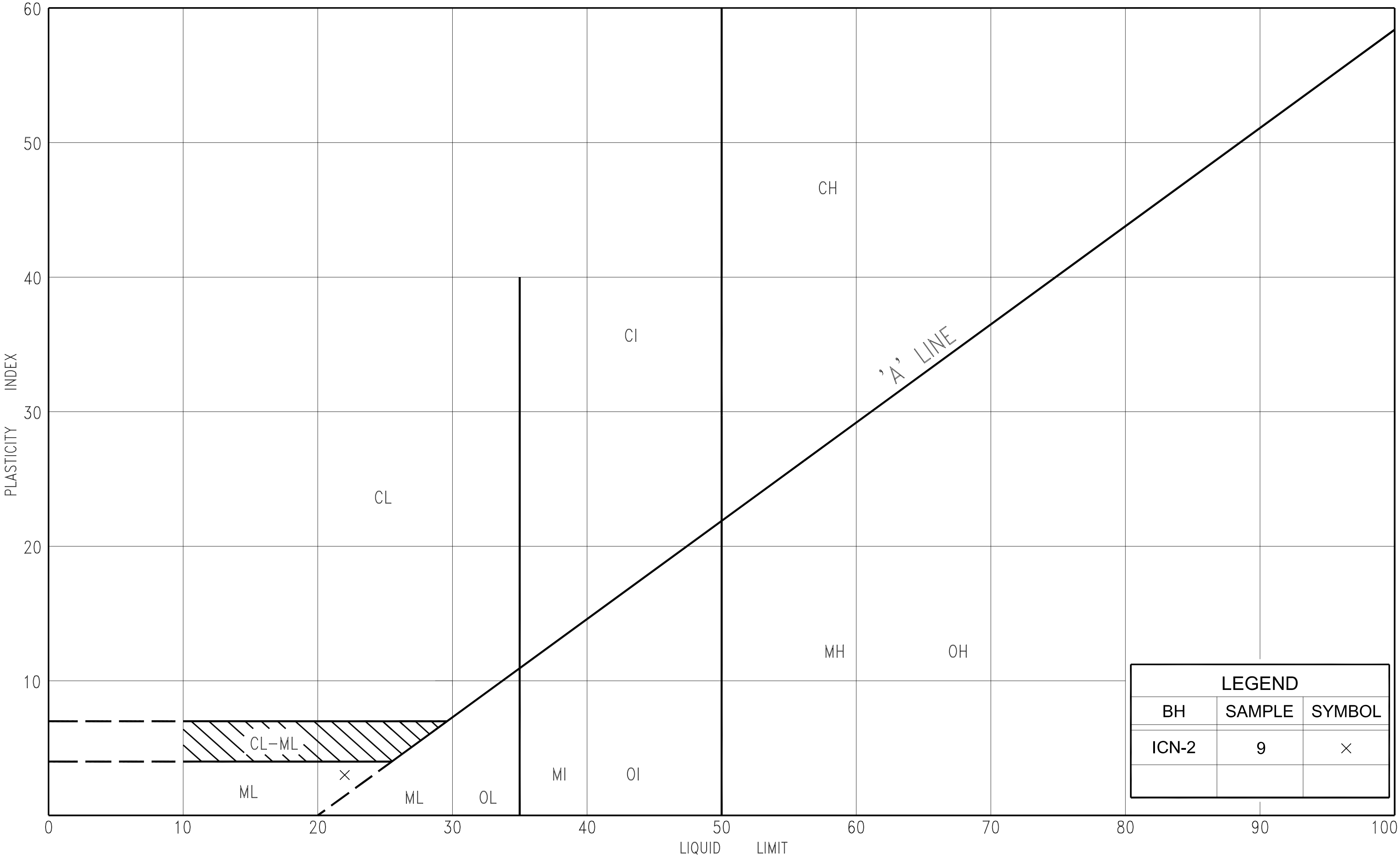
## CLAYEY SILT, trace sand (CL)

FIG No. ICN-PC-2

HWY: 11

G.W.P. No. 323-00-00

Ontario



LEGEND

| BH | SAMPLE | SYMBOL |
|---|---|---|
| ICN-2 | 9 | × |
| | | |

# PLASTICITY CHART

## SILT, some clay, trace sand (ML)

Ontario

| | |
|---|---|
| FIG No. | ICN-PC-3 |
| HWY: | 11 |
| G.W.P. No. | 323-00-00 |

# 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.

**COMPOSITION:** SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

| PERCENT BY MASS | 0 – 10 | 10 – 20 | 20 – 30 | 30 – 40 | > 40 |
|---|---|---|---|---|---|
| | *TRACE* | *SOME* | *WITH* | *ADJECTIVE (SILTY)* | *AND (AND SILT)* |

**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:

| 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 |
| $r_u$ | 1 | PORE PRESSURE RATIO |
| $\sigma$ | kPa | TOTAL NORMAL STRESS |
| $\sigma'$ | kPa | EFFECTIVE NORMAL STRESS |
| $\tau$ | kPa | SHEAR STRESS |
| $\sigma_1, \sigma_2, \sigma_3$ | kPa | PRINCIPAL STRESSES |
| $\epsilon$ | % | LINEAR STRAIN |
| $\epsilon_1, \epsilon_2, \epsilon_3$ | % | PRINCIPAL STRAINS |
| E | kPa | MODULUS OF LINEAR DEFORMATION |
| G | kPa | MODULUS OF SHEAR DEFORMATION |
| $\mu$ | 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 |
| $C_\alpha$ | 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 |
| $\sigma'_{vo}$ | kPa | EFFECTIVE OVERBURDEN PRESSURE |
| $\sigma'_p$ | kPa | PRECONSOLIDATION PRESSURE |
| $\tau_f$ | kPa | SHEAR STRENGTH |
| $c'$ | kPa | EFFECTIVE COHESION INTERCEPT |
| $\phi'$ | -° | EFFECTIVE ANGLE OF INTERNAL FRICTION |
| $c_u$ | kPa | APPARENT COHESION INTERCEPT |
| $\phi_u$ | -° | APPARENT ANGLE OF INTERNAL FRICTION |
| $\tau_R$ | kPa | RESIDUAL SHEAR STRENGTH |
| $\tau_r$ | kPa | REMOULDED SHEAR STRENGTH |
| $S_t$ | 1 | SENSITIVITY $= \frac{c_u}{\tau_r}$ |

### PHYSICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $\rho_s$ | $kg/m^3$ | DENSITY OF SOLID PARTICLES |
| $\gamma_s$ | $kN/m^3$ | UNIT WEIGHT OF SOLID PARTICLES |
| $\rho_w$ | $kg/m^3$ | DENSITY OF WATER |
| $\gamma_w$ | $kN/m^3$ | UNIT WEIGHT OF WATER |
| $\rho$ | $kg/m^3$ | DENSITY OF SOIL |
| $\gamma$ | $kN/m^3$ | UNIT WEIGHT OF SOIL |
| $\rho_d$ | $kg/m^3$ | DENSITY OF DRY SOIL |
| $\gamma_d$ | $kN/m^3$ | UNIT WEIGHT OF DRY SOIL |
| $\rho_{sat}$ | $kg/m^3$ | DENSITY OF SATURATED SOIL |
| $\gamma_{sat}$ | $kN/m^3$ | UNIT WEIGHT OF SATURATED SOIL |
| $\rho'$ | $kg/m^3$ | DENSITY OF SUBMERGED SOIL |
| $\gamma'$ | $kN/m^3$ | UNIT WEIGHT OF SUBMERGED SOIL |
| e | 1,% | VOID RATIO |

| | | |
|---|---|---|
| n | 1,% | POROSITY |
| w | 1,% | WATER CONTENT |
| $S_r$ | % | DEGREE OF SATURATION |
| $w_L$ | % | LIQUID LIMIT |
| $w_P$ | % | PLASTIC LIMIT |
| $w_S$ | % | SHRINKAGE LIMIT |
| $I_P$ | % | PLASTICITY INDEX $= w_L - w_P$ |
| $I_L$ | 1 | LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$ |
| $I_C$ | 1 | CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$ |
| DTPL | | DRIER THAN PLASTIC LIMIT |
| APL | | ABOUT PLASTIC LIMIT |
| WTPL | | WETTER THAN PLASTIC LIMIT |

| | | |
|---|---|---|
| $e_{max}$ | 1,% | VOID RATIO IN LOOSEST STATE |
| $e_{min}$ | 1,% | VOID RATIO IN DENSEST STATE |
| $I_D$ | 1 | DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$ |
| D | mm | GRAIN DIAMETER |
| $D_n$ | mm | n PERCENT - DIAMETER |
| $C_u$ | 1 | UNIFORMITY COEFFICIENT |
| h | m | HYDRAULIC HEAD OR POTENTIAL |
| q | $m^3/s$ | RATE OF DISCHARGE |
| v | m/s | DISCHARGE VELOCITY |
| i | 1 | HYDRAULIC GRADIENT |
| k | m/s | HYDRAULIC CONDUCTIVITY |
| j | $kN/m^3$ | SEEPAGE FORCE |

# RECORD OF BOREHOLE No ICN-1 1 of 1 METRIC

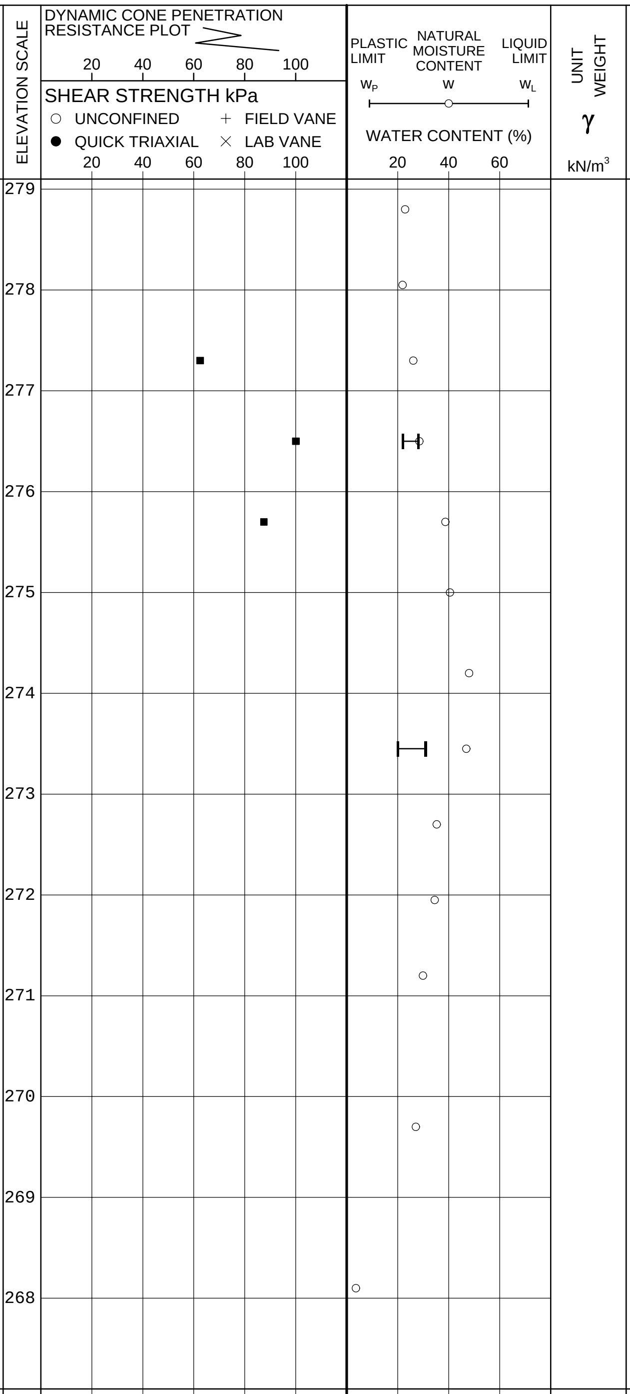
| | | | |
|---|---|---|---|
| **G.W.P.** 323-00-00 | **LOCATION** Co-ords: 5 101 932.4 N ; 316 558.4 E | | **ORIGINATED BY** F.P. |
| **DIST** North Bay **HWY** 11 | **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers | | **COMPILED BY** N.S.B. |
| **DATUM** Geodetic | **DATE** November 28, 2011 | | **CHECKED BY** C.N. |

**SOIL PROFILE**

| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 279.1 | Ground Surface |
| 0.0 | Topsoil |
| 278.6<br>0.5 | Sandy silt<br>Compact Grey Wet<br>Brown |
| 277.7<br>1.4 | Clayey silt, trace sand silt layers<br>Firm to stiff Mottled grey/brown Moist to wet<br>clay layers |
| 272.2<br>6.9 | Silt, some clay clayey silt layers sandy silt partings<br>Loose to compact Grey Wet |
| 269.5<br>9.6 | Sand trace silt, trace gravel<br>Dense Reddish brown Moist<br>(TILL)<br>cobbles and boulders |
| 267.1<br>12.0 | End of borehole<br>Refusal on probable boulder |

**SAMPLES**

| NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|
| 1 | SS | 3 | ▽* | | | | |
| 2 | SS | 11 | | | | | |
| 3 | SS | 5 | | | | | |
| 4 | SS | 9 | | 0 | 2 | 85 | 13 |
| 5 | SS | 11 | | | | | |
| 6 | SS | 3 | | | | | |
| 7 | SS | 4 | | | | | |
| 8 | SS | 5 | | 0 | 1 | 64 | 35 |
| 9 | SS | 5 | | | | | |
| 10 | SS | 9 | | | | | |
| 11 | SS | 8 | | | | | |
| 12 | SS | 17 | | | | | |
| 13 | SS | 39 | | 9 | 85 | (6) | |



* 2011 11 28

▽ Water level observed during drilling

■ Penetrometer test

NOTE: Auger grinding from 11.3m depth

5 Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

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# RECORD OF BOREHOLE No ICN-2

1 of 1

**METRIC**

|  |  |  |  |  |  |
|---|---|---|---|---|---|
| **G.W.P.** | 323-00-00 | **LOCATION** | Co-ords: 5 101 951.9 N ; 316 552.9 E | **ORIGINATED BY** | F.P. |
| **DIST** | North Bay **HWY** 11 | **BOREHOLE TYPE** | C.F.H.S.A. and 'N' Casing | **COMPILED BY** | N.S.B. |
| **DATUM** | Geodetic | **DATE** | November 24, 2011 | **CHECKED BY** | C.N. |

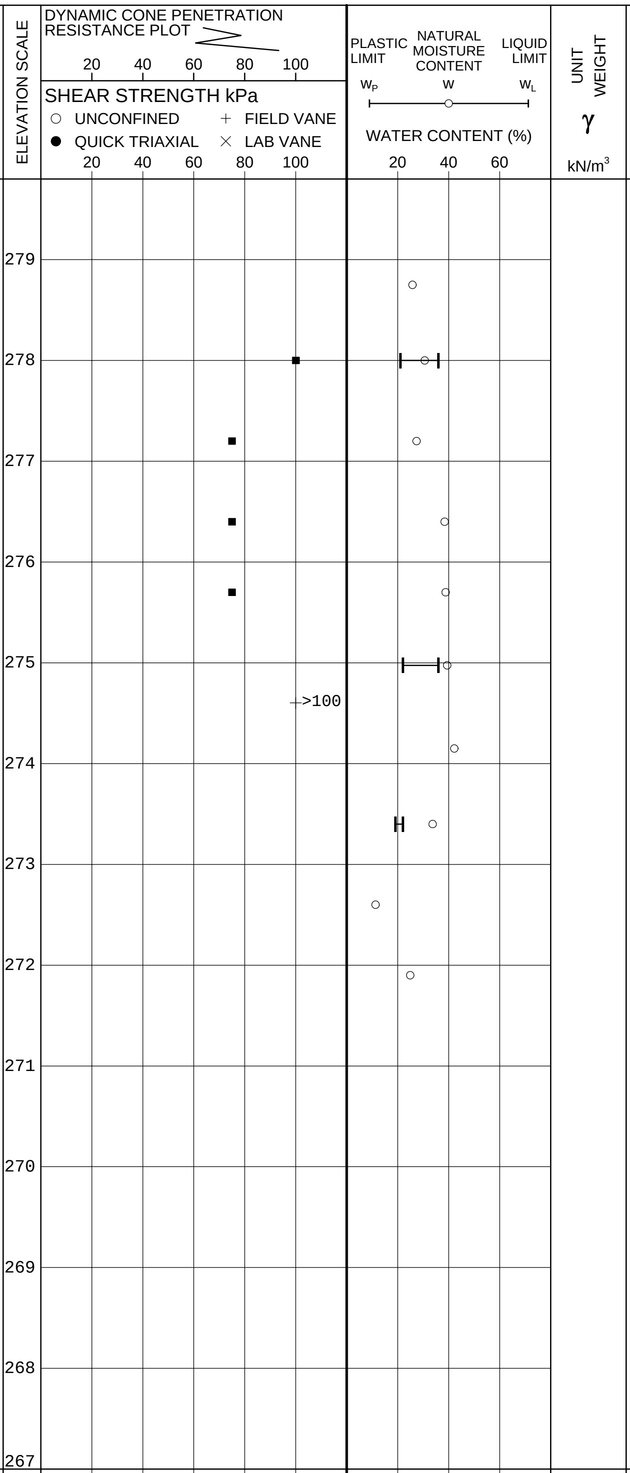
## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 279.8 | Ground Surface |
| 0.0 | Silty sand<br>Very loose Dark brown Moist |
| 279.2<br>0.6 | sand, trace silt<br>organic and glass inclusions<br>(FILL)<br>Sand, trace silt<br>Compact Brown Wet |
| 278.4<br>1.4 | Silty clay, trace sand<br>silt layers<br>Firm to stiff Brown Moist to wet<br>Grey |
| 273.8<br>6.0 | Silt, some clay<br>silty clay/clayey silt seams<br>oxidized partings<br>Compact Grey Wet |
| 272.8<br>7.0 | Silty sand, some gravel<br>cobbles and boulders to 7.6m<br>Dense Brown Moist<br>trace clay<br>Wet<br>(TILL)<br>gravelly sand layers<br>cobbles |
| 267.0<br>12.8 | End of borehole<br>Refusal on probable bedrock |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|
| 1 | SS | 3 | ∇* |  |
| 2 | SS | 10 |  |  |
| 3 | SS | 5 |  | 0 5 58 37 |
| 4 | SS | 7 |  |  |
| 5 | SS | 7 |  |  |
| 6 | SS | 6 |  |  |
| 7 | SS | 6 |  | 0 1 60 39 |
|  | FV |  |  |  |
| 8 | SS | 3 |  |  |
| 9 | SS | 10 |  | 0 5 79 16 |
| 10 | SS | 37 |  |  |
| 11 | SS | 37 |  | 20 42 34 4 |
| 12 | SS | 48 |  |  |

'N' casing advanced from 7.6m depth



* 2011 11 24

∇ Water level observed during drilling

■ Penetrometer test

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

5

Numbers refer to Sensitivity

$$15 -\!\overset{20}{\underset{10}{\bigcirc}}\!- 5$$

(%) STRAIN AT FAILURE

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# RECORD OF BOREHOLE No ICN-3

1 of 2

**METRIC**

| | | | | | |
|---|---|---|---|---|---|
| **G.W.P.** | 323-00-00 | **LOCATION** | Co-ords: 5 101 963.4 N ; 316 582.9 E | **ORIGINATED BY** | F.P. |
| **DIST** | North Bay **HWY** 11 | **BOREHOLE TYPE** | C.F.H.S.A. and NQ Diamond Coring | **COMPILED BY** | N.S.B. |
| **DATUM** | Geodetic | **DATE** | November 21, 2011 | **CHECKED BY** | C.N. |

SOIL PROFILE

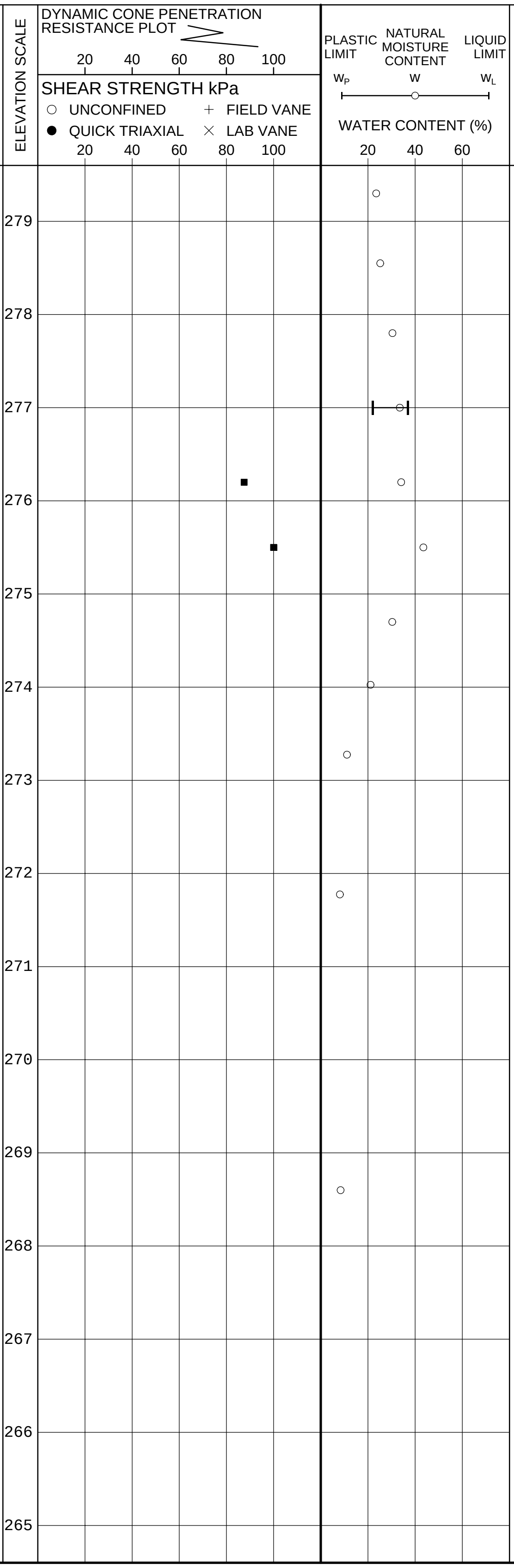
| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 279.6 | Ground Surface |
| 0.0 | Sand, trace silt, organics<br>Compact Brown Moist<br>topsoil inclusions<br>(FILL) |
| 278.2<br>1.4 | Sandy silt<br>Loose Grey Wet |
| 277.9<br>1.7 | Silty clay, trace sand silt layers<br>Soft to stiff Grey Moist to wet |
| 275.0<br>4.6 | Clayey silt, trace sand silt layers<br>Firm Grey Wet |
| 274.0<br>5.6 | Sand, some silt some gravel, trace clay cobbles and boulders to 6.1m<br>Dense to compact Brown Moist<br>(TILL) |
| 267.9<br>11.7 | Granite bedrock<br>Slightly weathered to unweathered, with highly weathered zone<br>High strength<br>Good to excellent quality |
| 264.6 | Cont'd |

SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1 | SS | 14 | |
| 2 | SS | 11 | |
| 3 | SS | 6 | |
| 4 | SS | 3 | 0 1 54 45 |
| 5 | SS | 10 | |
| 6 | SS | 6 | |
| 7 | SS | 7 | 0 3 71 26 |
| 8 | SS | 49 | |
| 9 | SS | 48 | |
| 10 | SS | 48 | 13 71 14 2 |
| 11 | SS | 21 | |
| 12 | SS | 42 | |
| 13 | RC NQ | REC 100% | RQD 75% |
| 14 | RC NQ | REC 100% | RQD 82% |
| 15 | RC NQ | REC 100% | RQD 79% |

GROUND WATER CONDITIONS *

UNIT WEIGHT γ kN/m³



5 Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

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Ontario

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CONSULTING ENGINEERS
Foundation Design

# RECORD OF BOREHOLE No ICN-3    2 of 2    METRIC

**G.W.P.** 323-00-00    **LOCATION** Co-ords: 5 101 963.4 N ; 316 582.9 E    **ORIGINATED BY** F.P.
**DIST** North Bay **HWY** 11    **BOREHOLE TYPE** C.F.H.S.A. and NQ Diamond Coring    **COMPILED BY** N.S.B.
**DATUM** Geodetic    **DATE** November 21, 2011    **CHECKED BY** C.N.

| SOIL PROFILE | | SAMPLES | | | | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × LAB VANE<br>20 40 60 80 100 | PLASTIC LIMIT $w_P$ NATURAL MOISTURE CONTENT w LIQUID LIMIT $w_L$<br>WATER CONTENT (%)<br>20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS * | ELEVATION SCALE | | | | |
| 264.6<br>15.0 | End of borehole<br>* Borehole charged with drilling water<br>■ Penetrometer test<br>NOTE: Boulder coring was carried-out from 5.6 to 6.1m<br>C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers | | | | | | | | | | |

5 Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

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# RECORD OF BOREHOLE No ICN-4

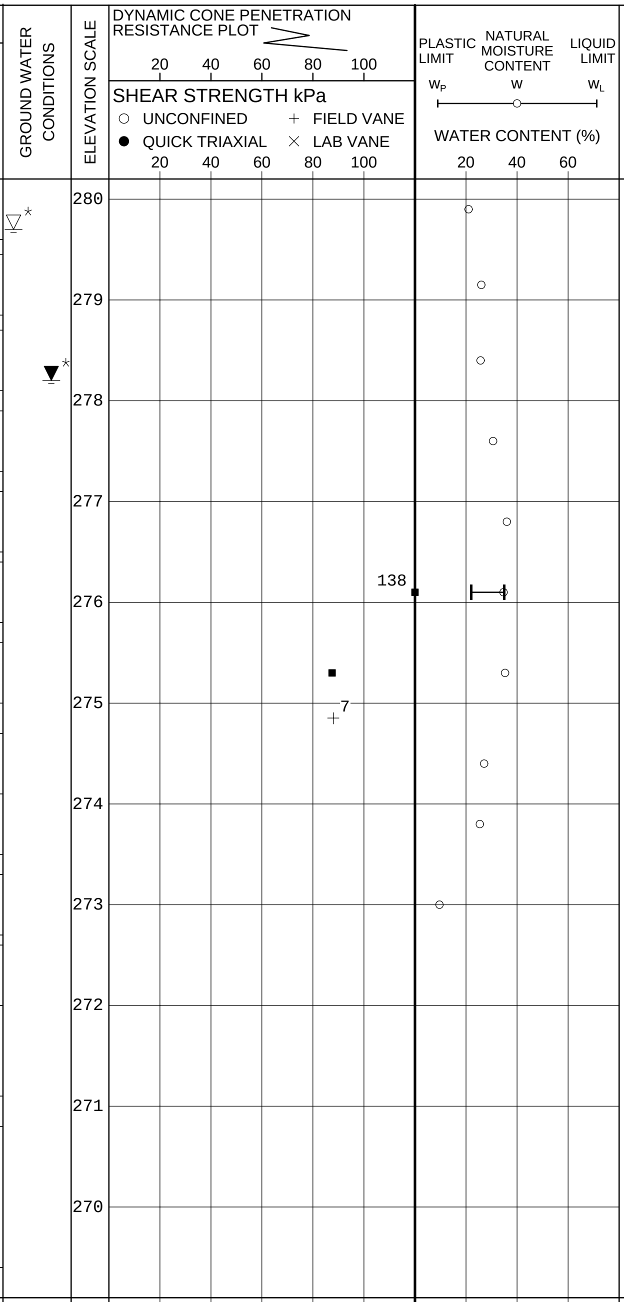
1 of 1

**METRIC**

**G.W.P.** 323-00-00
**LOCATION** Co-ords: 5 101 991.9 N ; 316 567.2 E
**ORIGINATED BY** F.P.
**DIST** North Bay **HWY** 11
**BOREHOLE TYPE** C.F.H.S.A. and 'N' Casing
**COMPILED BY** N.S.B.
**DATUM** Geodetic
**DATE** November 22, 2011
**CHECKED BY** C.N.

| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|
| 280.2 | Ground Surface | | | | | | | |
| 0.0 | Sand and gravel with organics silty sand seams | 1 | SS | 4 | | | | |
| | Loose Brown Wet<br>sand with silt, trace clay<br>Compact | 2 | SS | 11 | 0 | 74 | 23 | 3 |
| | (FILL) | 3 | SS | 10 | | | | |
| 278.2<br>2.0 | Silty clay, trace sand silt layers<br>Soft to very stiff Grey Wet | 4 | SS | 2 | | | | |
| | | 5 | SS | 8 | | | | |
| | | 6 | SS | 7 | 0 | 1 | 57 | 42 |
| | | 7 | SS | 3 | | | | |
| | | | FV | | | | | |
| 274.7<br>5.5 | Silt some clay, trace sand<br>Loose Grey Wet | 8 | SS | 9 | | | | |
| 274.0<br>6.2 | Sand, some silt some gravel, trace clay<br>Dense to very dense Brown Moist | 9 | SS | 41 | | | | |
| | | 10 | SS | 38 | | | | |
| | sand and gravel layers<br>(TILL) | 11 | SS | 38 | 24 | 49 | 25 | 2 |
| | cobbles and boulders from 8.7m | 12 | SS | 50/13cm | | | | |
| 269.1<br>11.1 | | 13 | SS | 87/30cm | | | | |
| | End of borehole<br>Refusal on probable bedrock<br>Samples 12 & 13: Sampler bouncing | | | | | | | |

REMARKS: 'N' casing advanced from 6.7m depth



UNIT WEIGHT γ kN/m³

* 2011 11 22

▽ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

5 Numbers refer to Sensitivity

20
15 ○ 5 (%) STRAIN AT FAILURE
10

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# RECORD OF BOREHOLE No ICN-5

1 of 1

**METRIC**

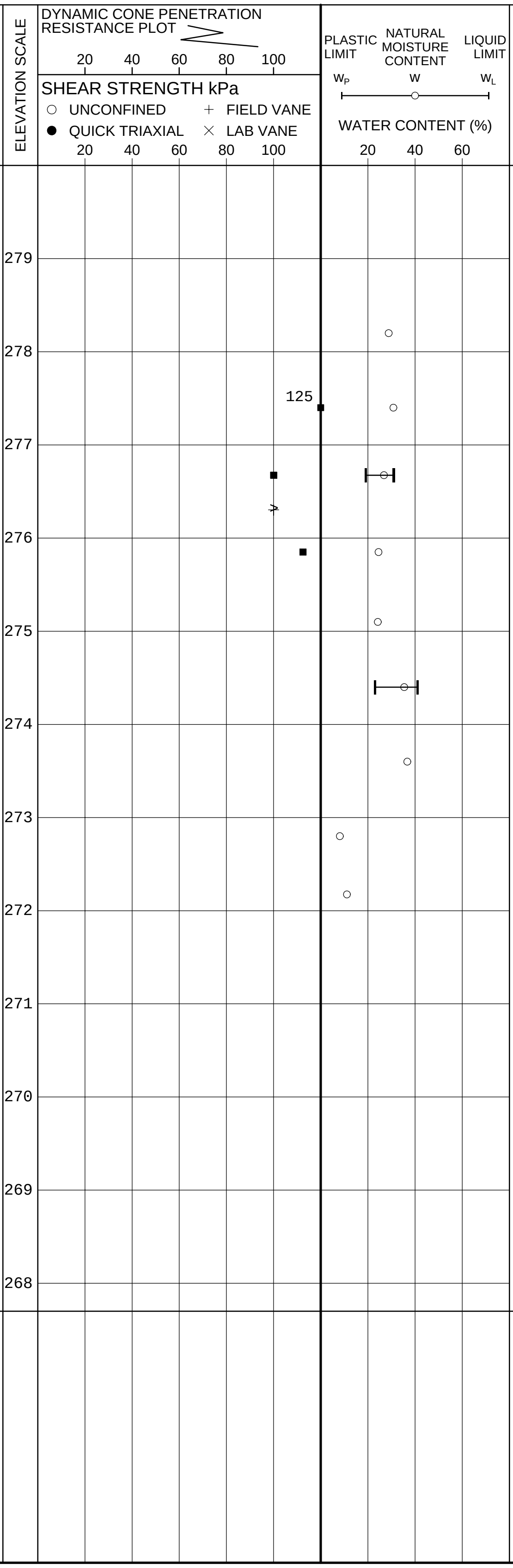
| | | | |
|---|---|---|---|
| **G.W.P.** 323-00-00 | **LOCATION** Co-ords: 5 101 980.9 N ; 316 539.3 E | **ORIGINATED BY** F.P. |
| **DIST** North Bay **HWY** 11 | **BOREHOLE TYPE** C.F.H.S.A. and NQ Diamond Coring | **COMPILED BY** N.S.B. |
| **DATUM** Geodetic | **DATE** November 25 to 28, 2011 | **CHECKED BY** C.N. |

| ELEV<br>DEPTH | SOIL PROFILE<br>DESCRIPTION | SAMPLES<br>NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS * | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|
| 280.0 | Ground Surface | | | | | |
| 0.0 | Peat, fine fibrous<br>Dark brown Moist | 1 | SS | 1 | | |
| 279.1<br>0.9 | Silty sand<br>Very loose Brown Wet | 2 | SS | 1 | | |
| | | 3 | SS | 7 | | |
| 278.2<br>1.8 | Silt, some clay<br>clayey silt seams<br>Loose Grey Wet | | | | | |
| 277.9<br>2.1 | Clayey silt, trace sand<br>silt layers<br>Firm to very stiff Black grey Moist | 4 | SS | 5 | | |
| | | 5 | SS | 9 | | 0 2 67 31 |
| | | | FV | | | |
| | | 6 | SS | 8 | | |
| 275.7<br>4.3 | Silty clay, trace sand<br>Firm to stiff Grey Moist | 7 | SS | 9 | | |
| | sand seams | 8 | SS | 9 | | 0 0 52 48 |
| 274.0<br>6.0 | Silt<br>some clay, trace sand<br>clayey silt seams<br>Compact Grey Moist | 9 | SS | 15 | | |
| 273.3<br>6.7 | Sand, with silt<br>some gravel, trace clay<br>Compact to very dense Brown Moist | 10 | SS | 28 | | 13 60 25 2 |
| | (TILL)<br>cobbles and boulders | 11 | SS | 54 | | |
| | | 12 | SS | 50/10cm | | |
| 270.8<br>9.2 | Granite bedrock<br>Unweathered to slightly weathered<br>High strength<br>Excellent quality | 13 | RC NQ | REC 100% | | RQD 99% |
| | | 14 | RC NQ | REC 100% | | RQD 100% |
| | | 15 | RC | REC 100% | | RQD 100% |
| 267.7<br>12.3 | End of borehole<br>Sample 12: Sampler bouncing | | | | | 'N' casing advanced from 8.1m depth |

* Borehole charged with drilling water

■ Penetrometer test

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers



5 Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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Foundation Design

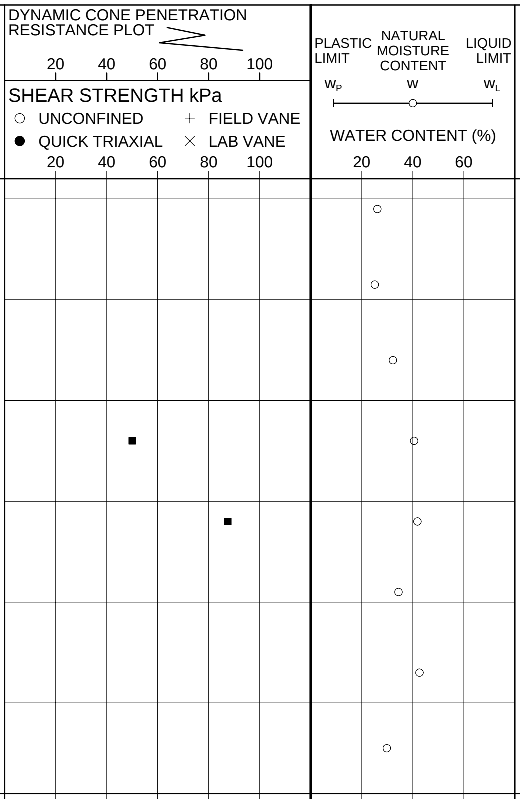
# RECORD OF BOREHOLE No ICN-6 1 of 1 METRIC

**G.W.P.** 323-00-00 **LOCATION** Co-ords: 5 102 010.9 N ; 316 560.9 E **ORIGINATED BY** F.P.

**DIST** North Bay **HWY** 11 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** N.S.B.

**DATUM** Geodetic **DATE** November 23, 2011 **CHECKED BY** C.N.

| SOIL PROFILE | | | SAMPLES | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS * | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | |
| 280.2 | Ground Surface | | | | | | | | | |
| 0.0<br>279.9 | Topsoil | | | | | | | | | |
| 0.3 | Sand, trace silt<br>Loose Brown Wet | | 1 | SS | 5 | | 280 | | | |
| | | | 2 | SS | 8 | | 279 | | | |
| 278.8<br>1.4 | Clayey silt, trace sand silt layers<br>Firm to stiff Brown Moist to wet | | 3 | SS | 4 | | 278 | | | |
| | | | 4 | SS | 4 | | | | | |
| | | | 5 | SS | 6 | | 277 | | | |
| | | | 6 | SS | 3 | | 276 | | | |
| | | | 7 | SS | 4 | | 275 | | | |
| 274.9<br>5.3 | Silt, some clay<br>Loose Brown Wet | | 8 | SS | 9 | | | | | |
| 274.1<br>6.1 | End of borehole<br>Refusal on probable boulder | | | | | | | | | |



* Borehole dry

■ Penetrometer test

5 Numbers refer to Sensitivity 20 15—○—5 10 (%) STRAIN AT FAILURE

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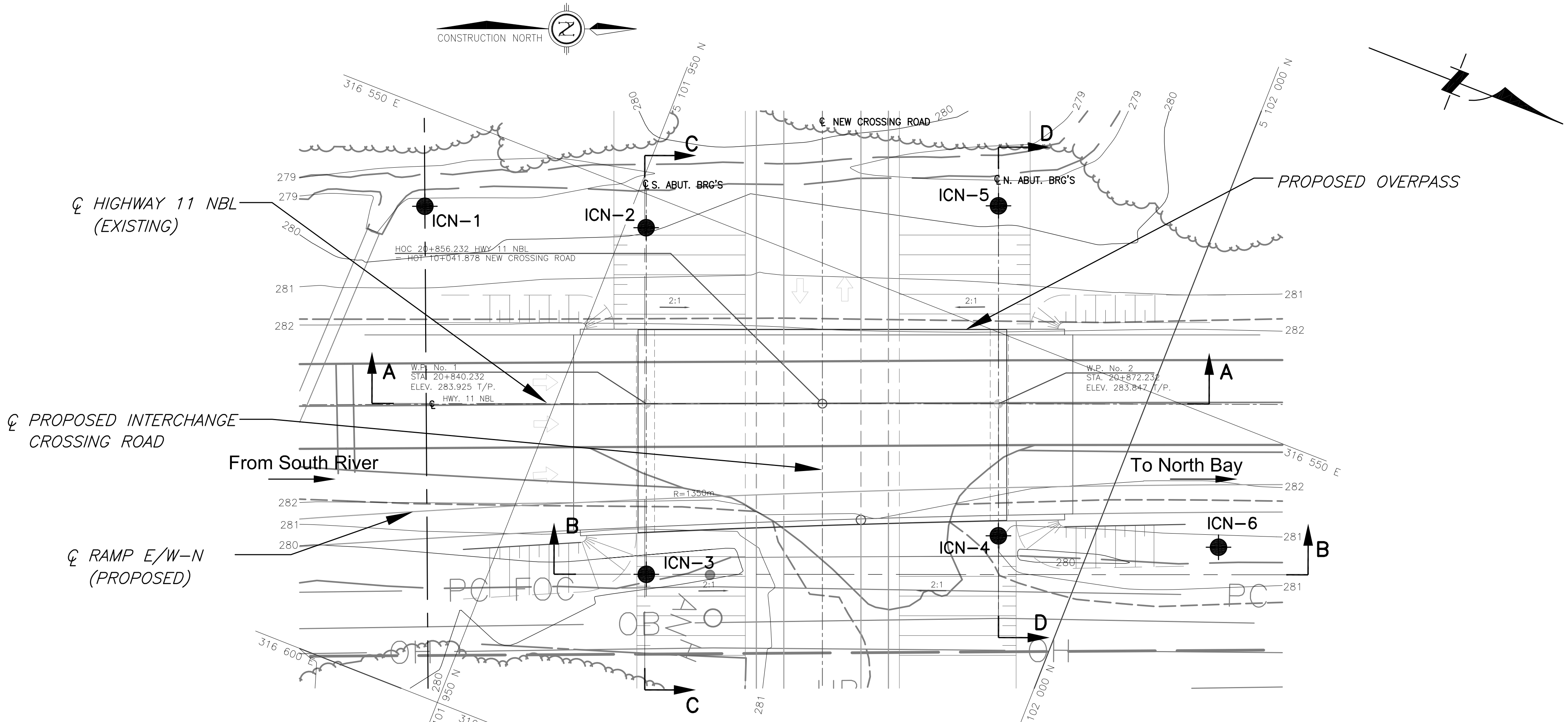
CONT No
GWP No 323-00-00
WP No 5213-10-01

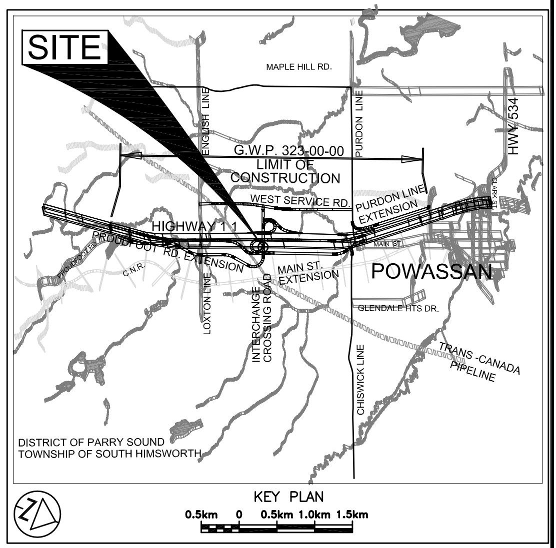
INTERCHANGE CROSSING ROAD OVERPASS
HIGHWAY 11 NORTHBOUND LANES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
1

**Peto MacCallum Ltd.**
CONSULTING ENGINEERS



PLAN
SCALE
5 0 5 10m

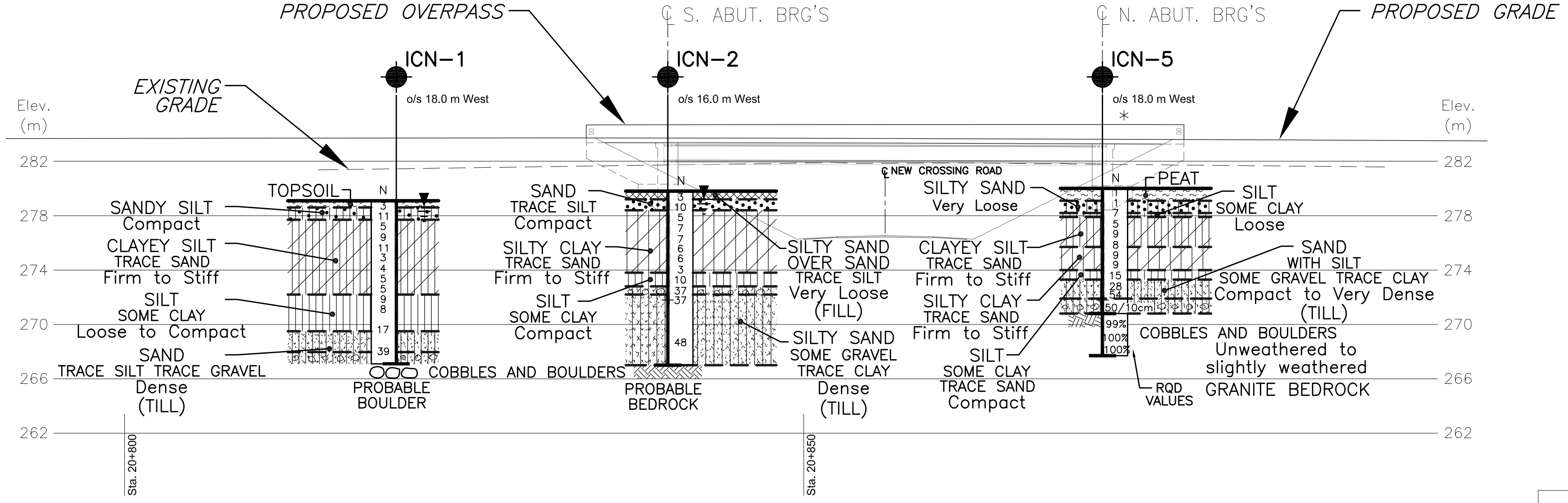


KEY PLAN
0.5km 0 0.5km 1.0km 1.5km

LEGEND

- ● Borehole
- ⊕ Cone
- ⊕ Borehole and Cone
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- ▼ WL at time of investigation Nov. 2011
- * Water level not established
- ▽ Head
- ARTESIAN WATER Encountered
- PIEZOMETER

| BH No | ELEVATION | NORTHINGS | EASTINGS |
|---|---|---|---|
| ICN-1 | 279.1 | 5 101 932.4 | 316 558.4 |
| ICN-2 | 279.8 | 5 101 951.9 | 316 552.9 |
| ICN-3 | 279.6 | 5 101 963.4 | 316 582.9 |
| ICN-4 | 280.2 | 5 101 991.9 | 316 567.2 |
| ICN-5 | 280.0 | 5 101 980.9 | 316 539.3 |
| ICN-6 | 280.2 | 5 102 010.9 | 316 560.9 |



PROFILE A - A ALONG ℄ HIGHWAY 11 (NBL)
SCALE
5 0 5 10m

NOTES:

1. BOREHOLES WERE DRILLED UPTO 10 m AWAY FROM ABUTMENT LOCATIONS. ALLOWANCES SHOULD BE MADE FOR VARIATIONS IN SUBSURFACE STRATIGRAPHY.
2. DRAWINGS IN-1 AND IN-2 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
3. REFER DRAWING IN-2 FOR PROFILE B-B AND SECTIONS C-C AND D-D.
4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
5. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

REGISTERED PROFESSIONAL ENGINEER
C M P Nascimento
Dec 03, 2012
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
Dec. 03, 2012
PROVINCE OF ONTARIO

REF AECOM Drawings: 60157537 Interchange Crossing Road Overpass_NBL_GA_ALT2A.dwg dated April 2011; Hwy 11 Base-Trow.dwg; Hwy 11-Design.dwg; Hwy11_MTO_InRoads_Contours.dwg

– 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-161

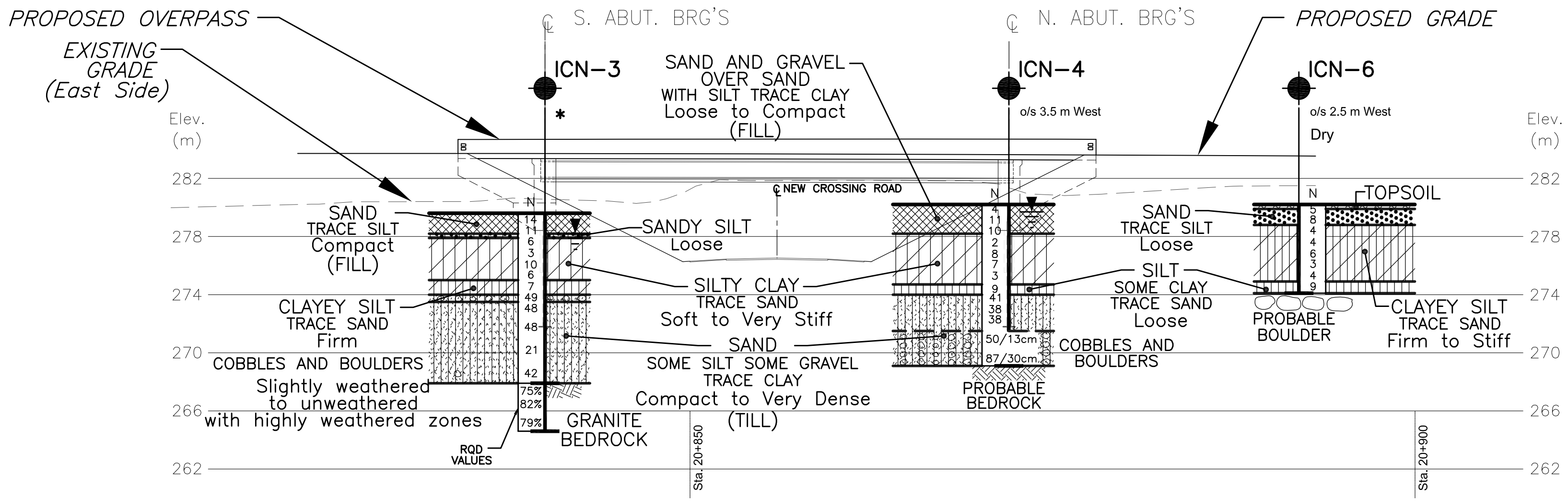
| | | | |
|---|---|---|---|
| HWY No 11 | | | DIST North Bay |
| SUBM'D NA | CHECKED NSB | DATE DEC. 03, 2012 | SITE 44-505/1 |
| DRAWN NA | CHECKED CN | APPROVED BRG | DWG IN-1 |

CONT No
GWP No 323-00-00
WP No 5213-10-01

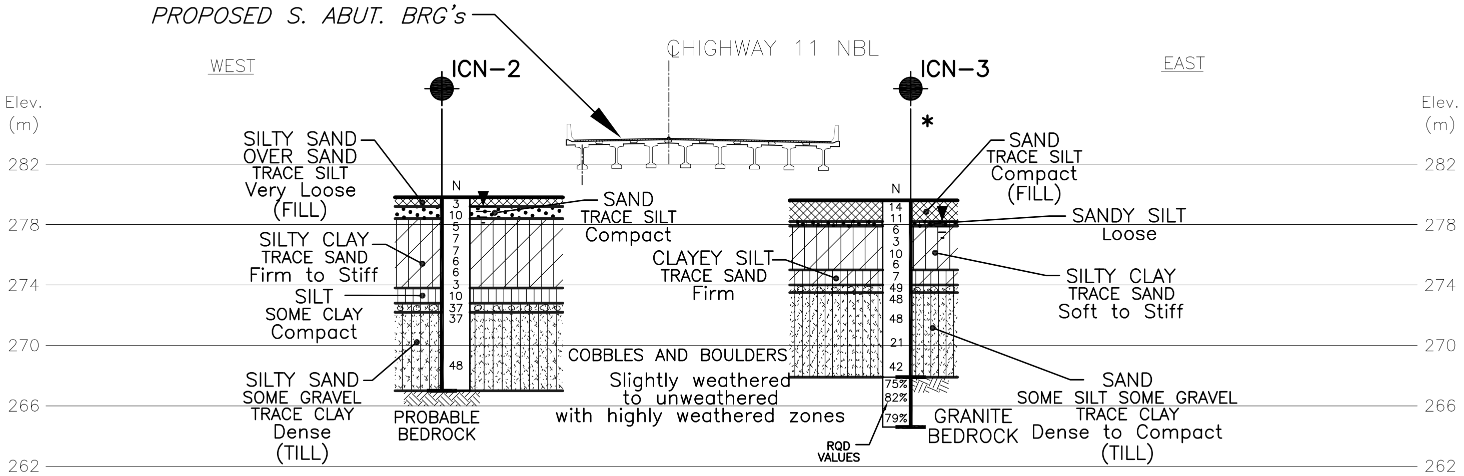
INTERCHANGE CROSSING ROAD OVERPASS
HIGHWAY 11 NORTHBOUND LANES
SOIL STRATA

SHEET
2

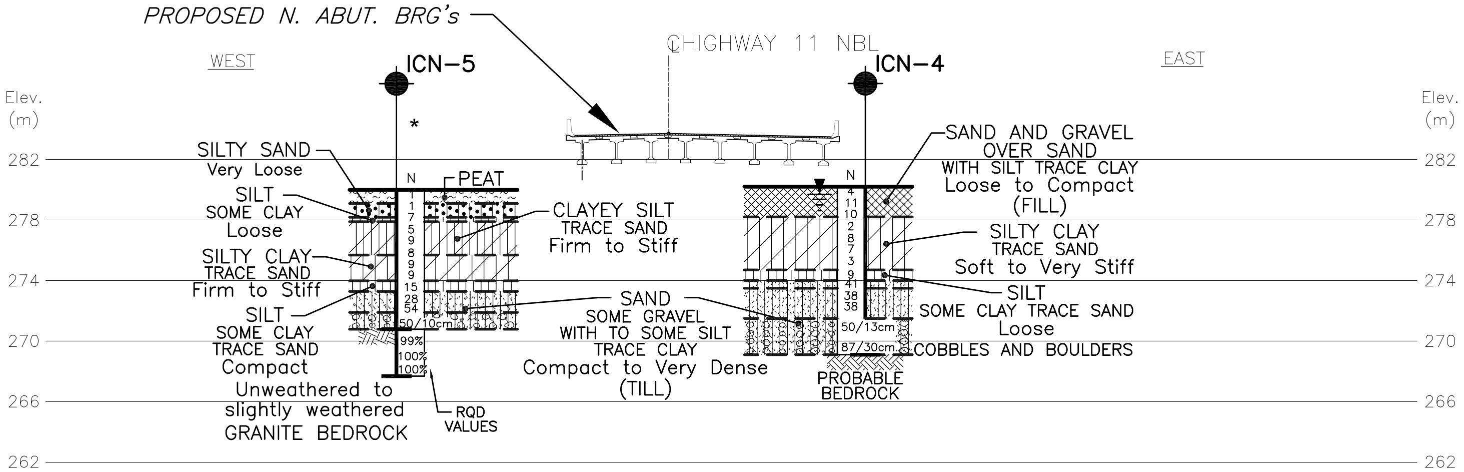
**Peto MacCallum Ltd.**
CONSULTING ENGINEERS



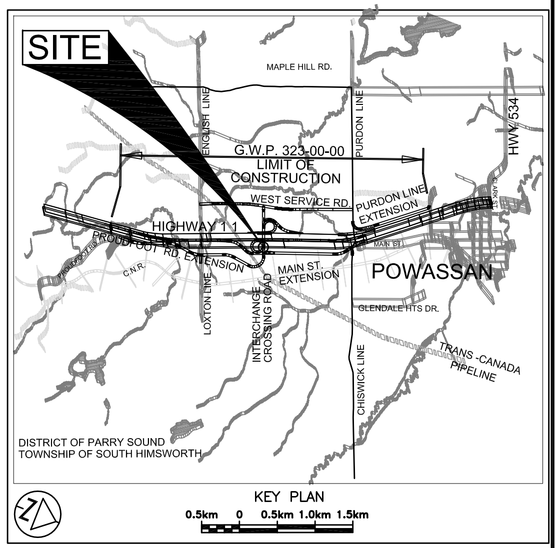
## PROFILE B - B



## SECTION C - C



## SECTION D - D



LEGEND

● Borehole

⊕ Cone

Borehole and Cone

N Blows/0.3m (Std. Pen Test, 475 J/blow)

CONE Blows/0.3m (60 Cone, 475 J/blow)

▽ WL at time of investigation Nov. 2011

* Water level not established

▽ Head
ARTESIAN WATER
Encountered

PIEZOMETER

| BH No | ELEVATION | NORTHINGS | EASTINGS |
|---|---|---|---|
| FOR DETAILS REFER DRAWING IN-1 | | | |

NOTES:

1. BOREHOLES WERE DRILLED UPTO 10 m AWAY FROM ABUTMENT LOCATIONS. ALLOWANCES SHOULD BE MADE FOR VARIATIONS IN SUBSURFACE STRATIGRAPHY.
2. DRAWINGS IN-1 AND IN-2 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
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SCALE
5 0 5 10m

C M P Nascimento
Dec 03, 2012

B. R. GRAY
Dec. 03, 2012

REF AECOM Drawings: 60157537 Interchange Crossing Road Overpass_NBL_GA_ALT2A.dwg dated April 2011; Hwy 11 Base-Trow.dwg; Hwy 11-Design.dwg; X-Hwy11-CONTOURS.dwg

– 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-161

| | | | |
|---|---|---|---|
| HWY No 11 | | | DIST North Bay |
| SUBM'D NA | CHECKED NSB | DATE DEC. 03, 2012 | SITE 44-505/1 |
| DRAWN NA | CHECKED CN | APPROVED BRG | DWG IN-2 |

# APPENDIX A

## Site Photographs

**Photograph 1**: View north from the existing Highway 11 NBL west shoulder. Drill rig at borehole ICN-5. (November 25, 2011)

**Photograph 2**: View north from the existing Highway 11 NBL east shoulder. Drill rig at borehole ICN-3. (November 21, 2011)

# APPENDIX B

Rock Core Photographs

**Photograph 1**: Cores retrieved from borehole ICN-3. Cores 13 to 15 from 11.7 to 15.0 m depth. RQD values ranged from 75 to 82%, indicating good to excellent rock quality.

**Photograph 2**: Cores retrieved from borehole ICN-5. Cores 13 to 15 from 9.2 to 12.3 m depth. RQD values were 99 and 100%, indicating excellent rock quality.

**Peto MacCallum Ltd.**
CONSULTING ENGINEERS

# FOUNDATION DESIGN REPORT
for
INTERCHANGE CROSSING ROAD OVERPASS
HIGHWAY 11 NORTHBOUND LANES
SITE NO. 44-505/1
TOWNSHIP OF SOUTH HIMSWORTH
NORTH BAY AREA, ONTARIO
G.W.P. 323-00-00

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PML Ref.: 10TF013A-S2
Index No.: 316FDR
GEOCRES No.:  31L-161
January 18, 2013

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

# FOUNDATION DESIGN REPORT
for
Interchange Crossing Road Overpass, Highway 11 Northbound Lanes
Site No. 44-505/1
Township of South Himsworth
North Bay Area, Ontario
GWP 323-00-00

## 1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and approach embankments for the proposed construction of an overpass to carry traffic on the Highway 11 Northbound Lanes (NBL) and interchange E/W-N ramp over the proposed unnamed Interchange Crossing Road in Powassan, Ontario. This report was prepared for AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation (MTO).

The proposed overpass over the proposed Interchange Crossing Road will be constructed along the existing Northbound Lanes of Highway 11 approximately between Station 20+840.2 and 20+872.2, Highway 11 NBL chainage, in the Township of South Himsworth. The overpass is proposed to be a single span structure with approximate length of 32 m between abutments and width of 17.5 m (refer to AECOM Drawing No. 1, dated April 2011, Highway 11 NBL Interchange Crossing Road Overpass, General Arrangement–ALT 2A).

The road grade of Highway 11 NBL at the overpass location is planned to be at elevation 283.9 at the south abutment and elevation 283.8 at the north abutment. The approach embankments to the structure at the existing highway centreline are envisaged to be about 1.5 and 1.4 m high above the existing road grade at the south and north abutments, respectively. The proposed E/W-N ramp widening grade at the south and north approach embankments will be about 4.0 and 2.8 m high above the existing grades. The road grade of the proposed Interchange Crossing Road is planned to be on a 6.5 to 6.7 m cut below the existing grade at elevation 275.7.

In summary, the surficial soil cover included 0.6 to 2.0 m thick fill units or 0.3 to 0.9 m thick peat / topsoil. This surficial soils overlay loose to compact cohesionless silt to sand deposit (0.3 to 1.2 m thick) which in turn is underlain by continuous cohesive typically stiff to very stiff silty clay to clayey silt (3.5 to 5.5 m thick) followed by a loose to compact cohesionless silt deposit (0.7 to 2.7 m thick). Below the silt, the stratigraphy included a 2.4 to 6.1 m thick typically dense sand till / silty sand till. At the south abutment location, an upper 0.5 to 0.6 m thick layer of cobbles and boulders was encountered below the silt / clayey silt deposit within the upper zone of till deposit. A lower layer of at least 0.7 to 2.4 m thick cobbles and boulders was also found in all boreholes except borehole ICN-3. Granite bedrock of high strength was contacted/inferred below the native soils at 9.2 to 12.8 m depths (elevation 267.0 to 270.8) in boreholes ICN-2 to ICN-5. The remaining two boreholes ICN-1 and ICN-6 were terminated by refusal on probable boulders at 12.0 and 6.1 m depths (elevation 267.1 and 274.1), respectively.

The groundwater levels were estimated at 2.5 to 3.8 m depths (elevation 276.2 to 277.6) and may vary due to seasonal fluctuations and rainfall patterns.

Based on the encountered subsurface conditions and the height of the embankments over the existing ground surface, conventional spread footings placed on the native clayey soils may be considered feasible.

Because of the high water level, cobbles and boulders present within the till deposits, the use of drilled cast-in-place caissons is not considered suitable for this site.

The overpass may be founded on deep foundations using steel H-piles driven to refusal on bedrock. The steel HP-section piles should be equipped with driving shoes (Titus H Bearing Points Standard Model) due to the presence of potentially sloping granite bedrock and boulders within the till soils.

It should be expected that estimated total settlement due to consolidation of the clayey soils of about 15 to 20 mm at the existing NBL centreline under the north and south approach embankments and 30 to 50 mm on the E/W-N ramp widening portion of approach embankments will occur. Preloading the embankment fill at least 0.5 and 1 month at the north and south approach embankments, respectively prior to installation of piles should be considered to eliminate or reduce the negative skin friction loads on the abutment piles and to satisfy the MTO embankment surface settlement criteria on freeways.

The "red flag" issues outlined in the preceding paragraph and the recommended methods of overcoming these issues noted in the following sections of the report are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.

The elevations referred in this report are expressed in meters. A list of the Ontario Provincial Standard documents referenced in this report is enclosed in Table 1.

## 2. FOUNDATIONS

### 2.1 General

It is considered that the fill and organic soils are highly compressible, therefore are not suitable to support the overpass foundations.

Spread footings placed on the stiff to very stiff cohesive native clayey soils may be used for semi-integral or conventional abutment design. In view of the proposed Interchange Crossing Road that is planned at about elevation 275.7 (up to 6.5 m cut below existing highway grade level), the overpass foundations may be placed on the dense till deposit for a conventional abutment design (rigid frame).

The assessment of the feasibility of using cast-in-place concrete drilled caissons bearing on the glacial till or on the bedrock to support the overpass should consider the difficulties and potential delays caused by the presence of cobbles and boulders in the till and loose to very loose water bearing cohesionless silt as well as the groundwater control requirements.

Founding the proposed overpass on steel H-piles driven to refusal on the bedrock is considered feasible for the south and north abutments.

Conventional, semi-integral and integral abutments are considered feasible at this site based on the foregoing considerations. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundations engineering perspective, use of integral abutments supported on piles driven to refusal on bedrock or very dense cobbles and boulder layer is the preferred type of abutment foundation.

The seismic site coefficient for the conditions at this site is 1.0 (Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00). The zonal acceleration ratio is 0.05. The bridge site is located in a Seismic Performance Zone 1.

All footings subject to frost action should be provided with 1.9 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.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

## 2.2 Spread Footings on Native Soils

### 2.2.1 Footings on Cohesive Soils

The reference founding levels for conventional or semi-integral abutments founded on spread footings placed on the native silty clay / clayey silt at the south and north abutments at approximate elevation 276.8 are provided on the following table:

| FOUNDATION UNIT | SUBGRADE TYPE | BOREHOLE | REFERENCE LEVELS | |
|---|---|---|---|---|
| | | | ELEVATIONS | DEPTHS* (m) |
| South Abutment | Stiff<br>Silty Clay / Clayey Silt | ICN-2 | 276.8 | 3.0 |
| | | ICN-3 | 276.8 | 2.8 |
| North Abutment | Stiff to Very Stiff<br>Silty Clay | ICN-4 | 276.8 | 3.4 |
| | | ICN-5 | 276.8 | 3.2 |

* Depth from the existing ground surface

The following geotechnical resistances should be used for the design of the spread footings on stiff to very stiff cohesive soils:

| FOUNDATION UNIT | SUBGRADE TYPE | FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa) | GEOTECHNICAL REACTION AT SLS (kPa) |
|---|---|---|---|
| South Abutment | Stiff to Very Stiff<br>Silty Clay / Clayey Silt | 250 | 150 |
| North Abutment | | | |

The geotechnical reaction at SLS normally allows for 25 mm of settlement.

This relatively low value of geotechnical resistance will require relatively large abutment footings and make the spread footing alternative placed at elevation 276.8 on cohesive soils impractical for structural design.

Since the Interchange Crossing Road will require earth cut to elevation 275.7, spreading footings placed on the dense cohesionless soils at elevation 272.8 to 273.6 as outlined in Section 2.2.2 is considered a feasible shallow foundation alternative and may be designed for higher geotechnical reactions.

Construction of the footings should be performed and monitored in accordance with OPSS 902 to verify the competency of the founding subgrade.

### 2.2.2 Footings on Cohesionless Soils

In view of the relatively low value of geotechnical resistance for above foundation alternative placed at elevation 276.8, the proposed overpass may be designed as a rigid frame founded on spread footings placed at or below elevation 273.6, that is the minimum 1.9 m foundation frost depth below the proposed Interchange Crossing Road that is to be cut at elevation 275.7.

For the rigid frame, the reference founding levels for spread footings placed on the native till deposit at the south and north abutments are provided on the following table:

| **FOUNDATION UNIT** | **SUBGRADE TYPE** | **BOREHOLE** | **REFERENCE LEVELS** | |
|---|---|---|---|---|
| | | | **ELEVATIONS** | **DEPTHS* (m)** |
| South Abutment | Dense Sand Till | ICN-2 | 272.8 | 7.0 |
| | | ICN-3 | 273.6 | 6.0 |
| North Abutment | Dense Sand Till | ICN-4 | 273.6 | 6.6 |
| | | ICN-5 | 273.3 | 6.7 |

* Depth from the existing ground surface

The following geotechnical resistances should be used for design of the spread footings:

| FOUNDATION UNIT | SUBGRADE TYPE | FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa) | GEOTECHNICAL REACTION AT SLS (kPa) |
|---|---|---|---|
| South Abutment | Dense Sand Till | 700 | 450 |
| North Abutment | | | |

The stabilized water level was assessed to be at least 1.0 m below the footing subgrade levels. The geotechnical reaction at SLS normally allows for 25 mm of settlement except for bedrock that is considered an unyielding founding medium.

Construction of the footings should be performed and monitored in accordance with OPSS 902 to verify the competency of the founding surfaces.

### 2.2.3 Sliding Resistance

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the founding soils. The following parameters should be used for sliding resistance of cast-in-place concrete spread footings placed on native soils.

| PARAMETER | STIFF TO VERY STIFF SILTY CLAY / CLAYEY SILT | DENSE SAND TILL |
|---|---|---|
| Friction angle, degrees | 0 | 35 |
| Cohesion, kPa | 50 | 0 |
| Unit weight, kN/m$^3$ | 19.0 | 23.0 |

## 2.3 Pile Foundation

Steel H-piles could be used to support the foundation loads at both abutments. The piles should be driven to refusal on bedrock anticipated at depths from the existing ground surface of 11.7 and 12.8 m (elevation 267.0 and 267.9) at the south abutment and 9.2 and 11.7 m (elevation 269.1 and 270.8) at the north abutment.

The piles will be driven through native soils containing compressible clayey soils at the abutment locations. The existing grade at the south and north abutments will be raised about 1.5 and 1.4 m above the existing grade, receptively. Consequently, the development of downdrag load on the piles should be considered if the area is not preloaded and/or surcharged as recommended in Section 4.3 of this report.

Refer to Section 4.3 for a discussion and recommendations on the treatment of approach embankment settlements.

The following factored geotechnical axial resistance at ULS for the following sections of steel piles is considered to be appropriate (refer to notes 5 and 6 in Section 3.3.3 of the Pile Driving Notes in the Structural Manual, June 2011):

| **PILE SECTION** | **FACTORED GEOTECHNICAL AXIAL RESISTANCE AT ULS (kN)** |
|---|---|
| HP 310 x 110 | 2000 |
| HP 360 x 152 | 2800 |

The geotechnical reaction at SLS allows for 25 mm compression of founding medium. Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm deformation of the bedrock would be larger than the factored geotechnical resistance at ULS.

Should the embankments grade raises be constructed after the piles are driven and without the preloading period the following downdrag loads should be added:

| **PILE SECTION** | **UNFACTORED DOWNDRAG LOAD (kN)** |
|---|---|
| HP 310 x 110 | 210 |
| HP 360 x 152 | 250 |

As indicated previously, cobbles and boulders were encountered in the boreholes. A NSSP should be prepared to advise the contractor of the potential presence of boulders at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in OPSS 903 and that heavy pile driving with local pre-augering may be required.

It is anticipated that the working platforms to drive the piles will be cut into the existing embankment. Any additional fill that may be required at these locations should comprise OPSS Granular A material to allow installation of the piles without damage. Alternative granular materials such as Granular B Type II could be employed provided the maximum particle size does not exceed 75 mm.

The piles will be driven through dense to very dense soils containing cobbles and boulders and to refusal on bedrock and should be equipped with driving shoes. OPSS 903 calls for the use of OPSD-3000.100 (Driving Shoe Details for H-piles) or Titus H Bearing Pile Points Standard Model on piles driven to bedrock under these pile driving conditions. In view of the slope angles of the bedrock surface found in the borehole varying from 1.7 and 6.8°, it is anticipated that rock points will not be required at this overpass site.

The piles should be installed and monitored in accordance with the requirements of OPSS 903. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 1.9 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.

### 2.3.1 Integral Abutment Considerations

For the integral abutment design, the H-piles should be also driven to refusal on bedrock anticipated at the depths/elevations and axial resistance that are indicated in the previous section. The minimum 5.0 m long free pile length below the abutment stem which should be incorporated in the design will not be a concern at this site.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

### 2.3.2 Lateral Resistance

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The recommended lateral resistance is as follows:

| | **NATIVE SILTY CLAY/ CLAYEY SILT** | | **GRANULAR BACKFILL OR NATIVE SAND TILL** | |
|---|---|---|---|---|
| Pile Section | HP 310 | HP 360 | HP 310 | HP 360 |
| Factored Lateral Resistance at ULS, kN | 160 | 160 | 120 | 170 |
| Lateral Resistance at SLS, kN | 65 | 80 | 50 | 70 |

If greater resistance is required, batter piles should be installed.

The coefficient of horizontal subgrade reaction, $k_s$, should be computed using the following equation to evaluate the point of contraflexture:

$k_s$ = $n_h$ z/b
where $n_h$ = coefficient related to soil density, kN/m$^3$
= 10,000 for granular backfill
z = depth, m
b = pile width, m

The coefficient of horizontal subgrade reaction, $k_s$, for the native silty clay clay/clayey silt units should be taken as 28,000 kN/m$^3$.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters/widths. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

| **PILE SPACING IN DIRECTION OF LOADING d = PILE DIAMETER OR WIDTH** | **SUBGRADE REACTION REDUCTION FACTOR, R** |
|---|---|
| 8d | 1.00 |
| 6d | 0.70 |
| 4d | 0.40 |
| 3d | 0.25 |

## 2.4 Comparison of Foundation Alternatives

Caisson foundations were not considered to be practical in view of installation difficulties due to high groundwater table with water bearing stratum and presence of layers of cobbles and boulders in the glacial till stratum. Spread footings placed on structural fill were not considered due to the presence of the proposed Interchange Crossing Road which is planned to be cut below the existing grade.

A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives discussed in the preceding paragraph is presented below.

| ADVANTAGES | DISADVANTAGES |
|---|---|
| **Footings on Cohesive Soils** | |
| • Lower cost than deep foundations<br>• Allows use of semi-integral abutments | • Relatively low geotechnical resistances will require wide footings and may render this alternative structurally impractical |
| **Footings on Cohesionless Soils** | |
| • Allows use of shorter rigid frame bridge<br>• Higher geotechnical bearing resistance than footings on cohesive soils<br>• Feasible and practical alternative on soils near the foundation frost depth | • Higher maintenance costs than for integral or semi-integral abutments<br>• Groundwater control (cofferdams and dewatering) required for footing construction |
| **Driven Piles** | |
| • Allows use of integral and semi-integral abutments design and construction<br>• Lower long-term maintenance costs of deck expansion joints with integral abutment design<br>• Negligible settlements of foundations | • More costly than shallow foundation alternatives<br>• Heavy equipment for pile driving is required.<br>• May require pre-augering through layers of boulders |

## 3. ABUTMENT WALLS

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 may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation, assuming a triangular pressure distribution:

$p = K(\gamma h + q) + Cp + Cs$

where $K$ = coefficient of lateral earth pressure (dimensionless)
$\gamma$ = unit weight of free-draining granular material, kN/m$^3$
$h$ = depth below final grade, m
$q$ = surcharge load, kPa, if present
$Cp$ = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
$Cs$ = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where $\varnothing$ = angle of internal friction of retained soil (35º for Granular B Type II)
$\delta$ = angle of friction between the soil and wall (23.5º for Granular B Type II)

The seismic site coefficient and zonal acceleration ratio for the conditions at this site were provided in Section 2.1 of this report.

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

| PARAMETERS | GRANULAR A OR GRANULAR B TYPE II | ROCKFILL |
|---|---|---|
| Angle of Internal Friction, degrees | 35 | 42 |
| Unit Weight, kN/m$^3$ | 22.8 | 18.0 |
| Coefficient of Active Earth Pressure $K_a$ | 0.27 | 0.20 |
| Coefficient of Earth Pressure At-Rest $K_o$ | 0.43 | 0.33 |
| Coefficient of Passive Earth Pressure $K_p$ | 3.69 | 5.04 |

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 earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load (q in the preceding equation).

A weeping tile system (MTO SP 405F03 and OPSD 3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the walls. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system.

Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standards Drawings for granular or rock backfill at abutments (OPSD 3101.150 and 3101.200), as applicable.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to OPSS 501 for additional information in this regard.

## 4. APPROACH EMBANKMENTS

### 4.1 General

The level of the approach embankments will be typically raised a maximum of about 1.5 m (south approach) and a maximum of about 1.4 m (north approach) above the existing road grade at the Highway 11 NBL centreline. The proposed E/W-N ramp widening for south and north approach embankments will be about 4.0 and 2.8 m above the existing grades.

It is anticipated that the new embankments will be constructed with granular material similar to the existing embankment or rockfill, if available. Construction of the embankment fill for the abutments on the existing soils is considered to be feasible.

The embankment widening and grade raise (change) for the E/W-N ramp should be carried out in general accordance with OPSD 203.020. The topsoil and loose fill identified in the boreholes located on the east side (ICN-1, ICN-4 and ICN-5) of the abutment locations and present along the alignment of the approach fills within 20 m of the abutments should be stripped prior to placement of the embankment fill. Backfilling adjacent to the structure abutments should be carried out in conformance to Ontario Provincial Standards Drawings for granular or rock backfill at abutments (OPSD 3101.150 or 3101.200), as applicable. Granular A or Granular B Type II (maximum particle size of 75 mm) should be employed within the embankment fill where piles will be driven, if applicable.

## 4.2 Slope Stability

Slope stability analyses were carried out for the south and north embankments for short-term (total stress analysis) and long-term (effective stress analysis) conditions. Based on the soil data and laboratory tests conducted on selected samples, the table below summarizes the soil parameters applied to the analyses.

| SOIL TYPE | UNIT WEIGHT (kN/m$^3$) | SHORT-TERM ANALYSIS | | LONG-TERM ANALYSIS | |
|---|---|---|---|---|---|
| | | COHESION (kPa) | FRICTION ANGLE (Degrees) | EFFECTIVE COHESION (kPa) | EFFECTIVE FRICTION ANGLE (Degrees) |
| Granular Fill | 23 | 0 | 35 | 0 | 35 |
| Sandy Silt to Silt | 19 | 0 | 30 | 0 | 28 |
| Clayey Soils | 19 | 50 | 0 | 8 | 26 |
| Sand to Silty Sand Till | 23 | 0 | 35 | 0 | 35 |

The stability of the approach embankment sections was analysed using the limit equilibrium methods and the SLOPE/W software developed by Geo-Slope International Ltd. The software analyses numerous potential failure surfaces and establishes a minimum safety factor aided by user input.

The results of the slope stability analyses are provided in Figures A-1 to A-4 attached in Appendix FDR-1 and listed below.

| LOCATION | SHORT-TERM CONDITION FACTOR OF SAFETY | LONG-TERM CONDITION FACTOR OF SAFETY | FIGURE NO. |
|---|---|---|---|
| South Abutment (Side Slope 2H:1V) | 2.05 | - | A-1 |
| | - | 1.71 | A-2 |
| North Abutment (Side Slope 2H:1V) | 2.40 | - | A-3 |
| | - | 1.87 | A-4 |

The factors of safety (FOS) values of 2.05 and 2.40 for the short-term and 1.71 and 1.87 for the long-term conditions at the south and north abutments are considered to be adequate for slope stability considerations.

The embankments should be constructed in accordance with OPSD 200.020, 201.020, 202.010 and OPSS 206. The side slopes of the approach embankments should be inclined no steeper than 2H: 1V for earth fill and 1.25H: 1V for rockfill, if utilized.

It is considered that the approach embankments constructed in accordance with the foregoing recommendations will be stable.

## 4.3 Embankment Settlements

Settlement of the road surface within the approach embankments should be expected as a result of consolidation of the new embankment fill, from the existing embankment fill and from the underlying native cohesive clayey soils and cohesionless silt. Settlements of the road surface fill due to consolidation of the compact to very dense glacial till and the bedrock at both embankments will be negligible.

The estimated magnitude of settlement of new granular material is in the order of 10 mm at the existing highway and about 15 to 20 mm at the E/W-N ramp widening near the abutments. These granular materials will settle during construction.

It is anticipated that the existing fill approach embankments will undergo estimated settlements of 10 mm within the existing highway and about 15 mm at the proposed E/W-N ramp widening due to the grade raise changes and these settlements will be completed within about one month of the fill placement.

Based on consolidation test results carried out for the N-E/W and E/W-S ramps of the interchange on cohesive soils with similar characteristics (Reference PML Report No. 10TF013A-H1), it is inferred that the underlying native cohesive clayey soils were subjected to preconsolidation pressures of 570 to 1,000 kPa. The measured initial void ratios ($e_o$) were 0.82 and 1.3, compression index ($C_c$) were 0.4 and 0.7 and coefficient of consolidation ($C_v$) were 1.7 and 1.9 $m^2$/month. These values were utilized for estimating the settlement of the cohesive soils at the bride approaches.

The consolidation settlement of the underlying clayey soils under the existing highway platform is estimated to be in the order of 20 mm (south approach) and 15 mm (north approach) due to 1.5 and 1.4 m new fill loading, respectively. The consolidation settlement of the underlying clayey soils under the proposed E/W-N ramp widening is estimated to be 50 mm (south approach) and 30 mm (north approach) due to 4.0 and 2.8 m new fill loading, respectively.

It is anticipated that the native cohesionless silt under the approach embankments will undergo estimated settlements of 15 and 5 mm at the centreline of the south and north approach embankments and 35 and 10 mm at the E/W-N ramp widening south and north approaches due to the grade raises. It is estimated that these settlements will be essentially completed within about 2 to 3 months after the fill placement.

The following table summarizes the settlement under the south and north approach embankments at the centreline and E/W-N ramp widening portion.

| **SOIL TYPE (*)** | **ESTIMATED TOTAL SETTLEMENT (mm)** | | | |
|---|---|---|---|---|
| | **SOUTH APPROACH** | | **NORTH APPROACH** | |
| | **CENTRELINE** | **E/W-N RAMP WIDENING** | **CENTRELINE** | **E/W-N RAMP WIDENING** |
| New Granular Fill | 10 | 20 | 10 | 15 |
| Existing Fill | 10 | 15 | 10 | 15 |
| Cohesive Clayey Soils | 20 | 50 | 15 | 30 |
| Cohesionless Silt | 15 | 35 | 5 | 10 |
| **TOTAL** | **55** | **120** | **40** | **70** |

(*) The estimated settlement of the dense silty sand till/sand till deposit due to the grade raise will be negligible.

The total settlement at the existing highway is estimated to be about 55 mm at the south approach and about 40 mm at the north approach. The total settlement at the E/W-N ramp widening portion is estimated to be about 120 mm at the south approach and about 70 mm at the north approach.

Settlements of the cohesionless soils and fill materials will likely be completed during or within 3 months of construction. The estimated settlement of cohesive soils at the south abutment is likely to take up to 4 months to occur to 90% completion, and the settlement at the north abutment is estimated to be 90% complete within 2 months following placement of the new fill. The magnitude of the resulting estimated differential settlements on the transverse direction at end of embankment construction at 90% completion of settlement is summarized as follows:

**ESTIMATED REMAINING DIFFERENTIAL SETTLEMENT ON TRANSVERSE DIRECTION (1)**

| COMPLETION OF SETTLEMENT | SOUTH APPROACH | | | | NORTH APPROACH | | | |
|---|---|---|---|---|---|---|---|---|
| | ELAPSED TIME (month) | CENTRE-LINE | WIDENING | DIFFERENTIAL SETTLEMENT (mm) | ELAPSED TIME (month) | CENTRE-LINE | WIDENING | DIFFERENTIAL SETTLEMENT (mm) |
| 0% (end of embankment construction) | 0 | 20 | 50 | 30 | 0 | 15 | 30 | 15 |
| At 50% | 1 | 10 | 25 | 15 | 0.5 | 8 | 15 | 7 |
| At 90% | 4 | 2 | 5 | 3 | 2 | 2 | 3 | 1 |

Note: (1) Settlement due to long term creep is considered to be negligible for this site.

It is recommended that the north and south approach embankments be preloaded for 0.5 and 1 months to reduce differential settlement to the acceptable limits indicated in the MTO memorandum dated March 2, 2010 titled Embankment Settlement Criteria for Design for Freeway Embankments for freeway embankments.

It is considered that earth fill utilizing local native soils will be susceptible to surface erosion, in view of the silty nature of these soils. Earth fill slopes should be protected against surface erosion by sodding (OPSS 803) and suitable vegetation. Also refer to OPSS 804 for time constraints and type of seed and mulch required. Local areas of concentrated surface water flow should be protected with additional measures, such as rip-rap, rock protection or granular sheeting (OPSS 511).

## 5. INTERCHANGE CROSSING ROAD CUT SLOPES

As indicated previously, the proposed Interchange Crossing Road is planned about 6.0 to 6.5 m below the existing grade at the overpass location. In addition, at the abutment locations the existing grade will be raised about some 2.8 to 4.0 m above existing grade at the E/W-N ramp widening portion. Slope stability analyses were carried out for the proposed Interchange Crossing Road cut slope for long-term condition (effective stress analysis which governs for the cut slope stability).

Based on the long-term analysis parameters summarized in Section 4.2, the slope stability analyses were carried out for cut slope angles of 2H:1V, 2.25H:1V and 2.5H:1V. The results of the slope stability analyses are provided in Figures A-5 to A-7 attached in Appendix FDR-1 and are listed below:

| LOCATION | CUT SLOPE | LONG-TERM CONDITION FACTOR OF SAFETY | FIGURE NO. |
|---|---|---|---|
| Interchange Crossing Road | 2H:1V | 1.36 | A-5 |
| | 2.25H:1V | 1.44 | A-6 |
| | 2.5H:1V | 1.51 | A-7 |

A factor of safety of 1.5 or greater is generally considered geotechnically adequate for a long-term stable slope. Based on analyses results, the Interchange Crossing Road cut slope should be shaped at 2.5H:1V or flatter to obtain the adequate long-term stable slope.

## 6. CONSTRUCTION CONSIDERATIONS

### 6.1 Excavation

All excavation at the structure foundation sites should be carried out in accordance with the Occupational Health and Safety Act (OHSA), local and MTO regulations. For this purpose, the upper typically loose to compact silt and firm to stiff clayey silt/silty clay are classified as Type 3 soil according to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Any cobbles or boulders exposed on the excavation slope faces must be removed.

## 6.2 Groundwater Control

At the abutments, groundwater was observed during drilling and was measured upon completion of drilling. The observed groundwater depths that typically ranged from 0.4 to 0.6 m (elevation 278.6 to 279.2) at the south abutment and 0.5 to 1.9 m (elevation 278.3 to 279.7) at the north abutment represent perched water conditions. The long term groundwater levels were estimated to be at 2.5 to 3.8 m depths (elevation 276.2 to 277.6) and may vary due to seasonal fluctuations and rainfall patterns.

It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the spread footing excavations at elevation 276.8 or pile cap foundation excavations that will extend to elevation 278.5 and 280.0, since these levels will be above the water levels encountered.

For the spread footings on cohesionless soils founded at elevation 272.8 to 273.6, conventional sump pumping techniques will not be sufficient to control the groundwater flow into the foundation excavations. Subject to the groundwater level at the time of construction, it is anticipated that temporary groundwater control and a dewatering system will be required for spread footings construction. The contractor is responsible for the selection, performance and detailed design of the dewatering system.

From the foundations standpoint the requirement for a permit-to-take-water (PTTW) will depend on the water tightness of the contractor’s selected type of dewatering system. The PTTW requirement will also depend on the groundwater levels at the time of construction since these are subject to seasonal fluctuations and precipitations patterns.

Surface water run-off should be diverted away from the excavations to ensure that the foundations are constructed in the dry.

## 7. CLOSURE

This Foundation Design Report was prepared by Mrs. N. S. Balakumaran, P. Eng., and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C. M. P. Nascimento, P. Eng., Project Manager conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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NB/CN/BRG:nb-nk-mi

# TABLE 1

## LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

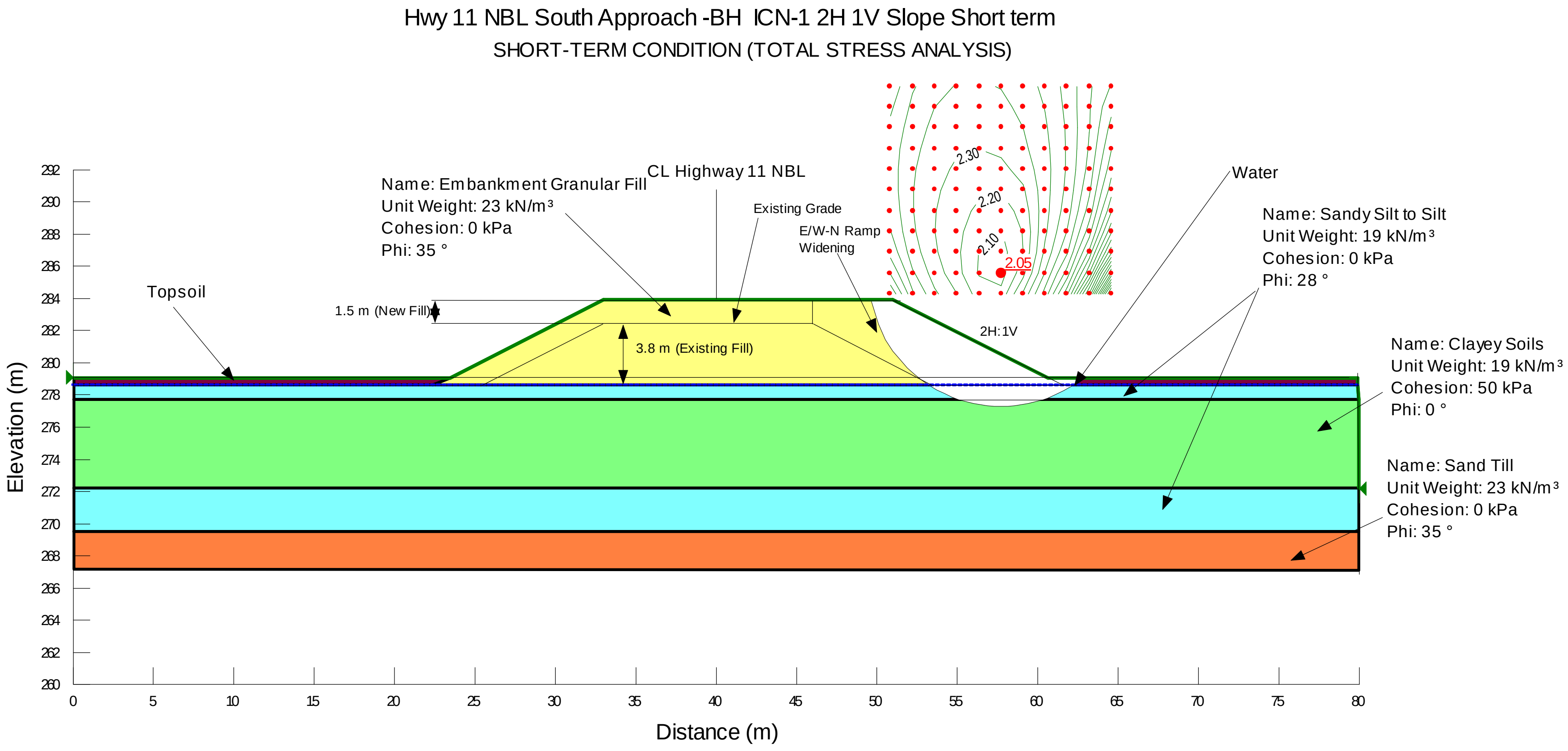
| DOCUMENT | TITLE |
|---|---|
| OPSS 206 | Construction Specification for Grading |
| OPSS 501 | Construction Specification for Compacting |
| OPSS 511 | Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting |
| OPSS 803 | Construction Specification for Sodding |
| OPSS 804 | Construction Specification for Seed and Cover |
| OPSS 902 | Construction Specification for Excavation and Backfilling - Structures |
| OPSS 903 | Construction Specification for Deep Foundations |
| SP 405F03 | Construction Specification for Pipe Subdrains |
| OPSD 200.020 | Earth/Shale Grading-Divided Rural |
| OPSD 201.020 | Rock Grading-Divided Rural |
| OPSD 202.010 | Slope Flattening Using Excess Material on Earth or Rock Embankment |
| OPSD 203.020 | Embankments Over Swamp – Existing Slope Excavated to 1H:1V |
| OPSD 3000.100 | Foundation Piles – Steel H-Pile Driving Shoe |
| OPSD 3101.150 | Minimum Granular Backfill Requirements - Abutments |
| OPSD 3101.200 | Rock Backfill - Walls Abutment |
| OPSD 3190.100 | Retaining Wall and Abutment Wall Drain Detail |

# TABLE 2

## GRADATION SPECIFICATION FOR SAND FILL IN PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

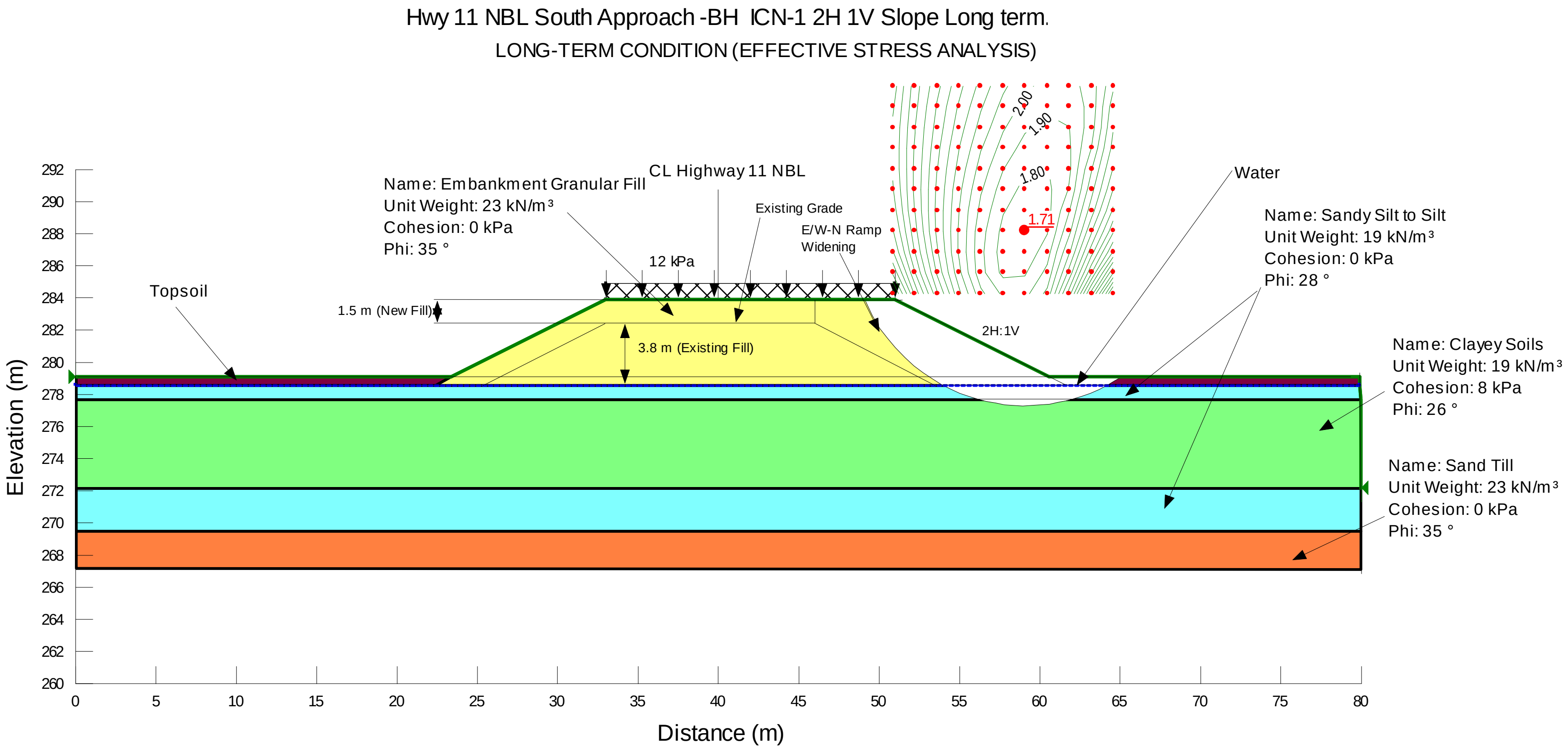
| **MTO SIEVE DESIGNATION** | | **PERCENTAGE PASSING BY MASS** |
|---|---|---|
| 2 mm | #10 | 100 |
| 600 µm | #30 | 80 – 100 |
| 425 µm | #40 | 40 – 80 |
| 250 µm | #60 | 5 – 25 |
| 150 µm | #100 | 0 – 6 |

# APPENDIX FDR-1



Note: Topsoil subexcavated below embankment. The criterion for the minimum FOS of 1.3 is met.

FIGURE A-1



Note: Peat subexcavated below embankment. The criterion for the minimum FOS of 1.5 is met.

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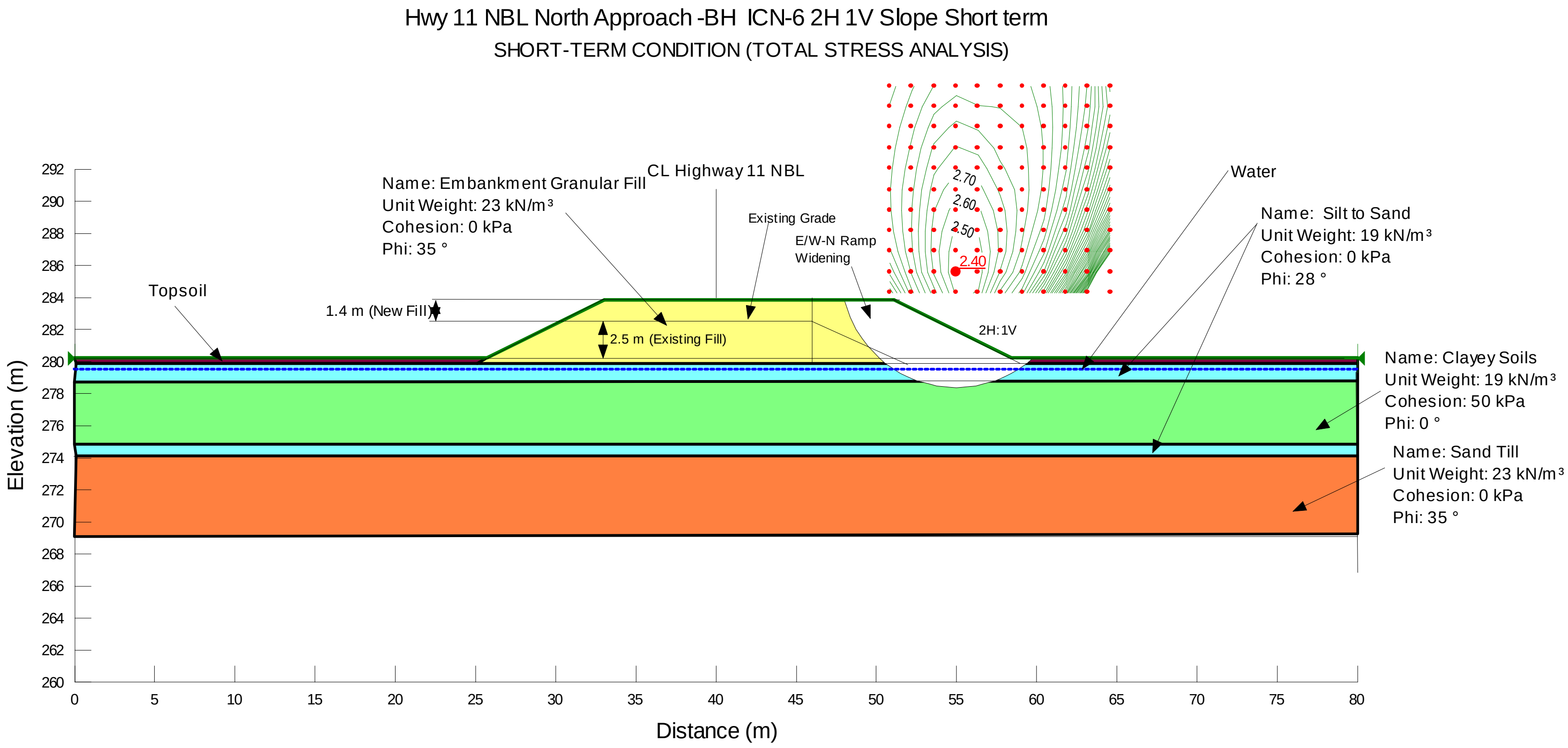
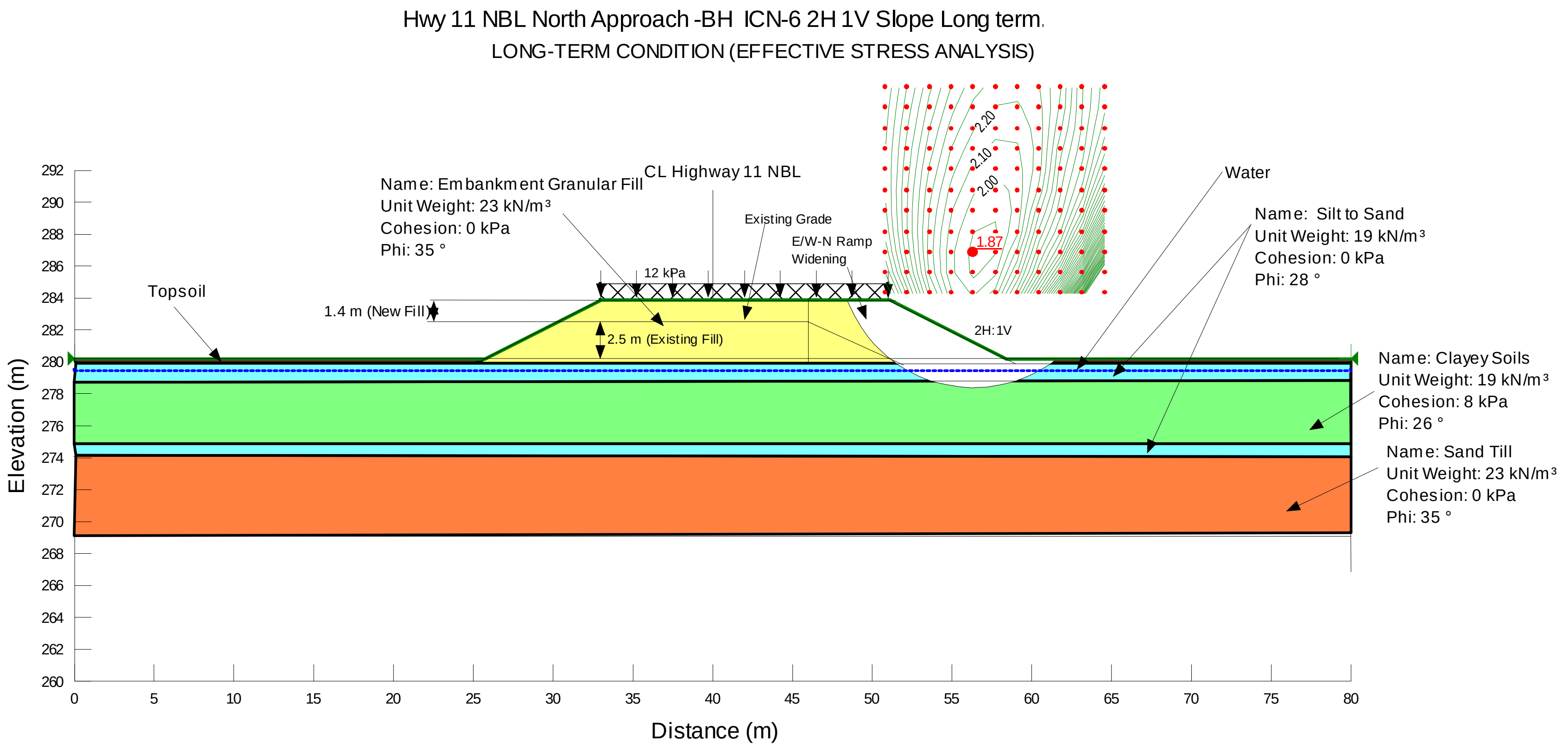
FIGURE A-2



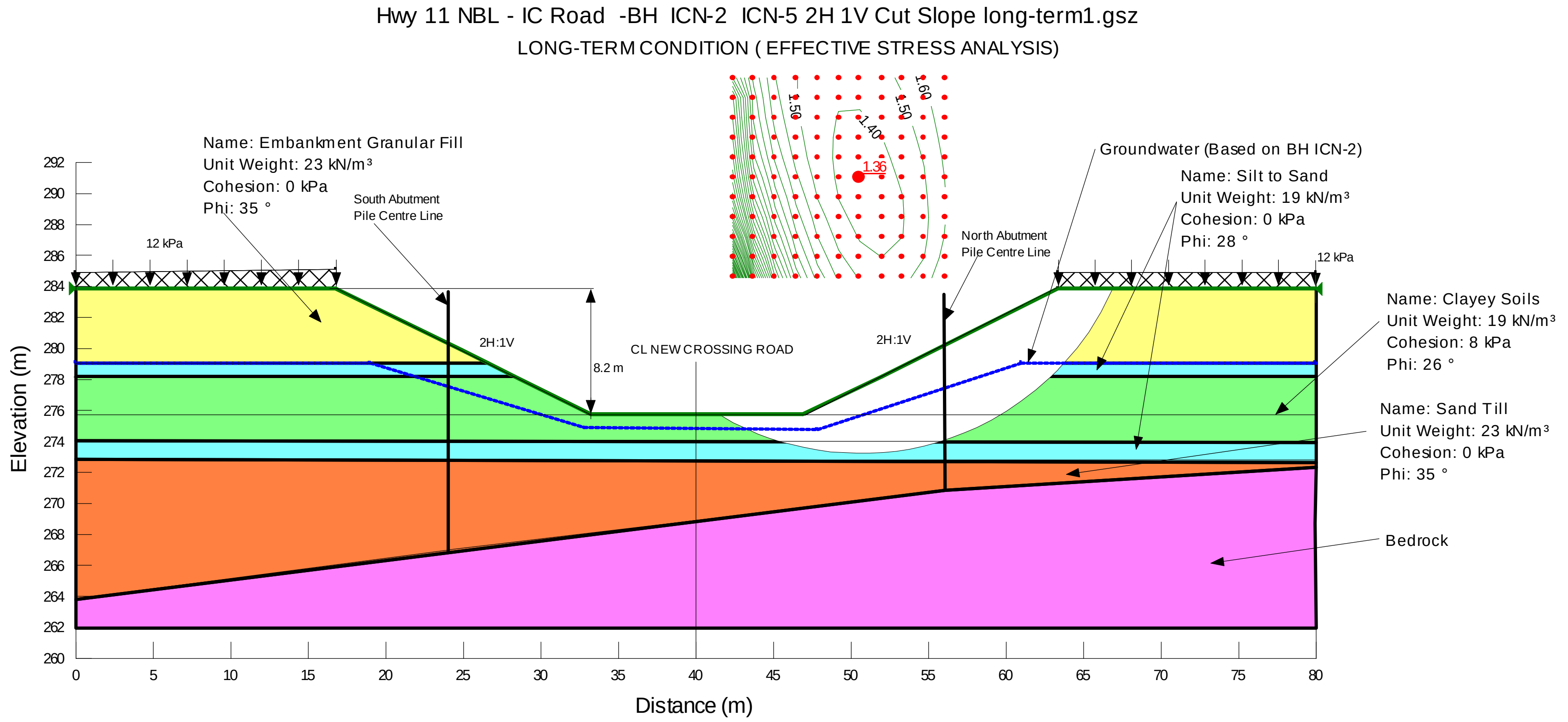
Note: Topsoil subexcavated below embankment. The criterion for the minimum FOS of 1.3 is met.

**FIGURE A-3**



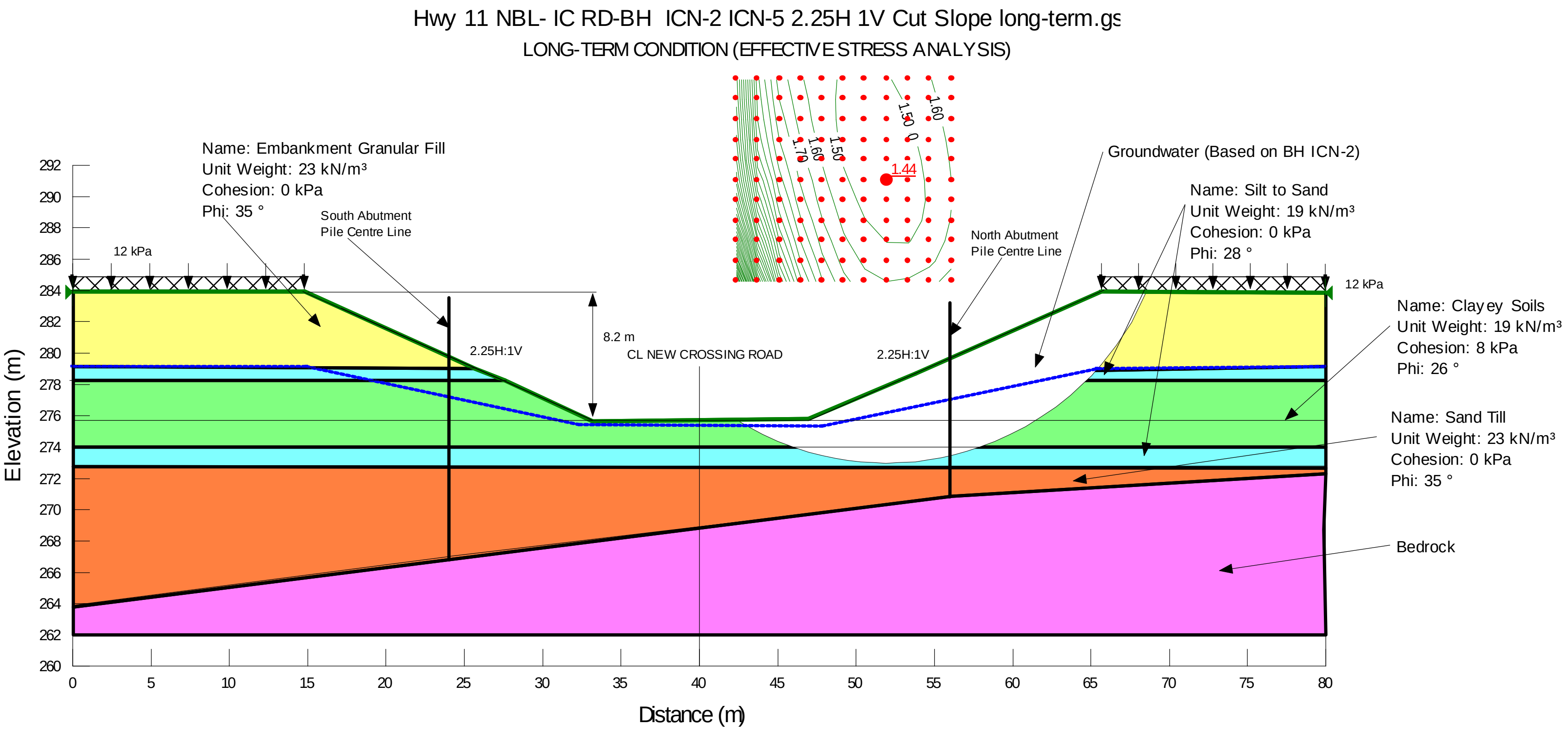
Note: Topsoil subexcavated below embankment. The criterion for the minimum FOS of 1.5 is met.

**FIGURE A-4**



Note: The criterion for the minimum FOS of 1.5 is not met.

**FIGURE A-5**

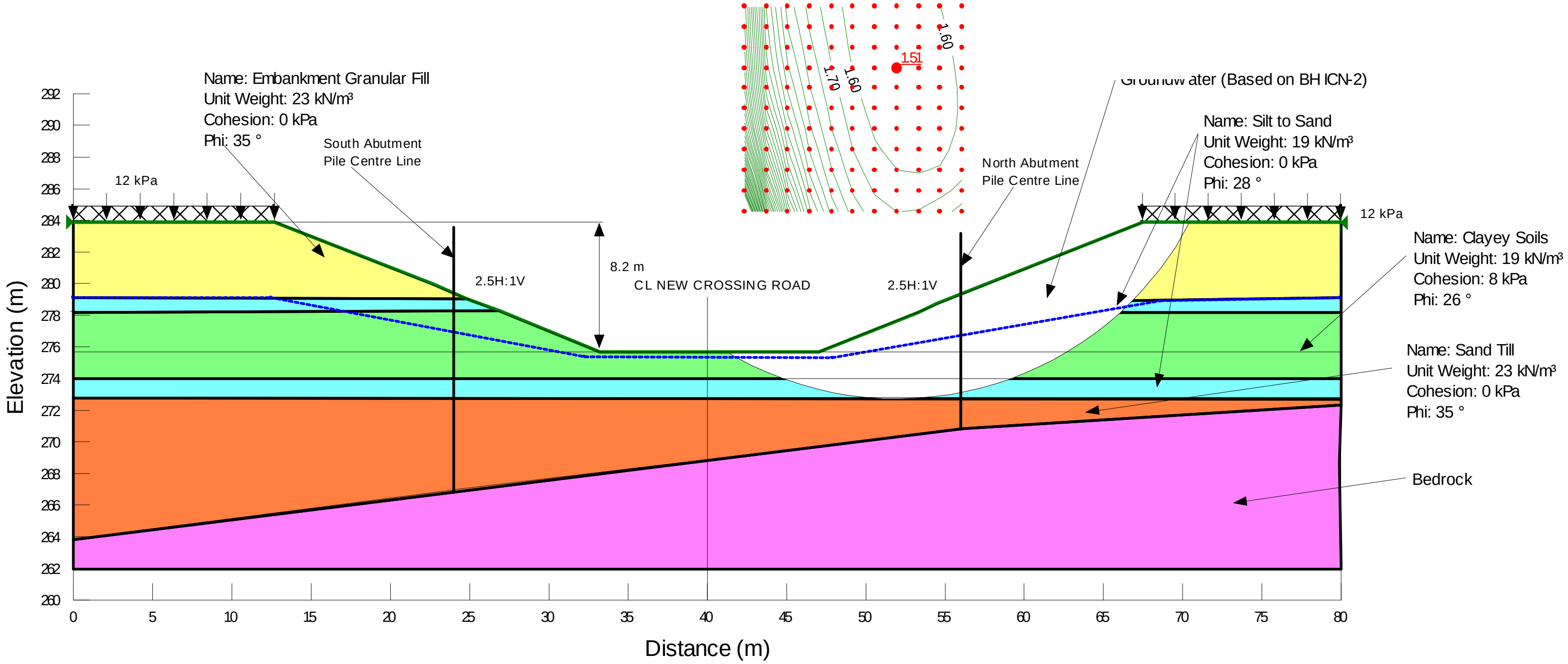


Note: The criterion for the minimum FOS of 1.5 is not met.

FIGURE A-6

Hwy 11 NBL- IC RD-BH ICN-2 ICN-5 2.5H 1V Cut Slope long-term.gsz

LONG-TERM CONDITION (EFFECTIVE STRESS ANALYSIS)



Note: The criterion for the minimum FOS of 1.5 is met.

**FIGURE A-7**