PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

KANTOLA ROAD/FIELDING ROAD UNDERPASS
SITE NO. 46-517
HIGHWAY 17 SUDBURY SOUTHWEST BYPASS FOUR-LANING
CITY OF GREATER SUDBURY
GWP 5825-05-00
TOWNSHIP OF WATERS
DISTRICT 54, SUDBURY, ONTARIO

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
Kantola Road/Fielding Road Underpass
Site No. 46-517
Highway 17 Sudbury Southwest Bypass Four-Laning
City of Greater Sudbury
GWP 5825-05-00
Township of Waters
District 54, Sudbury, Ontario

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1. INTRODUCTION

This report summarizes the results of the preliminary foundation investigation carried out for the future Kantola Road/Fielding Road Underpass at the Highway 17 Sudbury Southwest Bypass in the City of Greater Sudbury, Ontario. Peto MacCallum Ltd. (PML) conducted the preliminary investigation for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed underpass will carry the realigned Kantola Road/Fielding Road over the future four-lane Highway 17 eastbound and westbound lanes at about Sta. 17+070 (existing Highway 17 chainage). Stantec provided the preliminary layout and ground surface profile for the proposed structure.

This report provides preliminary subsurface information pertaining to the proposed underpass foundations and approach embankments within about 20 m of the abutments.

2. SITE DESCRIPTION AND GEOLOGY

The contemplated structure is located about 60 m west of the existing Kantola Road/ Fielding Road and Highway 17 intersection (as shown on Drawing L2-1). The site is about 2,100 m east from the existing interchange of the Highway 17 and Sudbury Municipal Road 55 (Middle Junction).

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Land use in the vicinity of the site includes the existing Kantola Road/Fielding Road and Highway 17 at-grade crossing and residential dwellings on Kantola Road and Moxam Landing Road south of the existing Highway 17 intersection. North of the intersection there are a recreational facility (Fielding Memorial Park) and an old earth borrow/gravel pit area (Inco's property). The alignments of the existing and realigned Kantola Road/Fielding Road are roughly parallel.

The local topography is relatively flat and gently slopes down westerly to the Junction Creek valley. The relief between the Junction Creek and the site is about 10 m. The ground cover beyond the highway near the structure site typically comprises grasses, bushes and stands of trees. Typical swamp vegetation is found to the southeast of the site.

The project is situated in the Hough Lake Group of the Huronian Supergroup. The typical rock type in the project area is Huronian metasedimentary rock which includes interbedded argillite, siltstone, greywacke and subarkose of the Pecors Formation. The soil/bedrock interface is found at variable depths, generally over 10 to 15 m deep.

3. INVESTIGATION PROCEDURES

The field subsurface investigation was carried out during the period of May 9 to June 21, 2007. Three sampled boreholes were put down at the site and were identified with the prefix L2. The boreholes were drilled to depths of 24.9 to 31.4 m at the locations shown on Drawing L2-1. Further details are summarized in the following table.

STRUCTURE	LOCATION	BOREHOLE NO.	DEPTH (m)		
			AUGER	ROCK CORE(1)	TOTAL
Kantola Road / Fielding Road Underpass	North abutment	L2-1	21.8	3.1	24.9
	Pier	L2-2	28.0	3.4	31.4
	South abutment	L2-3	25.9	3.2	29.1

(1) NQ diamond rock coring equipment

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

The boreholes were advanced using continuous flight hollow stem augers, powered by a track mounted CME-55 drill rigs, supplied and operated by specialist drilling contractors, working under the full-time supervision of a Field Supervisor from PML engineering staff.

Representative samples of the soils were recovered in the boreholes at frequent depth intervals of 0.75 and 1.5 m. The soil samples were obtained using a split spoon sampler in conjunction with standard penetration tests. In-situ vane shear and penetrometer testing was also performed to further assess the shear strength of cohesive soils encountered. It is noted that the penetrometer tests provide lower results than the actual values of shear strength due to sample disturbance.

Due to numerous boulders, boreholes L2-1 and L2-2 had to be advanced using NQ diamond drilling and wash boring techniques from 21.0 to 21.8 m and 23.2 to 28.0 m depth, respectively. The boreholes at each of the abutments and piers were extended 3.1 to 3.4 m into the bedrock using NQ diamond rock coring equipment supplemented by NW casing. The PML geologist examined the recovered rock core samples.

Photographs of the rock cores recovered in boreholes are enclosed in Appendix B and detailed descriptions provided in Table 1.

The 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 record of boreholes.

All boreholes were backfilled in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

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 included the following tests:

- Natural moisture content determinations (52)
- Grain Size analyses (12)
- Atterberg Limits (4)

The results of the laboratory natural moisture content determinations, grain size analyses and Atterberg limits are shown on the Record of Borehole sheets. The grain size distribution charts are presented on Figures GS-L2-1 to GS-L2-5. The Atterberg limits results are presented on Figures PC-L2-1 and PC-L2-2.

4. SUMMARIZED SUBSURFACE CONDITIONS

4.1 General

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

The borehole locations as well as the longitudinal soil profile are presented on Drawing L2-1.

The soil stratigraphy in the boreholes includes units of surficial fill or topsoil overlying continuous deposits of clayey silt/clay underlain by cohesionless silts which are in turn underlain by silty sand/sand. A local deposit of numerous cobbles and boulders was encountered in the two boreholes. The sand deposits mantle bedrock.

4.2 Fill and Topsoil

A surficial fill unit 300 mm thick is encountered in borehole L2-3 extending to elevation 255.7. The unit is composed of silty clay mixed with sand and gravel and organic inclusions.

A 200 mm thick surficial deposit of topsoil is present in boreholes L2-1 and L2-2 extending to elevations 256.1 and 256.6, respectively. The water content of one topsoil sample was 27%.

4.3 Clayey Silt/Clay

A deposit of cohesive clayey silt is encountered in boreholes L2-1 and L2-2 at depths of 1.4 and 0.2 m (elevations 254.9 and 256.6) extending to depths of 4.3 and 3.1 m (elevations 252.0 and 253.7), respectively. The deposit is soft to stiff, based on penetrometer test values ranging from 12 to 87 kPa.

Below the fill, a discontinuous deposit of cohesive clay containing sandy silt lenses and thin silt layers occurs in the borehole L2-3 extending to depth of 4.4 m (elevation 251.6). This deposit has a soft to stiff consistency with penetrometer and field vane test values ranging from 12 to 100 kPa.

The grain size distribution charts of a representative sample of the clayey silt are shown on Figure GS-L2-1 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-L2-1. The water content of the clayey silt varied from 24 to 35%, typically decreasing with depth. The liquid limits of the clayey silt ranged from 26 to 30 and the plastic limits from 18 to 23, giving the plasticity index values of 5 to 8.

The grain size distribution chart of a representative sample of the clay is shown on Figure GS-L2-2 and the Atterberg plasticity limit on the Plasticity Chart Figure PC-L2-2. The water content of the clay varied from 28 to 47%. The liquid limit of the clay was 52 and the plastic limit 21, giving a plasticity index of 31.

4.4 Silt

A 1.2 m thick upper deposit of cohesionless silt trace to with sand is present below the topsoil in borehole L2-1(north abutment location) extending to a depth of 1.4 m (elevation 254.9). The upper silt deposit exhibits loose to compact relative density with N values ranging from 4 to 10.

Below cohesive soils, cohesionless silt occurs in all of the boreholes at relatively uniform depths ranging from 3.1 to 4.4 m (elevations 251.6 to 253.7) extending to depths of 10.1 to 11.6 (elevations 244.4 to 246.7). This deposit has loose to compact relative density with N values ranging from 7 to 18. Two low N values of 4 and 6, which were recorded in borehole L2-2, are believed to reflect hydraulic disturbance during drilling.

A lower deposit of silt is encountered in the borehole L2-1 at depth of 13.1 m (elevation 243.2) extending to a depth of 16.2 m (elevation 240.1). This deposit was found to be compact in relative density, based on N values of 22 and 32.

The envelope of grain size distribution charts of representative samples of silt is shown on Figure GS-L2-3. The material was non-plastic according to one Atterberg limit determination and manual examination. The water content of silt soils varied from 19 to 27%.

4.5 Sandy Silt

A deposit of sandy silt was locally found at 13.1 m depth (elevation 243.7) below the silt and a sand layer (described in the following section) in borehole L2-2. In borehole L2-3, the sandy silt deposit was found below the silt between 11.6 and 13.4 m depths (elevations 244.4 and 242.6) where it was interbedded with a silty sand deposit (also described below). The sandy silt encountered in borehole L2-2 extended to 20.4 m (elevation 236.4). In borehole L2-3, the sandy silt layer was also found between 16.5 and 20.4 m (elevations 239.5 and 235.6). This deposit typically exhibited loose to compact relative density. N values typically ranged from 8 to 25.

The grain size distribution chart of a representative sample of the sandy silt is shown on Figure GS-L2-4. The water content of sandy silt soils varied from 18 to 25%.

4.6 Silty Sand/Sand

Two cohesionless silty sand layers were encountered in borehole L2-3 (south abutment). The upper layer extended between depths of 13.4 and 16.5 m (elevations 242.6 and 239.5, respectively) and the lower layer between 20.4 and 22.9 m (elevations 235.6 and 233.1). These deposits were compact to dense in relative density. N values ranged from 16 to 31.

An upper deposit of cohesionless sand was present in boreholes L2-1 and L2-2 at depths ranging from 10.1 to 11.6 m (elevations 244.7 to 246.7) and extended to an uniform depth of 13.1 m (elevations 243.2 to 243.7). This layer was found to be very loose to compact in relative density. N values ranged from 3 to 12. The single low N value of 3 is believed to relate to hydraulic disturbance during drilling.

A lower sand stratum occurred in all boreholes at depths ranging from 16.2 to 22.9 (elevations 233.1 to 240.1) and extending to the underlying bedrock levels which were encountered at depths from 21.8 to 28.0 m (elevations 228.8 to 234.5). Numerous cobbles and boulders were locally found within the lower zone of the sand stratum in boreholes L2-1 (north abutment) and L2-2 (pier), below depths of 21.0 m (elevation 235.6) and 23.2 m (elevation 233.6), respectively. This lower sand stratum exhibited dense/very dense relative density. N values ranged from 42 to 63.

The grain size distribution chart of representative samples of sand and silty sand is shown on Figure GS-L2-5. The water content of sand/silty sand soils varied from 13 to 24%.

4.7 Bedrock

Bedrock comprising of arkose in borehole L2-1 and argillite in boreholes L2-2 and L2-3 was encountered below the sand deposits. The bedrock was cored for 3.1 to 3.4 m at following levels:

LOCATION	BOREHOLE NO.	DEPTH (m)	ELEVATION	ROCK CORE LENGTH (1)
North abutment	L2-1	21.8	234.5	3.1
Pier	L2-2	28.0	228.8	3.4
South abutment	L2-3	25.9	230.1	3.2

(1) NQ diamond rock cores obtained.

The core recovery was 80 to 100% for all core samples. The bedrock exhibited a high strength and was found to be unweathered. The rock is of typically good to excellent quality as indicated by Rock Quality Designation (RQD) values which typically range from 54 to 100%. One lower RQD of 32% (poor quality) was recorded in the initial 1.5 m core run in borehole L2-2. The detailed rock core descriptions are provided on Table 1 and representative photographs of the cores are shown in Appendix B.

4.8 Groundwater

Groundwater was observed during drilling at depths of 1.2 to 3.1 m, elevations 253.2 to 254.8 in all boreholes. Upon completion of drilling and bedrock coring, the boreholes were charged with drill water from the rotary diamond drilling, therefore, the groundwater measurements were not carried out. In view of the relatively pervious nature of the sands and silts in the vicinity of the site, the groundwater levels are subjected to fluctuations due to seasonal variations and rainfall patterns.

5. MISCELLANEOUS

The field work was carried out under the supervision of Mr. F. Portela, Senior Technician, and the direction of Mr. C.M.P. Nascimento, P.Eng., Senior Foundation Engineer. Walker Drilling Ltd. and Marathon Drilling Inc. 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. C.M.P. Nascimento, P.Eng., with the assistance of Ms. N.S. Balakumaran, BSc., and independently reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for
Kantola Road/Fielding Road Underpass
Site No. 46-517
Highway 17 Sudbury Southwest Bypass Four-Laning
City of Greater Sudbury
GWP 5825-05-00
District 54, Sudbury, Ontario

6. ENGINEERING RECOMMENDATIONS

6.1 General

Part B of this report provides the preliminary foundation engineering recommendations regarding design and comments for construction of the proposed Kantola Road/Fielding Road Underpass at the future Highway 17. The recommendations are preliminary and based on the results of the limited subsurface investigation that was outlined in the Part A of this report.

Based on the preliminary drawing, the Kantola Road/Fielding Road Underpass will be a two-span structure with a total length of about 80 m between abutments. The proposed bridge will carry the traffic of the realigned Kantola Road/Fielding Road over the proposed Highway 17 four-laning configuration. For the purpose of this report we assumed that the existing grades of Highway 17 through the Kantola Road/Fielding Road intersection will remain unchanged. Therefore, the grades of the deck of the underpass are estimated about 7.5 to 8.0 m higher than the Highway 17 pavement level according to Stantec.

It is noted that the location of the bridge was moved after the field work was completed. The obtained data was considered adequate for preliminary design; however, boreholes should be drilled at the final structure location for Detail Design.

In summary, the soil stratigraphy revealed in the boreholes comprises units of surficial fill or topsoil overlying continuous deposits of soft to stiff clayey silt/clay underlain by cohesionless loose to compact silts which are in turn underlain by very loose to compact sand interlayered with sandy silt/silty sand. The sand deposits locally contained numerous cobbles and boulders at the north abutment and pier locations. The sand deposits mantle argillite and arkose bedrock. Groundwater was observed at 1.2 to 3.0 m elevation 253.2 to 254.8 in the process of drilling.

A list of the standard specifications referenced in this report is provided in Table 2.

6.2 Foundations

6.2.1 General

Based on the preliminary data, founding the proposed underpass on pile foundations driven to refusal on bedrock is considered feasible. The pile foundations may be used for integral or conventional abutment designs.

Placing the structure foundations on spread footing bearing on native soils is not considered feasible due to the compressible and soft to stiff cohesive soils underlain by marginally compact to loose/very loose cohesionless soils as well as presence of high groundwater table.

Caisson foundations are not considered practical in view of the encountered boulders and high groundwater conditions at the site.

The seismic site coefficient for the stratigraphic conditions at this site is 1.0 [soil profile Type I, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6].

The foundation frost depth for structure foundations at this site is 2.0 m, according to OPSD-3090.100.

6.2.2 Piles

For the preliminary design of piles for conventional or integral abutments, steel H-piles driven to refusal on bedrock should be used. The reference depths and elevations of the bedrock surfaces are summarized on following table.

FOUNDATION ELEMENT	REFERENCE BOREHOLE No.	FOUNDING DEPTH (m)	FOUNDING ELEVATION
North Abutment	L2-1	21.8	234.5
Pier	L2-2	28.0	228.8
South Abutment	L2-3	25.9	230.1

Based on high strength bedrock at the pile tips, the preliminary factored axial resistance at ULS for the two pile sections noted below is considered to be appropriate:

Pile Section	Factored Axial Resistance at ULS (kN)
HP 310 x 110	2,000
HP 310 x 132	2,280

The resistance at Serviceability Limit States (SLS) normally allows for 25 mm of 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 since the required loads causing appreciable deformation of the pile are much larger than the ULS factored capacity.

The piles will have to be driven through the native soils containing cobbles and boulders at the north abutment and pier locations. Consequently, there is a risk that the piles may reach refusal on this layer or may be damaged during heavy driving through cobbles above the bedrock surface. If the piles encounter refusal on the layer of boulders or are damaged during driving, additional or replacement piles should be installed.

The piles should be provided with driving shoes (SP 903S01) to minimize the potential for damage when driving through boulders and setting the piles on the bedrock.

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.

The lateral loading could be resisted fully or partially by battered piles. For vertical piles such as those used for integral abutments, the resistance to lateral loading will be derived from the soils in front of the piles. The lateral resistance recommended for the pile sections is as follows:

Steel H-Pile, 310 x 110 Steel H-Pile, 310 x 132	**FIRM/STIFF CLAY/ CLAYEY SILT**	**COMPACT/ DENSE SILT/SAND**	**GRANULAR BACKFILL**
Factored Lateral Resistance at ULS, kN	120	115	120
Lateral Resistance at SLS, kN	35	45	50

6.3 Approach Embankments

Boreholes were not carried out for the approach embankments to the Kantola Road/Fielding Road Underpass. The approach embankments should be founded on cohesionless soils following subexcavation of the 4.3 to 4.4 m thick soft to stiff compressible cohesive soils inferred from the boreholes drilled at the proposed north and south abutments. Further subsurface investigations should be carried out at these locations for detail design.

The approach embankments should be designed and constructed in accordance with OPSD-200.010, 201.010, 202.010 and SP 206S03. The side slopes of the approach embankments will be stable where they are inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rockfill.

It is noted that where the embankment fill height exceeds 8 or 10 m for earth and rockfill, respectively a 2 m wide mid-height berm will be required. The earth fill slopes, if employed, should be protected against surface erosion by sodding (OPSS 571) and suitable vegetation.

The backfill should be constructed with granular materials adjacent to the abutments to minimize the post-construction settlement of the road surface due to "consolidation" of the backfill. Assuming about 8 m high embankments adjacent to the abutments and about 4.0 m cohesive soils removed below existing grades, the anticipated settlement of the approach fill and native soils is in the order of 100 mm.

6.4 Construction Considerations

6.4.1 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 topsoil, cohesionless silty and sandy soils encountered in the boreholes are considered Type 3 soils. The cohesionless soils below the groundwater should be reclassified as Type 4 soils. The soft to stiff cohesive soils are considered Type 3 soil according to OHSA.

For slopes with multiple soil types, the slope geometry is governed by the soil type with the highest numerical designation.

6.4.2 Groundwater Control

Groundwater was observed during the course of the field work at depths ranging from 1.2 to 3.1 m (elevations 253.2 to 254.8) at the abutments and pier locations. Construction dewatering will be required to remove the about 4.0 m deep soft to stiff/very stiff cohesive soils found at the location of the approach embankments. It is considered that seepage from soil and surface water run-off that enters the excavation for pile caps should be readily handled by conventional sump pumping techniques.

Groundwater conditions should be further assessed during detail design by drilling boreholes to the full excavation depth contemplated for the proposed construction of the approach embankments.

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.

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

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

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 Ø = angle of internal friction of retained soil (35^o for Granular A or Granular B Type II or Type III)

δ = angle of friction between the soil and wall (23.5^o 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, Ø (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.

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 subdrain tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipes should be installed on a positive grade and lead to frost-free outlets.

7. SCOPE OF ADDITIONAL FOUNDATION INVESTIGATION

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 foregoing interpretation and recommendations are only provided for planning purposes and feasibility studies.

Based on the proposed structure configuration, the limited number of borehole data and a visual assessment of the underpass structure site, the recommended additional scope of the foundation investigation for detail design is as follows:

- One borehole should be carried out at each foundation element of the bridge structure. The borehole should extend a minimum of 3 m below refusal as defined by material for which Standard Penetration Test exceeds 100 blows per 0.3 m. If encountered, bedrock should be cored for 3.0 m.
- One probe should be advanced at the ends of the foundation elements to investigate the slope of the bedrock surface.
- One borehole should be carried out for each of the approach embankments and extend to the full height to the embankment and at least 3.0 m into competent soil to assess its adequacy to carry the embankment loading.

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 due to the compressible soils encountered. Suitable founding conditions for caissons were not encountered. Consequently, the foundation alternatives were limited to the use of driven piles and a discussion of their advantages and disadvantages is not considered to be necessary.

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

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

9. CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. C.M.P. Nascimento, P.Eng., with the assistance of Ms. N.S. Balakumaran, BSc, and independently reviewed by Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.

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CN/BRG:nb-lnr-mi

TABLE 1

ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE NO.	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
L2-1	18	21.8 – 22.8	100	100	21.8 – 24.9	ARKOSE: Dark grey, fine grained, with white encrustation and/or metallic mineralization on partings, high strength, unweathered, close to moderate spaced dipping partings, smooth to slickensided planar, tight, excellent quality
	19	22.8 – 24.3	100	100		
	20	24.3 – 24.9	100	100		
L2-2	18	28.0 – 29.5	80	32	28.0 – 31.4	ARGILLITE: Grey to dark grey, fine crystalline, (slate like appearance) dipping to near vertical schistocity, with metallic mineralization on partings, high strength, unweathered, close to moderate spaced dipping partings, smooth to slickensided planar, tight to oxidized, poor to good quality.
	19	29.5 – 30.8	98	71		
	20	30.8 – 31.4	92	88		
L2-3	20	25.9 – 26.5	88	54	25.9 – 29.1	ARGILLITE: Dark grey to black (greenish tinge), fine crystalline, dipping to near vertical schistocity, serpentine on some partings, occasional metallic mineralization on partings, high strength, unweathered, close to moderate spaced dipping partings, smooth to slickensided planar, tight, with vertical partings throughout, fair to excellent quality.
	21	26.5 – 27.3	100	90		
	22	27.3 - 28.5	98	65		
	23	28.5 – 29.1	100	100		

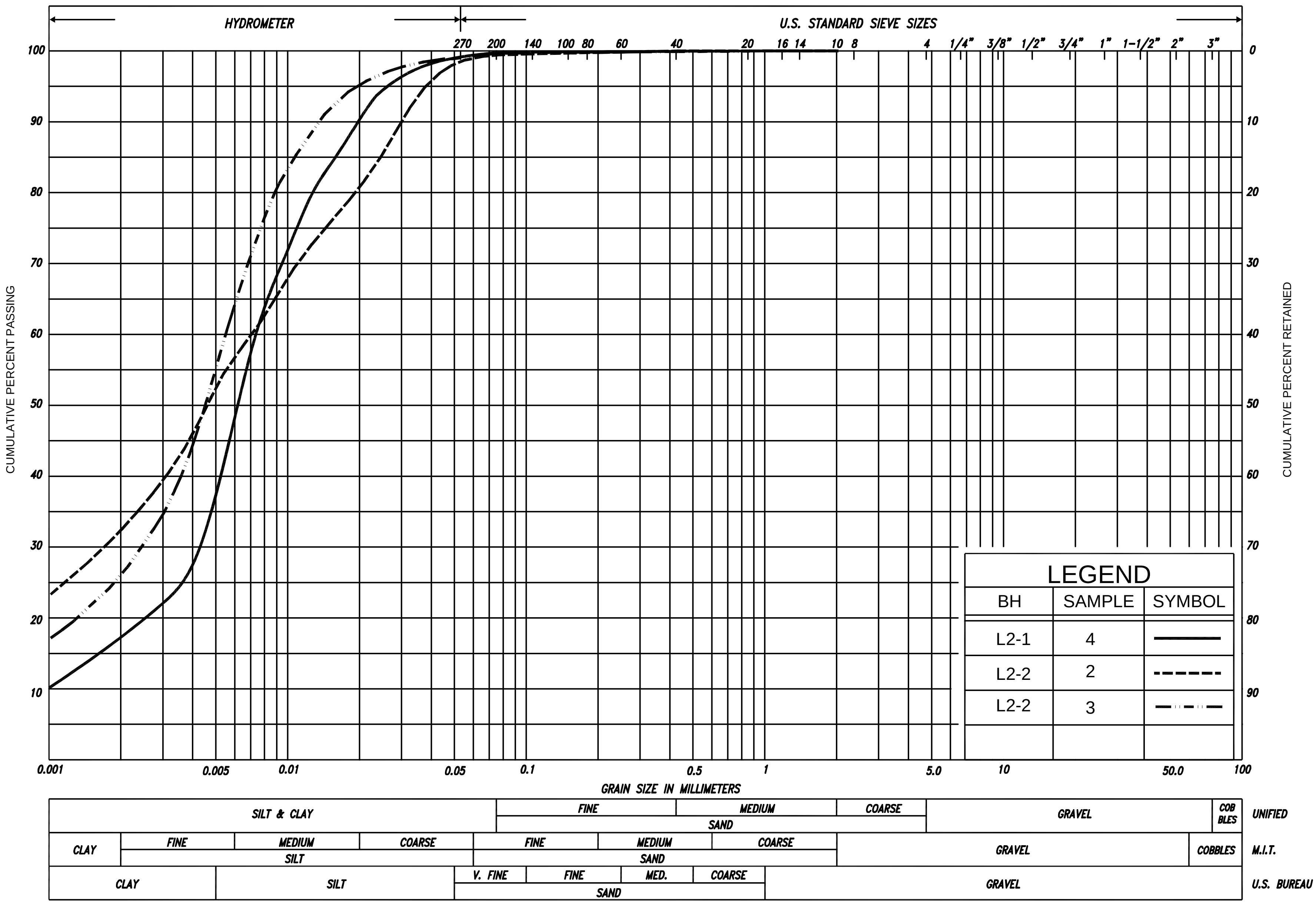
RQD: Rock Quality Designation

Originated: FP
Compiled: JFW
Checked: NB/CN

TABLE 2

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 571	Construction Specification for Sodding
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
SP 903S01	Construction Specification for Piling
OPSD-200.010	Earth/Shale Grading – Undivided Rural
OPSD-201.010	Rock Grading-Undivided Rural
OPSD-202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD-3090.100	Foundation Frost Depth for Northern Ontario
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail

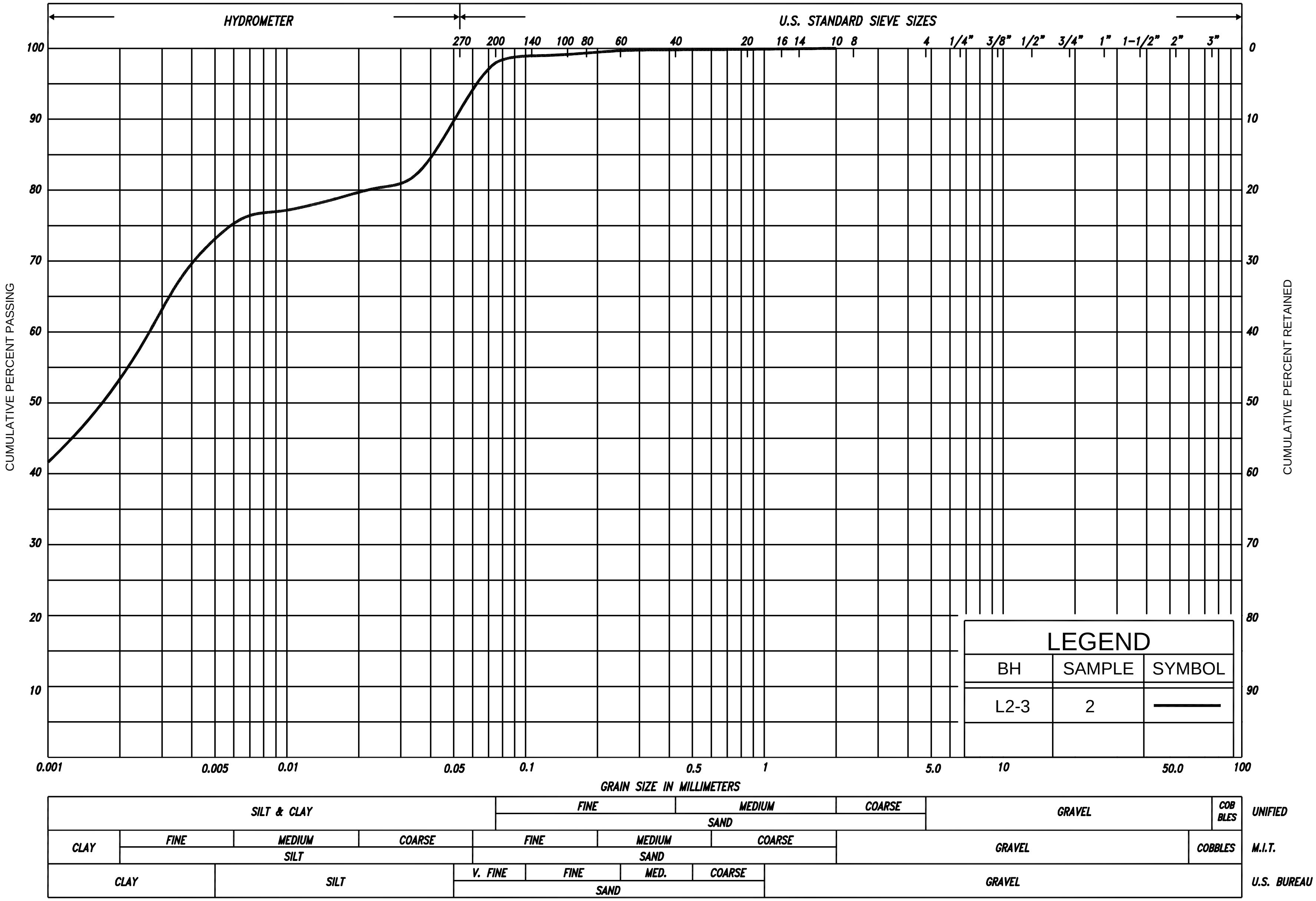


Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand

FIG No.	GS-L2-1
HWY	17
G.W.P. No.	5825-05-00



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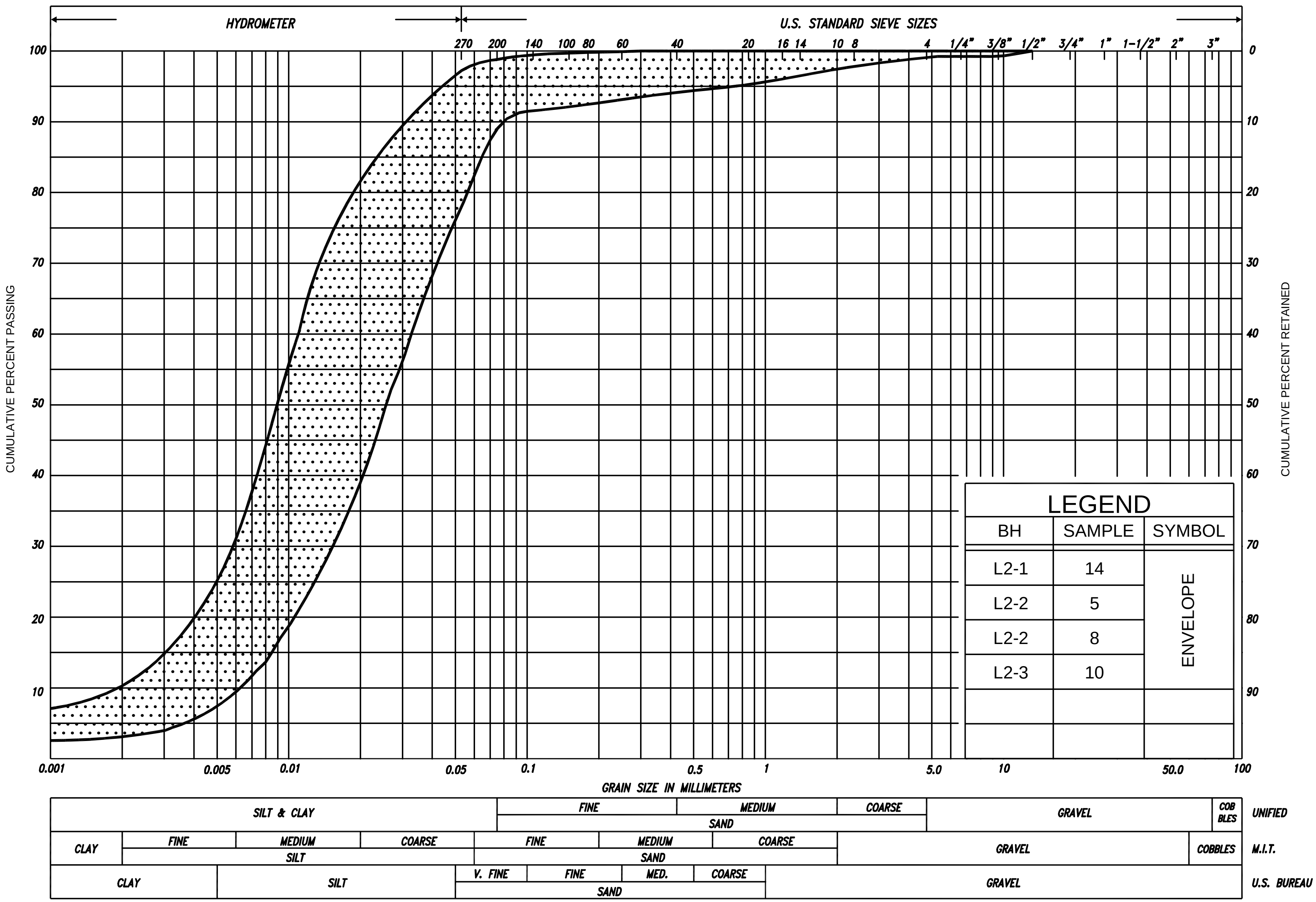
GRAIN SIZE DISTRIBUTION

CLAY, trace sand

FIG No.	GS-L2-2
HWY	17
G.W.P. No.	5825-05-00



LEGEND

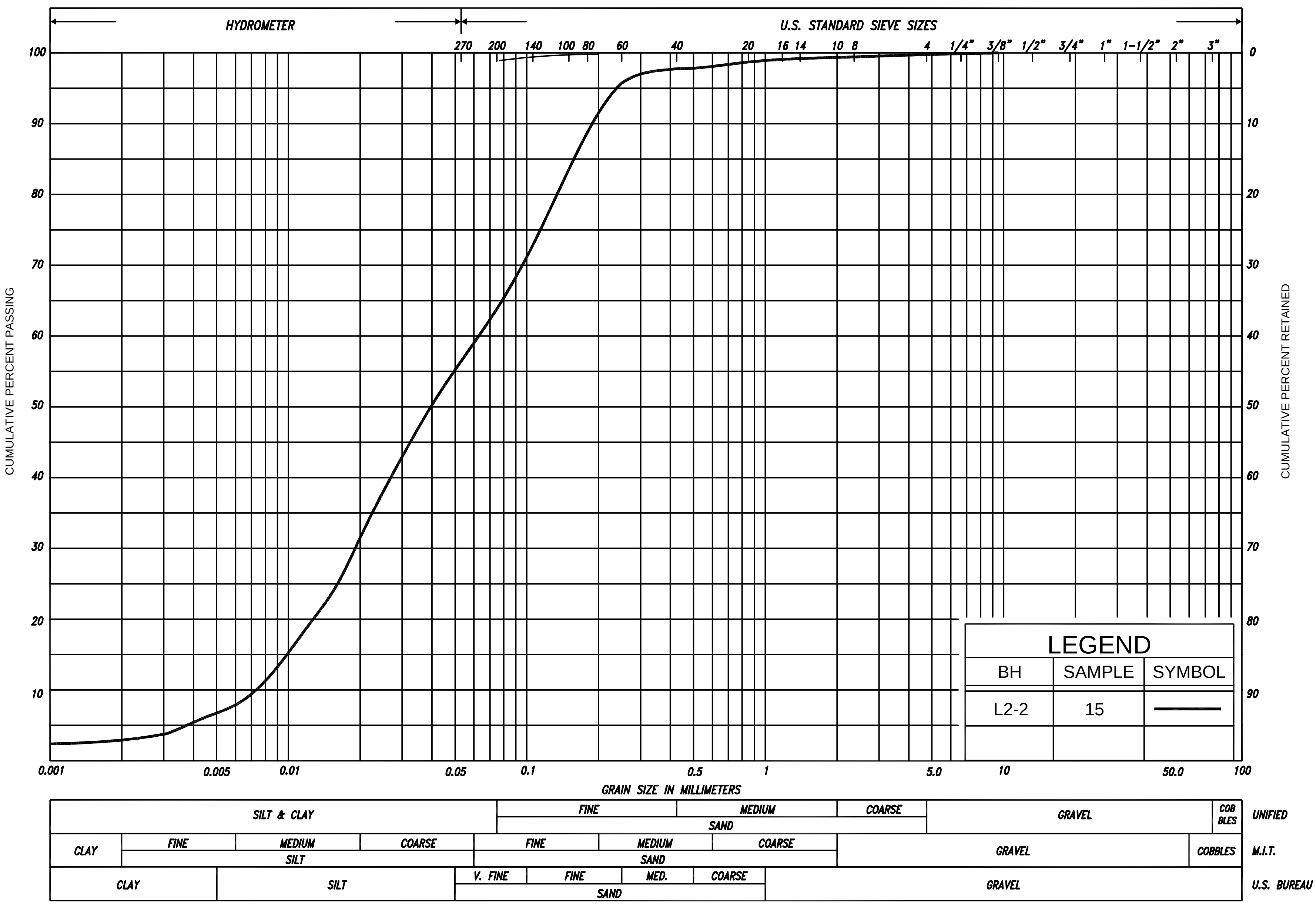
BH	SAMPLE	SYMBOL
L2-1	14	ENVELOPE
L2-2	5	
L2-2	8	
L2-3	10	

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILT, trace clay, trace sand, trace gravel

FIG No.	GS-L2-3
HWY	17
G.W.P. No.	5825-05-00

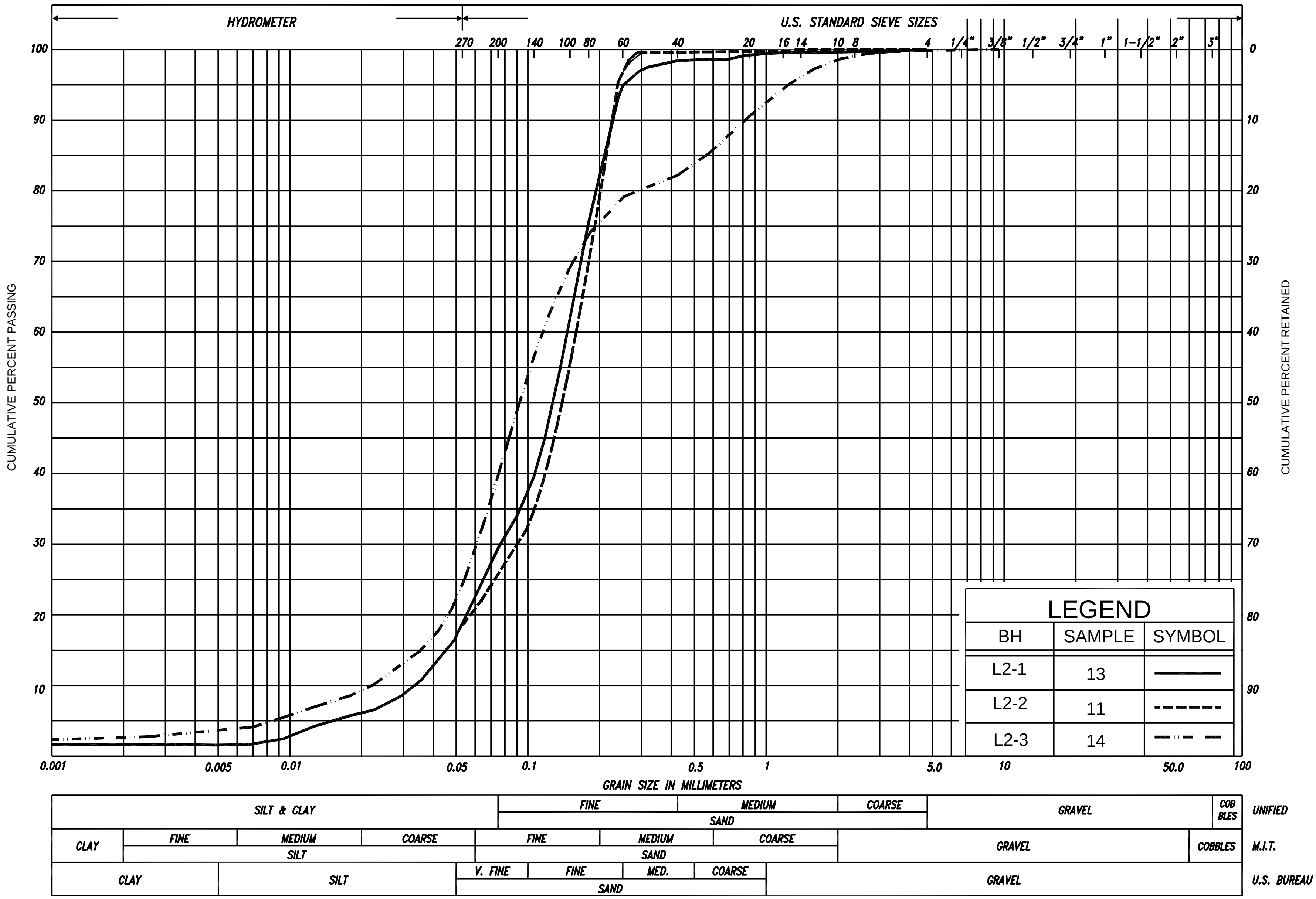


Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

SANDY SILT, trace clay, trace gravel

FIG No.	GS-L2-4
HWY	17
G.W.P. No.	5825-05-00



LEGEND

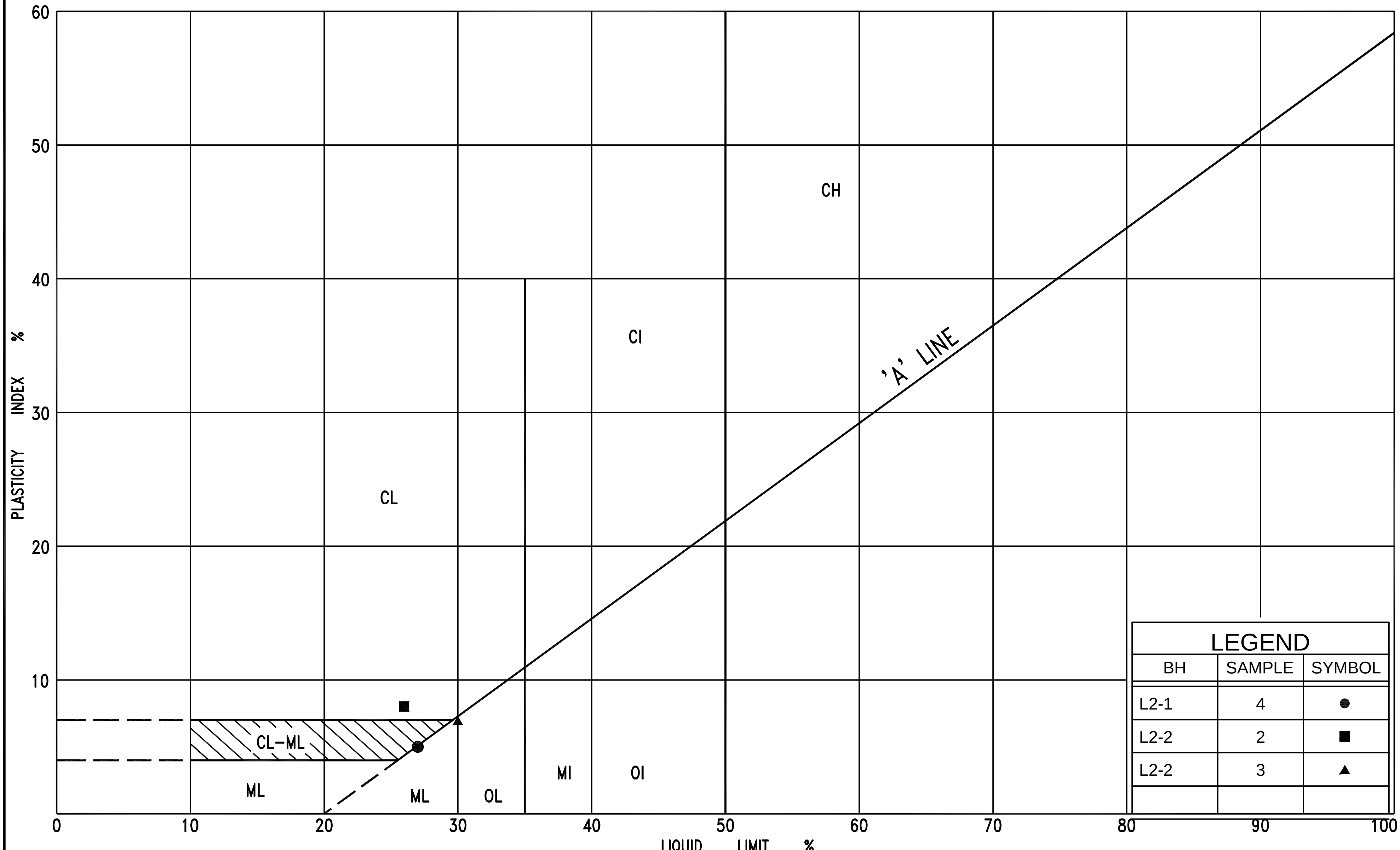
BH	SAMPLE	SYMBOL
L2-1	13	——— (solid)
L2-2	11	– – – (dashed)
L2-3	14	—·· — (dash-dot)

Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

SAND, with silt to SILTY SAND, trace clay

FIG No.	GS-L2-5
HWY	17
G.W.P. No.	5825-05-00



LEGEND

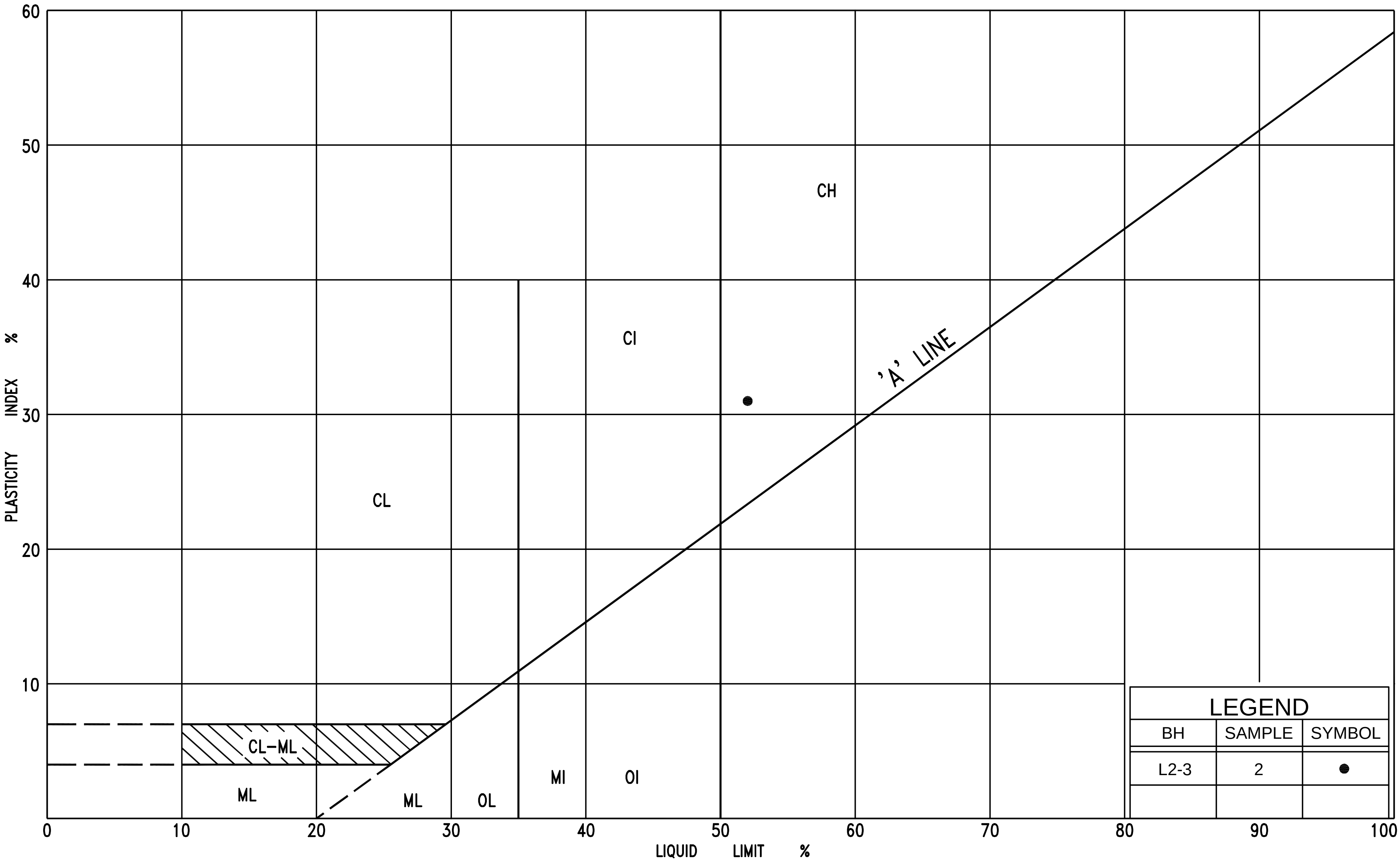
BH	SAMPLE	SYMBOL
L2-1	4	●
L2-2	2	■
L2-2	3	▲

Ministry of Transportation Ontario

PLASTICITY CHART

CLAYEY SILT, trace sand

FIG No.	PC-L2-1
HWY	17
G.W.P. No.	5825-05-00



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PLASTICITY CHART

CLAY, trace sand

FIG No.	PC-L2-2
HWY	17
G.W.P. No.	5825-05-00

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.

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
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
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No L2-1

METRIC

G.W.P. 5825-05-00	LOCATION Co-ords: 5 142 744 N; 297 267 E		ORIGINATED BY F.P.
DIST 54 HWY 17	BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring		COMPILED BY N.S.B
DATUM Geodetic	DATE June 19, 2007		CHECKED BY NB/CN

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.3	Ground Surface						
0.0	Topsoil						
0.2	Silt, trace sand Loose Brown Moist	1	SS	4		256	
	with sand	2	SS	10			
254.9 1.4	Clayey silt layers of silt Soft to stiff Brown Wet					255	
		3	SS	5			
		4	SS	8		254	0 0 83 17
					∇*		
		5	SS	8		253	
		6	SS	7			
252.0 4.3	Silt, trace clay Loose to compact Brown Wet					252	
		7	SS	9			
		8	SS	7		251	
		9	SS	10		250	
						249	
		10	SS	18			
						248	
	layers of sandy silt	11	SS	7		247	
						246	
	trace gravel Grey	12	SS	18			
244.7 11.6	Sand with silt, trace clay Compact Grey Wet					245	
		13	SS	12		244	0 70 28 2
243.2 13.1	Silt, some sand trace clay, trace gravel layers of sand Compact Grey Wet					243	
		14	SS	22			1 10 84 5
						242	
	Cont'd						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

+7, ×5: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No L2-1

METRIC

G.W.P. 5825-05-00 LOCATION Co-ords: 5 142 744 N; 297 267 E ORIGINATED BY F.P.

DIST 54 HWY 17 BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring COMPILED BY N.S.B

DATUM Geodetic DATE June 19, 2007 CHECKED BY NB/CN

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.3							
	Dense	15	SS	32	241	o	
240.1 16.2	Sand trace silt, trace gravel cobbles. Very dense Grey Wet to dense				240		
		16	SS	63	239	o	
		17	SS	49	238	o	
					237		
					236		
	cobbles and boulders 0.3m thick	17A	RC NQ	**	235		
234.5 21.8	Bedrock. Arkose. Dark grey. High strength. Unweathered. Excellent quality	18	RC NQ	REC 100%	234		RQD 100%
		19	RC NQ	REC 100%	233		RQD 100%
		20	RC NQ	REC 100%	232		RQD 100%
231.4 24.9	End of borehole						

* 2007 06 19

∇ Water level observed during drilling

** Rotary diamond coring from 21.0m due to cobbles and boulders

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

+7, ×5 : Numbers refer to Sensitivity   20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No L2-2

METRIC

G.W.P. 5825-05-00	LOCATION Co-ords: 5 142 711 N; 297 259 E		ORIGINATED BY F.P.
DIST 54 HWY 17	BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring		COMPILED BY N.S.B
DATUM Geodetic	DATE May 09, 15 and 16, 2007		CHECKED BY NB/CN

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE / SHEAR STRENGTH	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.8	Ground Surface									
0.0	Topsoil									
0.2	Clayey silt, trace sand layers of silt Stiff Brown Wet	1	SS	2						
		2	SS	10		256				0 1 66 33
		3	SS	7		255				0 1 73 26
		4	SS	11	∇*	254				
253.7 3.1	Silt, trace clay trace sand, trace gravel Compact Brown Wet to Loose	5	SS	12				Non-plastic		1 1 88 10
		6	SS	10		253				
		7	SS	10		252				
						251				
		8	SS	13		250				0 2 95 3
		9	SS	4		249				
						248				
		10	SS	6		247				
246.7 10.1	Sand with silt, trace clay Very loose Brown Wet to compact									
		11	SS	3		246				0 75 (25)
	____ ____ ____ ____ Grey					245				
		12	SS	10		244				
243.7 13.1	Sandy silt trace clay, trace gravel Loose Grey Wet to compact					243				
		13	SS	7						
	Cont'd					242				

Plot scales: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

+7, ×5 : Numbers refer to Sensitivity

20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No L2-2

METRIC

G.W.P. 5825-05-00 LOCATION Co-ords: 5 142 711 N; 297 259 E ORIGINATED BY F.P.

DIST 54 HWY 17 BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring COMPILED BY N.S.B

DATUM Geodetic DATE May 09, 15 and 16, 2007 CHECKED BY NB/CN

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.8							
		14	SS	10	241	≈25	
		15	SS	16	240	≈24	1 35 61 3
		16	SS	25	239 / 238	≈17	
236.4 20.4	Sand, trace silt, trace gravel, cobbles. Dense Grey Wet				237 / 236		
		17	SS	42	235	≈20	
	cobbles and boulder 0.6m thick				234		
		17A	RC NQ	**	233		
	Boulder 0.2m thick	17B	RC NQ	**	232		
	Boulder 0.3m thick	17C	RC NQ	**	231 / 230		
228.8 28.0	Bedrock, Argillite, Grey to dark grey, High strength, Unweathered, Poor to good quality	18	RC BQ	REC 80%	229 / 228 / 227		RQD 32%
	Cont'd						

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT.

+7, ×5 : Numbers refer to Sensitivity   20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
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RECORD OF BOREHOLE No L2-2

METRIC

G.W.P. 5825-05-00 | LOCATION Co-ords: 5 142 711 N; 297 259 E | ORIGINATED BY F.P.

DIST 54 | HWY 17 | BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring | COMPILED BY N.S.B

DATUM Geodetic | DATE May 09, 15 and 16, 2007 | CHECKED BY NB/CN

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L (WATER CONTENT (%))	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
226.8											
			19	RC BQ	REC 98%						RQD 71%
							226				
225.4			20	RC BQ	REC 92%						RQD 88%
31.4	End of borehole										

* 2007 05 09

∇ Water level observed during drilling

■ Penetrometer test

** Rotary diamond drilling from 23.2m due to cobbles and boulders

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

Legend: SHEAR STRENGTH kPa — O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

+7, ×5: Numbers refer to Sensitivity

20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No L2-3

METRIC

G.W.P. 5825-05-00	LOCATION Co-ords: 5 142 655 N; 297 271 E		ORIGINATED BY F.P.
DIST 54	HWY 17	BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring	COMPILED BY N.S.B
DATUM Geodetic	DATE June 20 and 21, 2007		CHECKED BY NB/CN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
256.0	Ground Surface
0.0	Silty clay, organics sand and gravel inclusions Firm Brown Moist (FILL)
0.3	Clay, trace sand Stiff Brown Moist; sandy silt lenses; Soft to firm; silt layers; very stiff
251.6 / 4.4	Silt trace clay, trace sand Compact Grey Wet
244.4 / 11.6	Sandy silt, trace clay sand lenses Compact Grey Wet
242.6 / 13.4	Silty sand, trace clay Compact Grey Wet
	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5	
2	SS	4	0 2 45 53
3	SS	4	
4	SS	3	
5	SS	3	
	FV		
6	SS	13	
7	SS	12	
8	SS	17	
9	SS	8	
10	SS	18	0 1 93 6
11	SS	21	
12	SS	21	
13	SS	18	
14	SS	16	0 60 38 2

GROUND WATER CONDITIONS: ∇* (at approx. elev. 255)

ELEVATION SCALE: 255, 254, 253, 252, 251, 250, 249, 248, 247, 246, 245, 244, 243, 242

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

+[7], ×[5]: Numbers refer to Sensitivity

20 / 15 —○— 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No L2-3

METRIC

G.W.P. 5825-05-00 LOCATION Co-ords: 5 142 655 N; 297 271 E ORIGINATED BY F.P.

DIST 54 HWY 17 BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring COMPILED BY N.S.B

DATUM Geodetic DATE June 20 and 21, 2007 CHECKED BY NB/CN

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%) (approx.)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
241.0						
		15	SS	26	~20	
239.5 / 16.5	Sandy silt, trace clay sand lenses. Loose to compact, Grey, Wet					
		16	SS	8	~24	
		17	SS	20	~21	
235.6 / 20.4	Silty sand. Dense, Grey, Wet					
		18	SS	31	~22	
233.1 / 22.9	Sand, trace silt. Dense, Grey, Wet					
		19	SS	57	~21	
230.1 / 25.9	Bedrock. Argillite. Dark grey to black. High strength. Unweathered. Fair to excellent quality	20	RC NQ	REC 88%		RQD 54%
		21	RC NQ	REC 100%		RQD 90%
		22	RC NQ	REC 98%		RQD 65%
		23	RC NQ	REC 100%		RQD 100%
226.9 / 29.1	End of borehole					

Cont'd

Elevation scale: 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227

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

+7, ×5: Numbers refer to Sensitivity. 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No L2-3

METRIC

G.W.P. 5825-05-00 LOCATION Co-ords: 5 142 655 N; 297 271 E ORIGINATED BY F.P.

DIST 54 HWY 17 BOREHOLE TYPE C.F.H.S.A. + Rotary Diamond Coring COMPILED BY N.S.B

DATUM Geodetic DATE June 20 and 21, 2007 CHECKED BY NB/CN

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
226.0											

Sample 16: N - value affected by hydraulic disturbance

* 2007 06 20

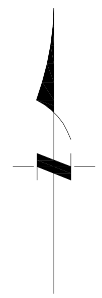
∇ Water level observed during drilling

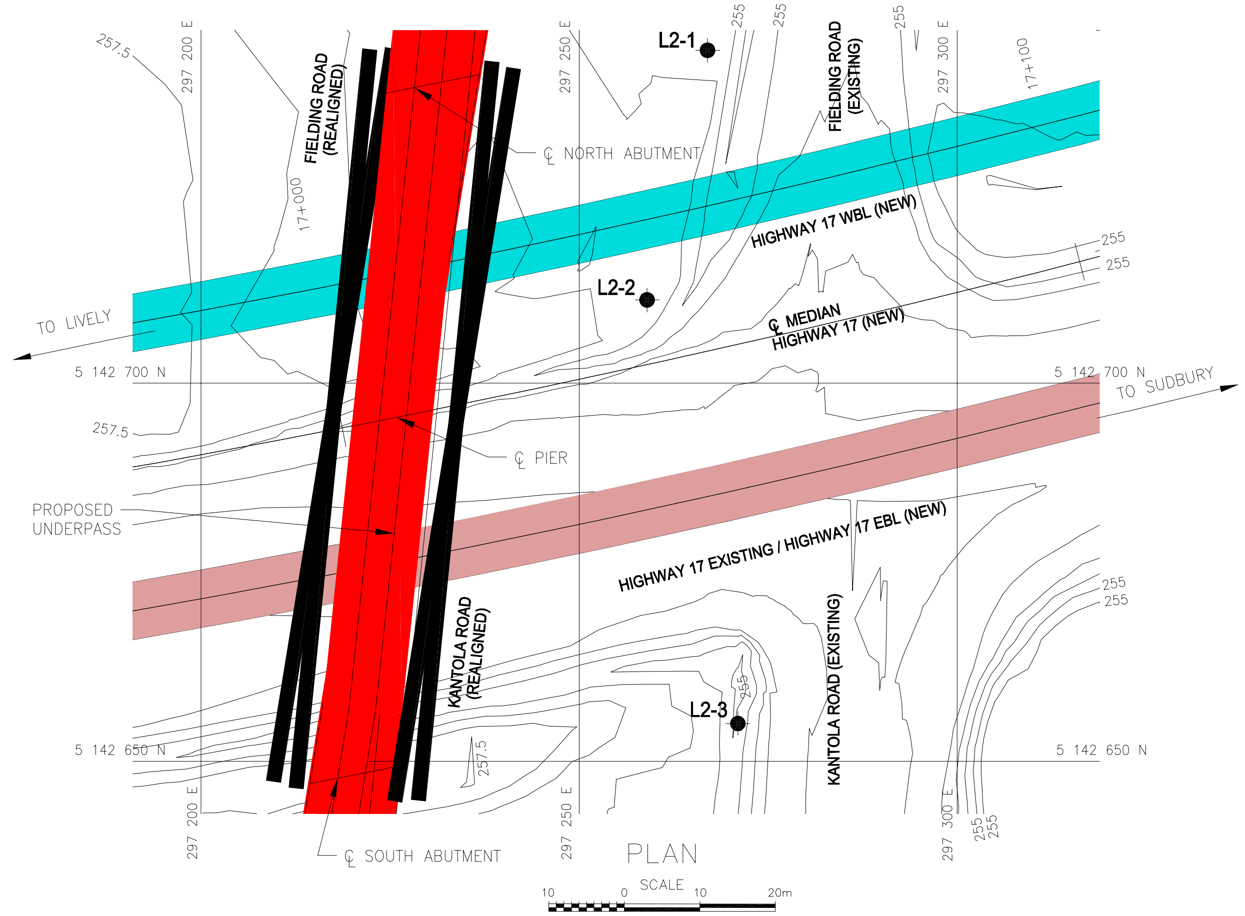
■ Penetrometer test

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

+7, ×5 : Numbers refer to Sensitivity

20 / 15—○—5 / 10 (%) STRAIN AT FAILURE





METRIC

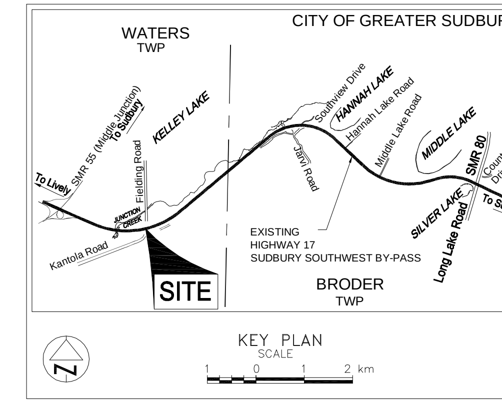
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
GWP No 5825-05-00

FIELDING ROAD/KANTOLA ROAD UNDERPASS
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS

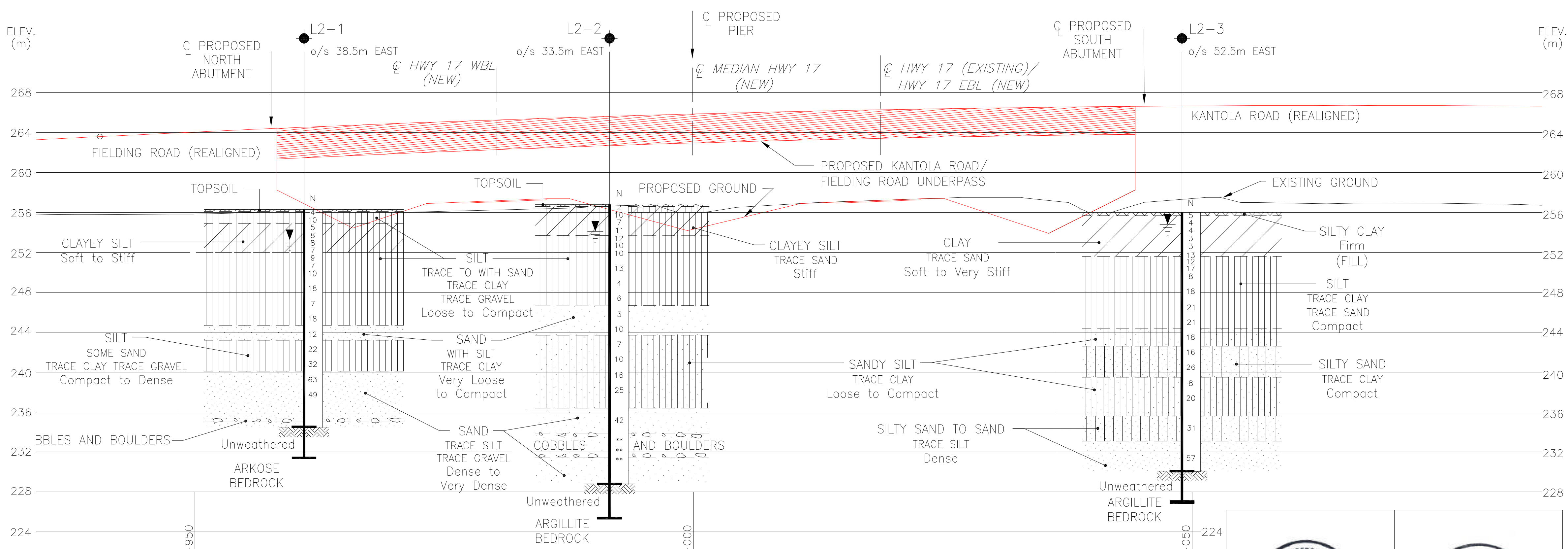


KEY PLAN
SCALE

LEGEND

Symbol	Description
●	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 May–June 2007
▽	Head ARTESIAN WATER Encountered
▮	PIEZOMETER

BH No	ELEVATION	COORDINATES NORTHINGS	COORDINATES EASTINGS
L2-1	256.3	5 142 744	297 267
L2-2	256.8	5 142 711	297 259
L2-3	256.0	5 142 655	297 271



℄ PROFILE
FIELDING ROAD/ KANTOLA ROAD (REALIGNED)
SCALE

NOTE:

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
2. NEW UNDERPASS LOCATION MOVED 40m WESTERLY FROM ORIGINALLY SELECTED PREFERRED ALIGNMENT.

REF No. Stantec Drawings; 599_Alternate 1.dwg; 599_base.dwg; 599_Contours.dwg; Received January 16, 2008

– 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. 41I-227

HWY No 17			DIST 54
SUBM'D NSB	CHECKED CN	DATE JULY 08, 2008	SITE 46-517
DRAWN NA	CHECKED CN	APPROVED BRG	DWG L2-1

APPENDIX A

Site Photographs

Photograph 1: Looking north at borehole L2-3 from west shoulder of Kantola Road (at about 15 m west of existing Highway 17 and Kantola Road intersection). Proposed centre pier at first light pole. Note swampy areas exist east and west of Kantola Road / Fielding Road.

Photograph 2: Looking north from west ditch slope area of Fielding Road at Borehole L2-2. Borehole L2-1 about 35 m drilled further north.

APPENDIX B

Rock Core Photographs

Photograph 1: Arkose rock core from borehole L2-1, RC-18 and RC-19. (RQD 100% for both runs)

Photograph 2: Arkose rock core from borehole L2-1, RC-19 and RC-20. (RQD 100% for both runs)

Photograph 3: Argillite rock core from L2-2, RC-18, RC-19 and RC-20. RQD value of 32% for RC-18, 71 and 77% for RC-19 and RC-20.

Photograph 4: Argillite rock core from borehole L2-3, RC-20 and RC-21. RQD values of 54 and 90%.

Photograph 5: Argillite rock core from borehole L2-3, RC-22 and RC-23. RQD values of 65 and 100%.