Peto MacCallum Ltd.
CONSULTING ENGINEERS

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

SOUTHVIEW DRIVE OVERPASS WESTBOUND
SITE NO. 46-518/2
HIGHWAY 17 SUDBURY SOUTHWEST BYPASS FOUR-LANING
CITY OF GREATER SUDBURY
GWP 5825-05-00
TOWNSHIP OF BRODER
DISTRICT 54, SUDBURY, ONTARIO

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TABLE OF CONTENTS

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
Southview Drive Overpass Westbound
Site No. 46-518/2
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City of Greater Sudbury
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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 Southview Drive Overpass Westbound 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 single span overpass will carry the new Highway 17 westbound lanes over the realigned Southview Drive at about Station 11+885 (new Highway 17 four-lane chainage).

Stantec provided the preliminary layout and ground surface profile for the proposed Southview Drive Overpass Westbound for use in this investigation.

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

2. SITE DESCRIPTION AND GEOLOGY

The study area is located in the Township of Broder, approximately 150 m east of the existing at-grade intersection of Southview Drive and Highway 17. Photographs of the crossing site are enclosed in Appendix A.

The study area is essentially undeveloped, with a few residences located south of the Highway 17 and Southview Drive intersection (Jarvi Road).

The overpass site is generally located in the Huronian Area of the Canadian Shield where the typical geology is comprised of bedrock outcrops alternating with swamps and glaciolacustrine deposits.

Exposed rock outcrops and ridges are noted along the existing Highway 17 alignment, where rock cuts up to 8 m high (east of existing Southview Drive) were blasted to construct the existing highway platform.

In addition, swamps occur between rock outcrops, north of the site area, and at the intersection of existing Southview Drive with Highway 17. The swampy sections are traversed by means of embankments up to about 6 m high.

The overpass will be located across the existing embankment. The study area contains a few forested areas mostly with birch trees and a few evergreens and the lowlands are covered with grasses, brush and alders. The drainage of the study corridor is influenced by the typically shallow rocky and undulating terrain of the Canadian Shield, exhibiting relatively good drainage characteristics in the hilly outcrop sections and poorly drained conditions in the swampy lowland sections.

3. INVESTIGATION PROCEDURES

The subsurface investigation included four boreholes, L5-1 to L5-4, and was carried out during the period between April 23 and May 1, 2007. The four boreholes were advanced through the soil cover to depths of 0.1 to 8.5 m at the locations shown on Drawing SDW-1. In addition, the four boreholes, L5-1 to L5-4, were cored a minimum of 3.0 m into the bedrock to depths of 3.5 to 11.5 m.

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

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. The 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 rig, equipped for NQ diamond rock coring, supplied and operated by a specialist drilling contractor. The drilling crews worked under the full-time supervision of a member of our engineering staff. Photographs of the rock cores are shown in Appendix A.

All boreholes were backfilled in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment procedures.

Representative samples of the soils encountered in the boreholes were recovered at 0.75 to 1.5 m depth intervals. Soil samples were obtained using a split spoon sampler in conjunction with standard penetration tests. Where standard penetration tests were not carried out, the consistency/relative density of the encountered soils and rockfill was estimated from manual examination or the rate (ease) of advances of the augers. One Penetrometer test was carried on a stiff sample of clay. The Penetrometer results are typically lower than the actual values due to sample disturbance.

The groundwater conditions at the borehole locations were assessed by visual examination of the soil, and where appropriate, by measurement of the water level in the open holes. The water level observations are noted on the attached Record of Borehole sheets.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. The laboratory testing program consisted of 9 moisture content determinations, 4 grain size analyses and 2 Atterberg limits determinations. The grain size distribution charts are reported on Figures GS-L5-1 to GS-L5-3 and the Atterberg plasticity charts on Figures PC-L5-1 and PC-L5-2. The laboratory test results are included on the Record of Borehole sheets.

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 of the Southview Drive Overpass Westbound are shown on the Drawing SDW-1.

The soil stratigraphy in the boreholes drilled north of the existing Highway 17 is comprised of local deposits of sand/sand and gravel fill locally covering rockfill material overlying clayey material which in turn covers localized deposits of silt (borehole L5-1) mantling bedrock. On the south side of Highway 17 the boreholes were drilled through very shallow rockfill placed over a massive rock outcrop.

4.2 Fill

A surficial deposit of fill material made up of a mixture of sand, gravel, silt and clay was encountered in boreholes L5-1 and L5-3 extending to 5.2 and 1.6 m below ground surface, from elevations 263.1 and 266.2 to elevations 257.9 and 264.6, respectively. Inclusions of asphalt were encountered in the fill material in borehole L5-3. This fill was in a loose to compact condition with N values ranging from 8 to 17.

Below this fill in borehole L5-3, rockfill which comprised mostly of cobbles and boulders in a gravelly sand matrix was encountered to 5.2 m depth, elevation 261.0. In boreholes L5-2 and L5-4 the rockfill was 700 and 100 mm thick, respectively, extending from elevations 263.0 and 265.8 to elevations 262.3 and 265.7.

4.3 Clay

A localized 1.8 m thick deposit of clay was encountered in borehole L5-1 below the fill material extending to 7.0 m below ground surface, elevation 256.1. The consistency of the clay material is stiff. One N value of 10 was obtained. The natural moisture content determined for the clay was 39%.

The grain size distribution chart of one clay sample is presented in Figure GS-L5-2. The plasticity chart of the sample is presented in Figure PC-L5-1.   The determined liquid and plastic limits of the clay are 57 and 23, respectively, giving a plasticity index of 34.

4.4 Silt

A 1.5 to 2.4 m thick deposit of silt was encountered in borehole L5-1 below the clay layer and below the fill material in borehole L5-3. The silt extended to 8.5 and 7.6 m depths, elevations 254.6 and 258.6, respectively, where the boreholes encountered bedrock. The relative density of the silt is compact. N values of 24 and 28 were obtained. One natural moisture content was determined to be 23%.

The grain size distribution chart of one silt sample is presented in Figure GS-L5-3. The plasticity chart of the sample is presented in Figure PC-L5-2.   The determined liquid and plastic limits are 19 and 18, respectively, with a plasticity index of 1, indicating that the material is non-plastic.

4.5 Bedrock

A detailed description of the rock cores retrieved from boreholes L5-1 to L5-4 is provided in Table A and summarized on the record of borehole logs.

At borehole locations L5-1 and L5-3, drilled north of Highway 17, bedrock was encountered below the silt at depths of 8.5 and 7.6 m below ground surface, elevations 254.6 and 258.6, respectively. The bedrock was confirmed in boreholes L5-1 and L5-3 by drilling two core holes of 3.0 m extending to 11.5 and 10.6 m below ground surface, elevations 251.6 and 255.6, respectively. It is inferred from the borehole logs and visual inspection of the site that the bedrock surface is locally sloping down from east to west along the Highway 17 corridor.

The bedrock comprises medium to dark grey argillite in both boreholes. The measured core recovery is typically 100%, with two isolated values of 93% in borehole L5-1 and 72% in borehole L5-3. The Rock Quality Designation (RQD) determined from the borehole L5-1 rock cores ranges from 70% to 100%, indicating good to excellent quality rock. The condition of the cores is unweathered exhibiting high strength. The range of RQD for the borehole L5-4 cores is between 55 and 100%, indicating fair to excellent quality. The cores retrieved exhibit high strength and are in unweathered condition.

At borehole locations L5-2 and L5-4, bedrock was encountered below fill materials at 0.7 and 0.1 m depths, elevations 262.3 and 265.7, respectively. The bedrock was confirmed in boreholes L5-2 and L5-4 by drilling two core holes, 3.2 and 3.4 m long, respectively to depths of 3.9 and 3.5 m, elevations 259.1 and 262.3.

The bedrock retrieved from boreholes L5-2 and L5-4 comprises light to dark grey argillite in both boreholes. The measured core recovery is typically 90% to 100%, with two isolated values of 42% in borehole L5-2 and 53% in borehole L5-4. The RQD determined from the L5-2 rock cores ranges from 42% to 73%, indicating poor to fair quality rock. The condition of the cores is slightly weathered to unweathered exhibiting high to very high strength. The range of RQD values for the L5-4 cores is between 53 and 85%, indicating fair to good quality. The cores recovered exhibit high strength and are in unweathered condition.

4.6 <u>Groundwater</u>

Groundwater was observed in borehole L5-1 during and upon completion of drilling at 7.0 and 7.6 m, respectively, below ground surface. Borehole L5-2 and L5-4 were charged with drillwater upon completion of the rock coring.

No groundwater was encountered in borehole L5-3 during and upon completion of drilling.

Groundwater levels are subjected to fluctuations due to seasonal 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 Inc. supplied the soil and rock core 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 Mr. N. Rahman, BASc, and independently reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART B
PRELIMINARY FOUNDATION DESIGN REPORT

for
Southview Drive Overpass Westbound
Site No. 46-518/2
Highway 17 Sudbury Southwest Bypass Four-Laning
City of Greater Sudbury
GWP 5748-04-00
District 54, Sudbury, Ontario

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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 Southview Drive Overpass Westbound for the proposed four-laning of the Highway 17 Sudbury Southwest Bypass. 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 that Stantec prepared for the Southview Drive Overpass Westbound, we assumed that the proposed bridge deck will be at about elevation 267.5, about 7.5 m above the realigned Southview Drive pavement (elevation 260.0). The Highway 17 westbound approach embankments will be also about 7.5 to 8.0 m high at the abutments.

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.

Construction of the overpass structure foundations is expected to require bedrock cuts up to 5.0 m deep at the south end of the abutments. Standard rock excavation and construction procedures should be suitable. The rock blasting should be controlled to prevent disturbance to the existing residences on the nearby Jarvi Road, highway embankment and utilities and to preserve the condition of the bedrock founding level. A precondition survey of the existing highway embankment and residences should be conducted prior to construction. Restrictions may be required to limit the effects of rock excavation by blasting (noise and vibration).

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Construction of the overpass will require the removal of the existing rockfill at and beyond the proposed structure location. It is recommended that the new structure be constructed after completion of the EBL overpass to allow for the closure of the existing highway. This will avoid the difficult road protection requirements using shoring through the rockfill embankment for traffic staging.

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

6.2 Foundations

6.2.1 General

Based on the preliminary data, placing spread footings on the existing fill or native soils is not feasible since these soils will not provide adequate bearing resistances. However, it is considered that placing the structure foundations on spread footings bearing on bedrock or structural fill is feasible. Footings may be used for conventional or semi-integral abutment design.

An alternative scheme with pile foundations at the west and east abutments for integral abutments is also possible. Normally, this alternative will require the excavation of a trench in the bedrock where necessary to accommodate the minimum free pile length of 5 m below the abutment stem.

We consider that drilled cast-in-place concrete caissons are not practical to support the foundations for this site due to the presence of cobbles/boulders, sloping bedrock and wet soil cover that would cause construction difficulties.

The foundation frost depth for structure foundations at this site is 2.0 m, according to OPSD-3090.100. Frost protection is not required for spread footings placed directly on the bedrock.

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].

6.2.2 Spread Footings on Bedrock

For preliminary design, the abutment footings were assumed to be a minimum of about 3.0 m below the level of the bridge deck, that is about elevations 264.5 and 265.0 at the west and east abutments respectively. The following are the anticipated depths and elevations of bearing surfaces for the abutment spread footings founded on bedrock.

FOUNDATION ELEMENT	PML BH NO.	FOUNDING DEPTH (m)	FOUNDING ELEV.
West Abutment	L5-1	8.5	254.6
	L5-2	0.7	262.3
East Abutment	L5-3	7.6	258.6
	L5-4	0.1 *	265.7 *

Notes: Founding depths refer to existing grades
(*) Rock excavation will be required to the founding level (estimated at about elevation 265.0).

For preliminary design purposes, the bedrock is assumed to be high strength and recommended preliminary geotechnical resistances for footings bearing on bedrock are as follows:

Factored Geotechnical Resistance at ULS, kPa	8,000
Geotechnical Resistance at SLS, kPa	N/A

The geotechnical resistance at Serviceability Limit States (SLS) normally allows for 25 mm of total compression of the founding medium. Considering the bedrock to be unyielding, the design is not governed by settlement criteria since the loading required to produce the above deformation of the bedrock would be larger than the factored geotechnical resistance at Ultimate Limit States (ULS).

The recommended geotechnical resistance values apply to vertical and concentric loads only. The designer should consider the effects of inclined loads and eccentricity, as applicable.

The lateral loads imposed on the foundations will be partly resisted by the friction developed between the underside of the concrete footing and the bedrock. These forces should be calculated in accordance with the CHBDC. An unfactored coefficient of friction of 0.7 may be assumed between concrete footings and the bedrock.

6.2.3 Spread Footings on Structural Fill

Supporting part of the abutment footings on structural fill placed in the approach embankments could also be employed. The foundation level for the structural fill pad is variable and slopes down to the northwest. Assuming that the footings are placed at about elevation 262.0 (about 2 m above the Southview Drive level with earth cover provided for frost protection), the structural fill pads would be about 4.0 to 7.4 m thick (from elevations 254.6 and 258.0) at the north end of the abutments. The footings would be founded on rock at the south end of the abutments, elevations 262.3 and 265.7 in boreholes L5-2 and L5-4, respectively.

The structural fill should consist of Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density. Footings should not be constructed on rock fill. However, rock fill may be placed adjacent to the Granular A core. A sketch of the recommended engineered fill construction scheme is shown on Figure A, attached.

The recommended bearing resistance for minimum 2.0 m wide footings constructed on structural fill (bearing resistance is independent of fill thickness at this location because the engineered fill should be placed directly on the bedrock) is as follows:

Factored Geotechnical Resistance at ULS, kPa	900
Geotechnical Resistance at SLS, kPa	350

The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 2.0 m was assumed for computation of the ULS resistance.

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

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 structural fill. An unfactored friction factor of 0.70 is recommended for footings on structural fill.

6.2.4 Piles

For the preliminary design of piles for integral abutments, steel H-piles driven to refusal on the bedrock underlying the site should be used. The anticipated depths and elevations of the bedrock surface are the same as indicated for spread footings.

The design of integral abutments should be evaluated from the economic viewpoint. Assuming the west and east abutment stems extending to about elevations 264.5 and 265.0, respectively, the pile tips will have to be established at or below about elevations 259.5 to 260.0, subject to structural detail design. Trenches up to 2.8 and 5.7 m deep (at the southern borehole locations) will have to be excavated into the bedrock to provide the minimum 5.0 m pile length required below the abutment stem.

Based on high bedrock strength assumed at the base of the excavated rock trenches, the preliminary factored axial resistances at ULS for the three pile sections noted below should be used:

Pile Section	Factored Axial Resistance at ULS (kN)
HP 310 x 79	1,450
HP 310 x 110	2,000
HP 310 x 132	2,400

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be a non-yielding material and the short pile length required, 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 compacted granular fill pad used for the installation of the abutment piles should consist of OPSS Granular A or Granular B Type II or Type III materials to allow installation of the piles without damage. In addition, two concentric CSPs that extend at least 3 m below of the abutments should be placed around the pile to create an annular space for integral abutments. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type 1. Refer to MTO Report SO-96-01 for further details.

The piles should have rock points (OPSD-3000.201 or SP 903S01) to minimize the potential for damage when setting on the bedrock.

A minimum of 2.0 m of soil cover of the equivalent thermal equivalent insulation should be provided for frost protection to the pile caps.

The rock section between the rock cut for the Southview Drive and the trench for the integral abutment piles will have to be partially removed and the upper section of the piles and CSPs laterally supported with a RSS wall. A high performance, high appearance rated RSS wall should be employed. The design, supply and construction of the RSS wall should conform to SP 599S22.

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 pile length providing resistance for integral abutment piles should be considered the dimension below the annular space. The assessed lateral resistance provided by the CHBDC for the pile sections noted previously is as follows:

Steel H-Pile, 310 x 79 **Steel H-Pile, 310 x 110** **Steel H-Pile, 310 x 132**	**GRANULAR BACKFILL**
Factored Lateral Resistance at ULS, kN	120
Lateral Resistance at SLS, kN	50

6.3 Approach Embankments

Boreholes were not carried out for the approach embankments to the Southview Drive Overpass Westbound. We anticipate, however that construction of the approach embankments will only be required for short sections, probably less than 20 m long behind each abutment, where the existing embankment was removed to facilitate the construction of the proposed overpass. The replacement of the fill behind the abutments should be straightforward. However, further subsurface investigations should be carried out at these locations for final design to verify the existing founding conditions of the embankment fill.

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 or rock fill, respectively, construction of 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 adjacent to the abutments will be about 8 m high. The embankments 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. The magnitude of the "consolidation" of these fills depends on the workmanship employed by the contractor and, if placed in 200 mm thick lifts compacted to 100% of standard Proctor maximum dry density in accordance with the requirements of SP 206S03 and OPSS 501 (Method A), the estimated settlements should be in the order of 20 mm. These estimated total settlements of the approach fill surface near the abutments should be essentially complete within 1 to 2 months after placement of the fill.

Remote from the abutments, where the embankments are constructed with rockfill placed in accordance with the same standards, estimated settlements are expected to be in the 40 mm range. About 50% of the settlements are expected to occur within the first 6 to 12 months after placement and the remaining during the following 5 to 10 years. The backfill may be changed to a compacted granular material to reduce the total settlements to about 20 mm.

6.4 Excavation Considerations

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 cohesionless fill materials, stiff clay or silt encountered in the boreholes are considered Type 3 soils. The rock is considered a Type 1 soil according to OHSA.

Excavation of the rock should follow the conventional methods of rock excavation such as blasting (OPSS 120) and jack hammering. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of cut and relative depth of excavation into the bedrock. This will be primarily dependent on detailed design and should be investigated further during detailed design investigation.

Groundwater was encountered north of existing Highway 17 and about 1.5 m above the bedrock footing founding level in borehole L5-1. It is noteworthy that groundwater levels are subject to seasonal fluctuations and rainfall patterns.

It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations for construction. More positive groundwater control measures using a combination of cut-off trenches and sumps may be warranted.

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
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**	**ROCK BACKFILL**
Internal Friction Angle, Ø (degrees)	35	42
Unit weight, γ (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 assigned geotechnical parameter values are the same for all granular materials in view of their similar physical characteristics.

Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed in the design of integral abutments. The coefficient of earth pressure at-rest should be used for the design of rigid and unyielding walls, and the active earth pressure coefficient for the design of unrestrained structures.

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

A subdrain system (SP 405F03) and/or weep holes (OPSD-3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the wall. The subdrain tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.

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

The recommended additional scope of the foundation investigation is as follows:

- Boreholes should be carried out at the centre (1), 2.5 m in front (2) and 2.5 m behind (2) the west and east abutments centreline and one additional borehole should be allowed at a strategic location within each foundation footprint to define the surface of the bedrock. At least three of the boreholes (at the corners and at the centre) of each foundation should be cored 3.0 m into the bedrock.
- Two boreholes should be carried out for each of the approach embankments.

8. DISCUSSION OF FOUNDATION ALTERNATIVES

8.1 Advantages and Disadvantages of Foundation Alternatives

The following table summarizes the advantages and disadvantages and inferred risks/consequences of each of the foundation alternatives for the proposed Southview Drive Overpass Westbound at the Highway 17 Sudbury Southwest Bypass.

Spread Footings Founded on Bedrock

Advantages

- Less costly than deep foundation alternative
- Conventional design and construction of foundations
- Allows semi-integral abutment design

Disadvantages

- Long-term maintenance costs of expansion joints for conventional abutment and deck design
- May require mass concrete or stepped footing to achieve a level founding subgrade on bedrock

Footings on Structural Fill

Advantages

- Reduced height of abutment
- Allows semi-integral abutment design

Disadvantages

- More costly than spread footings founded on bedrock
- Requires structural fill construction
- Construction of structural fill pad requires wider area than footings on bedrock

Driven Piles (Integral Abutment)

Advantages

- Allows integral abutment design and construction
- Lower long term maintenance cost expansion joints with integral abutment design

Disadvantages

- More costly than spread footings alternative
- Heavy equipment for pile driving is required
- Requires bedrock excavation to achieve minimum required pile length
- Requires false abutment RSS wall to support pile backfill

8.2 Preferred Foundation Option Considerations

From the foundation perspective, spread footings founded on bedrock or structural fill and driven pile foundations are considered feasible. The spread footing foundations for conventional or semi-integral abutments are considered to be the least costly alternative for construction. The semi-integral or integral abutments will have lower long-term maintenance and user costs.

Consequently, the most economical alternative in the long-term is the semi-integral abutment alternative on spread footings foundation for construction. This foundation scheme is considered to be the preferred foundation system from the geotechnical standpoint, subject to the results of the foundation investigation conducted during the detailed design.

It is noted that the selected foundation alternative also depends on other considerations, such as structural design and road grades, which are being evaluated separately by Stantec.

9. CLOSURE

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

Yours very truly,

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TABLE A

ROCK CORE DESCRIPTION

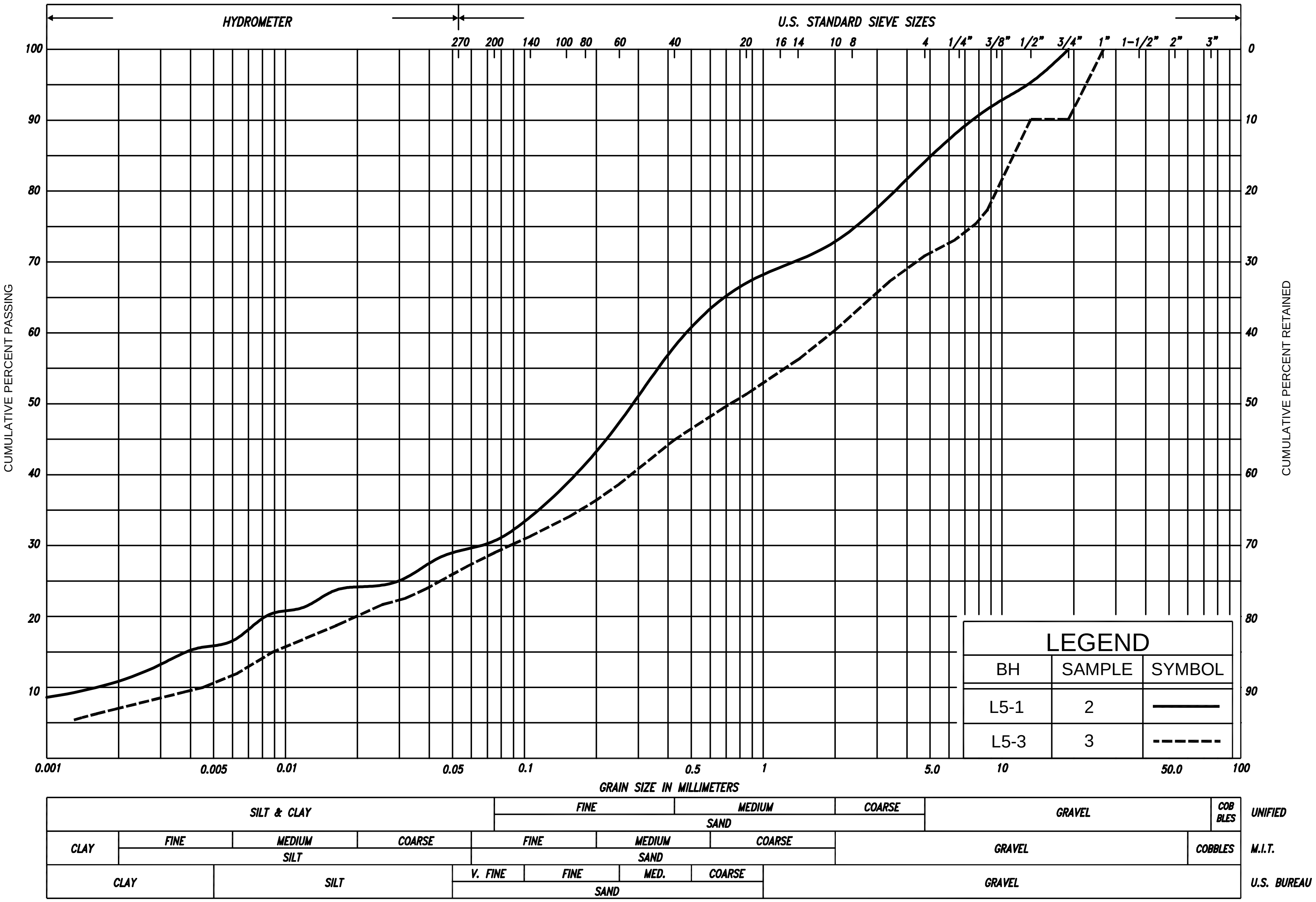
CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
L5-1	1 2	8.5 – 10.0 10.0 – 11.5	100 93	100 75	8.5 – 11.5	ARGILLITE: Grey to dark grey, fine crystalline, dark green to black on partings with thin metallic mineralization, high strength, unweathered, close to moderate spaced dipping partings, rough planar to slickensided undulating, tight, good to excellent quality.
L5-2	1 2 3	0.7 – 2.0 2.0 – 3.6 3.6 – 3.9	96 100 42	69 73 42	0.7 – 3.9	ARGILLITE: Medium to dark grey, fine crystalline, dipping to near vertical folliation, rust oxidation on partings with thin metallic mineralization, high to very high strength, slightly weathered to unweathered, very close to close spaced flat to dipping partings, smooth to rough planar, some vertical partings, slickensided planar, tight to oxidized, poor to fair quality.
L5-3	1 2 3	7.6 – 8.4 8.4 – 9.6 9.6 – 10.6	72 100 100	55 92 100	7.6 – 10.6	ARGILLITE: Medium to dark grey, fine crystalline, dark green to black parting surfaces with rust oxidation with thin metalic mineralization, high strength, unweathered, close spaced flat to dipping partings, rough planar, some slickensided planar, tight to oxidized, fair to excellent quality.
L5-4	1 2 3	0.1 – 1.4 1.4 – 2.8 2.8 – 3.5	90 98 53	85 72 53	0.1 – 3.5	ARGILLITE: Light to dark grey or black, fine crystalline, (arkose like), high strength, unweathered, close spaced dipping partings, rough planar, some slickensided planar, tight, fair to good quality.

Originated:	FP
Compiled:	PML
Checked:	CN

TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 120	General Specification for the Use of Explosives
OPSS 501	Construction Specification for Compacting
OPSS 571	Construction Specification for Sodding
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
SP 599S22	Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)
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-3000.201	Oslo Points for Foundation, Piles, Steel HP310
OPSD-3090.100	Foundation Frost Depth for Northern Ontario
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail



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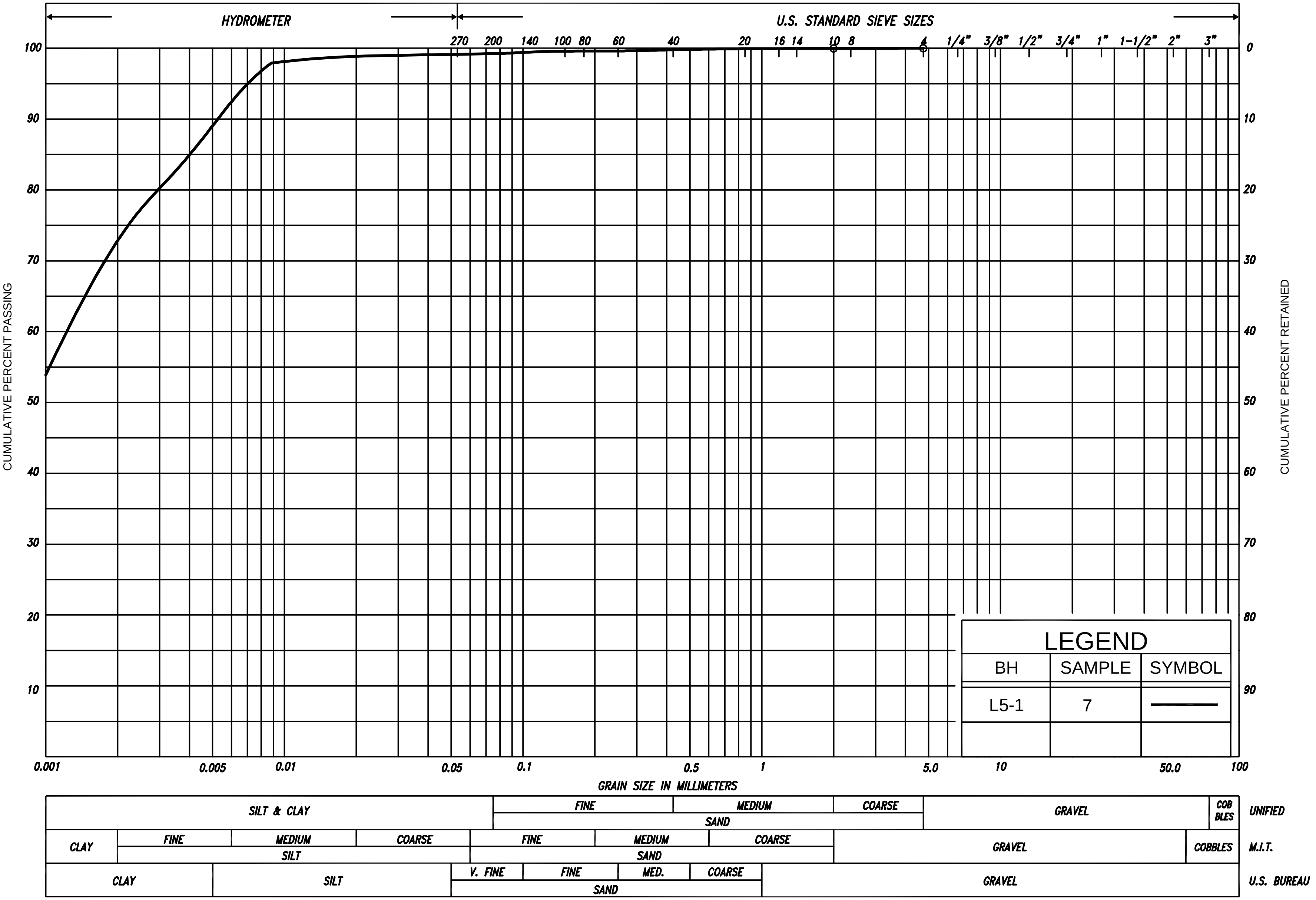
GRAIN SIZE DISTRIBUTION

SAND trace to some clay some silt, some to with gravel
(FILL)

FIG No.	GS-L5-1
HWY	17
G.W.P. No.	5825-05-00



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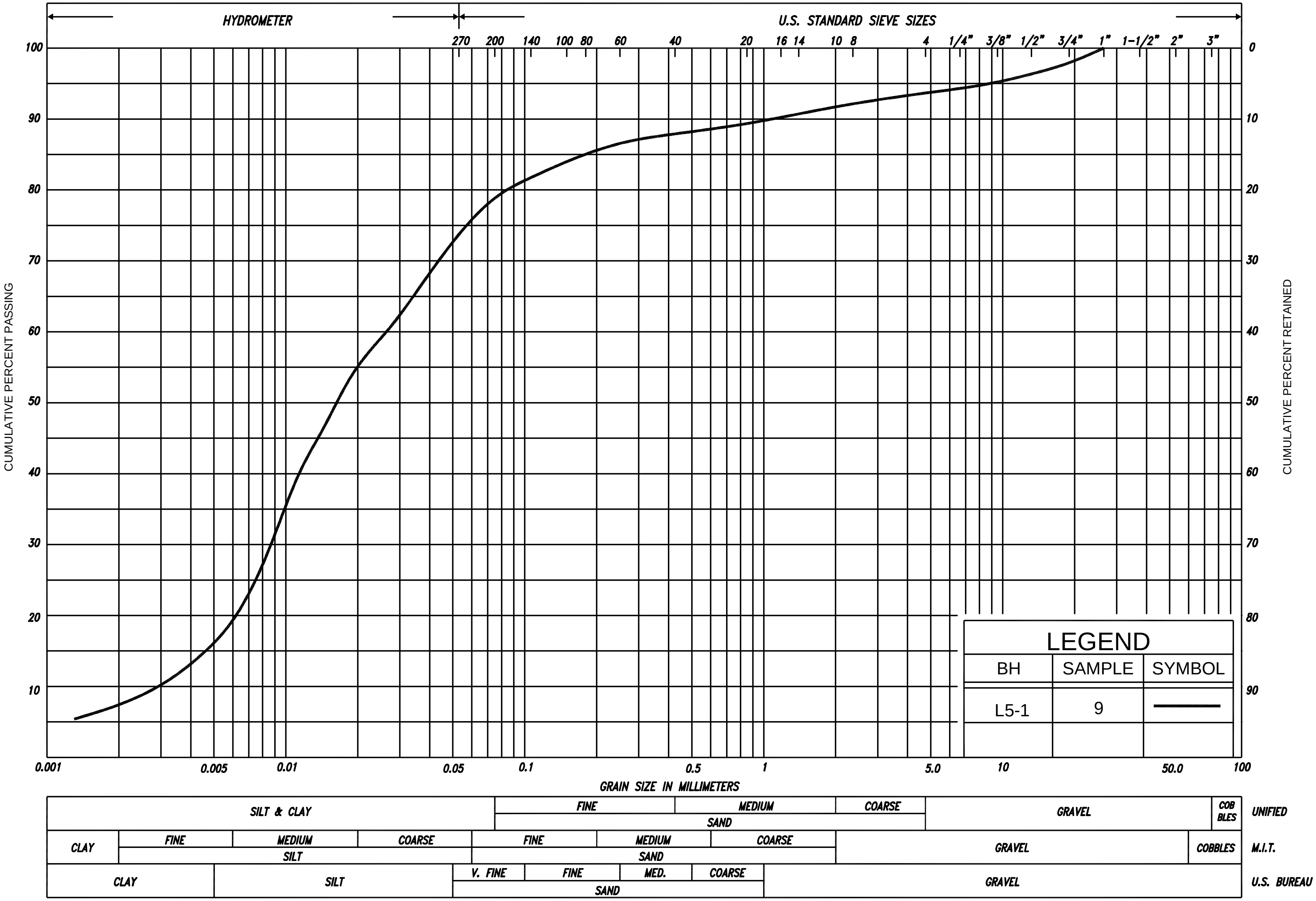
GRAIN SIZE DISTRIBUTION

CLAY, trace sand

FIG No.	GS-L5-2
HWY	17
G.W.P. No.	5825-05-00



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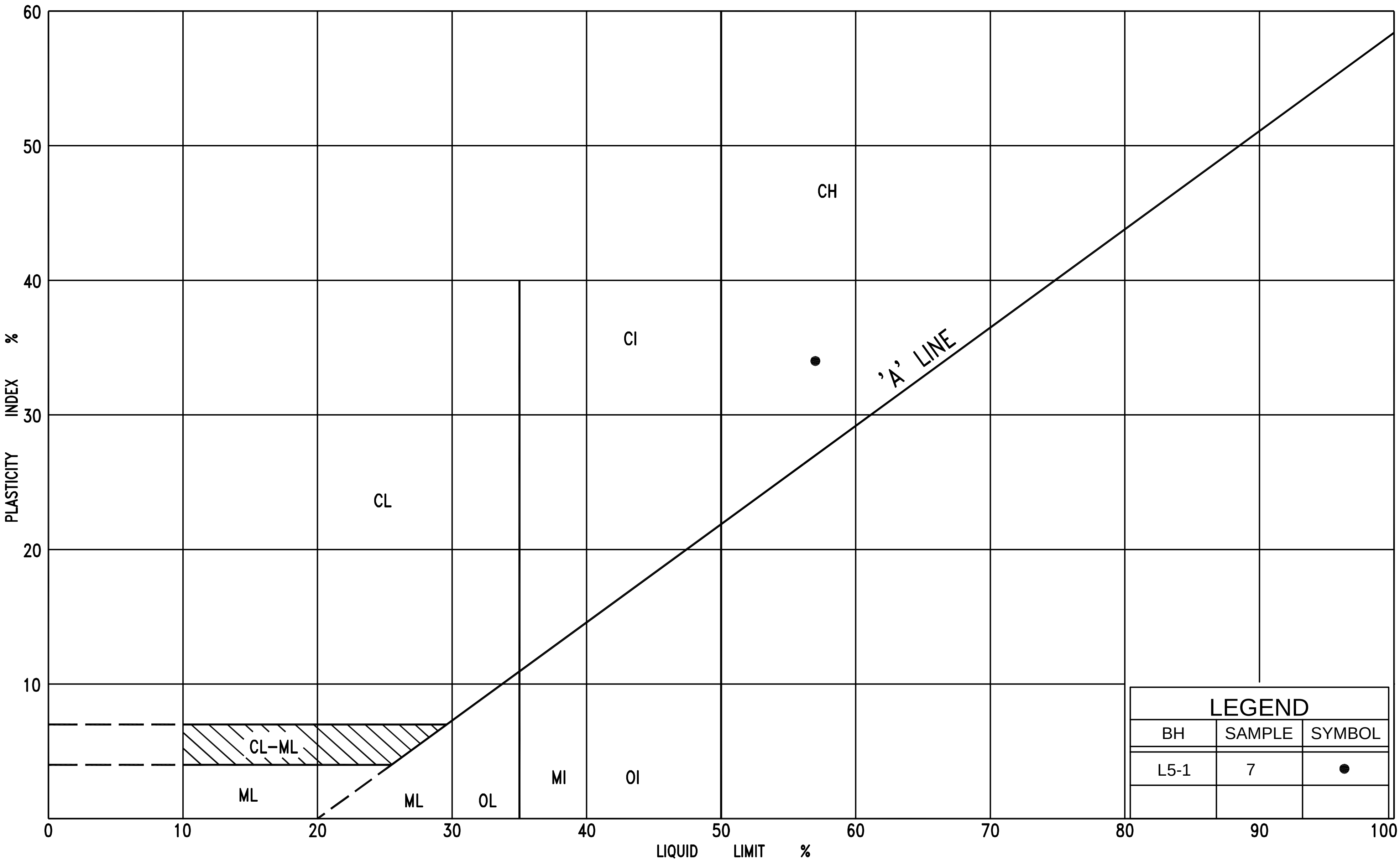
GRAIN SIZE DISTRIBUTION

SILT, some sand, trace clay, trace gravel

FIG No.	GS-L5-3
HWY	17
G.W.P. No.	5825-05-00



LEGEND

BH	SAMPLE	SYMBOL
L5-1	7	●

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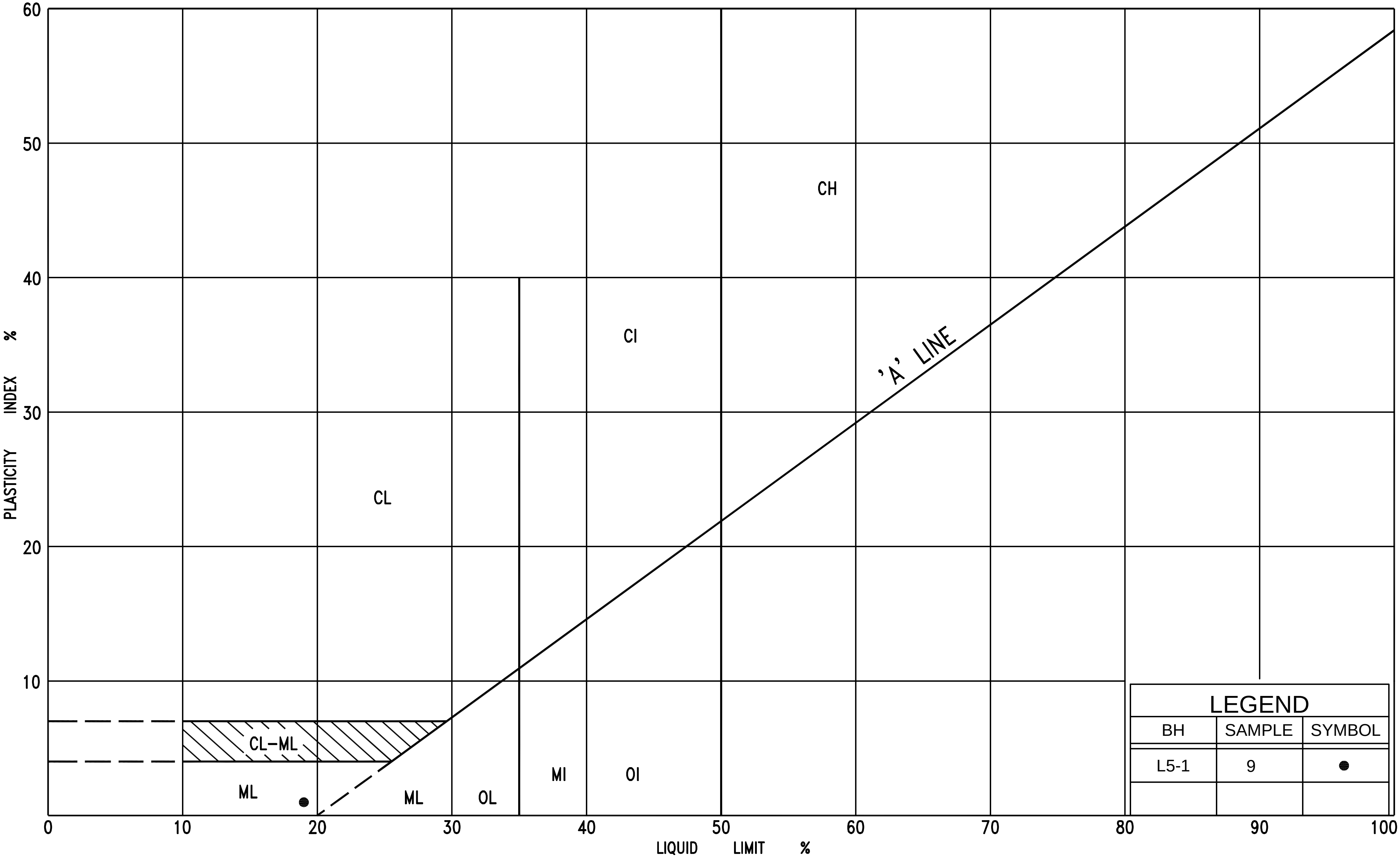
PLASTICITY CHART

CLAY, trace sand

FIG No.	PC-L5-1
HWY	17
G.W.P. No.	5825-05-00



LEGEND

BH	SAMPLE	SYMBOL
L5-1	9	●

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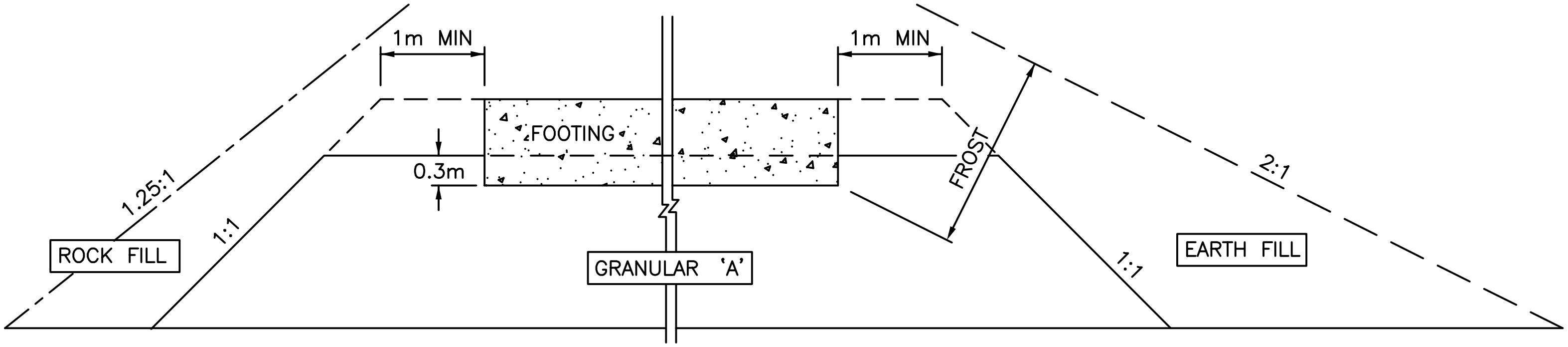
PLASTICITY CHART

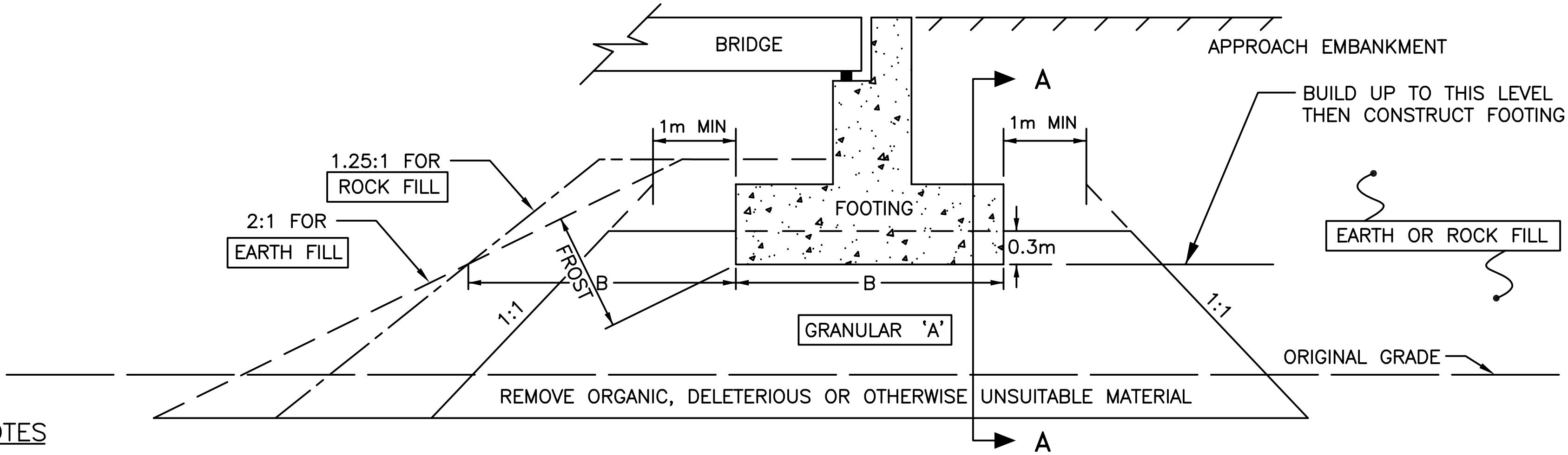
SILT, some sand, trace clay, trace gravel

FIG No.	PC-L5-2
HWY	17
G.W.P. No.	5825-05-00



CROSS SECTION A–A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

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

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Ontario

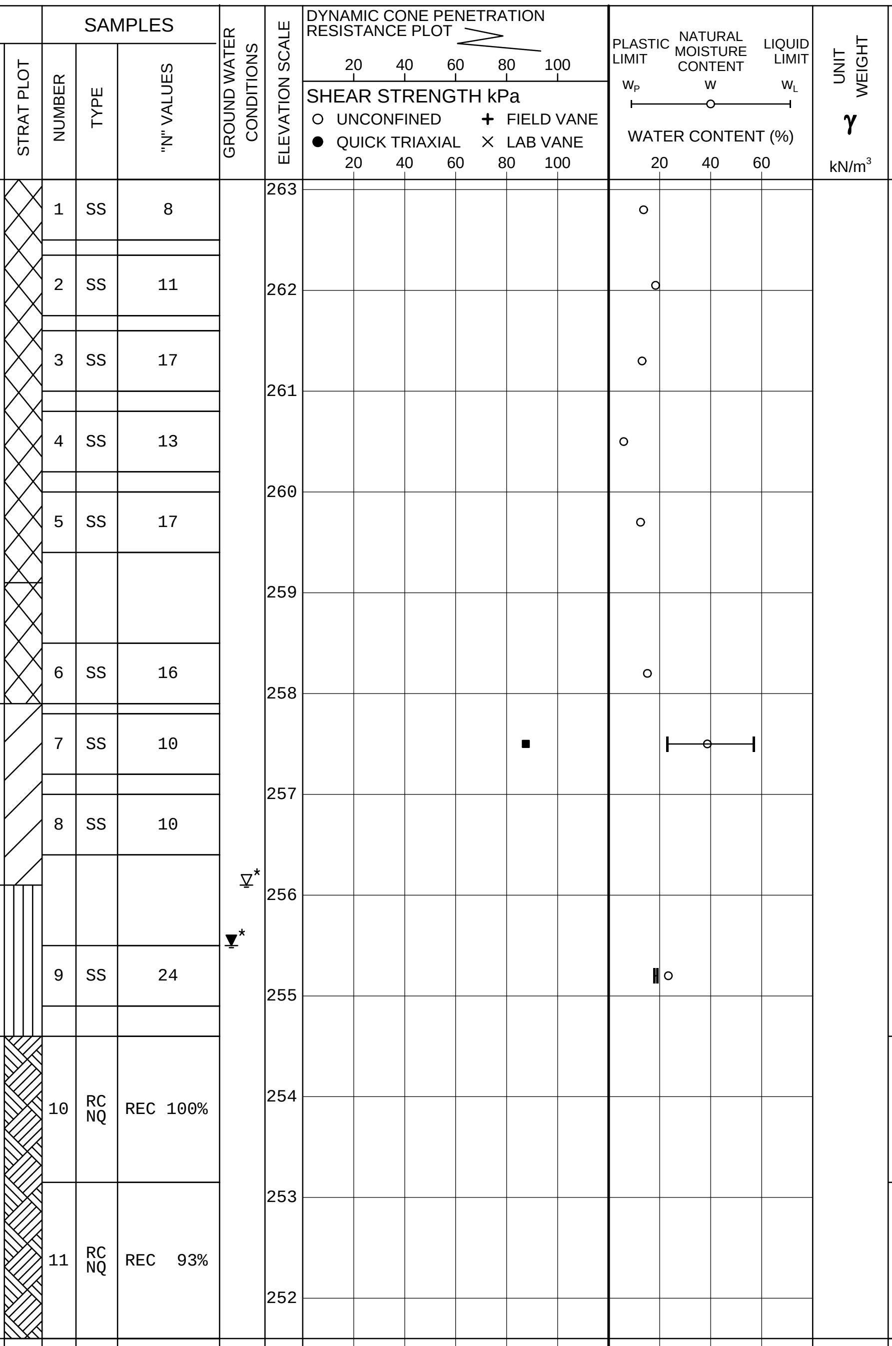
Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No L5-1

1 of 1 **METRIC**

G.W.P. 5825-05-00
LOCATION Co-ords: 5 144 837 N; 300 661 E
ORIGINATED BY F.P.
DIST 54 HWY 17
BOREHOLE TYPE Continuous Flight Hollow Stem Augers+ NW Casing
COMPILED BY N.S.B
DATUM Geodetic
DATE April 30, 2007
CHECKED BY NR/CN

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
263.1	Ground Surface					
0.0	Sand some gravel, some silt silty clay pockets Loose to compact Brown Moist	1	SS	8		
		2	SS	11		16 54 19 11
		3	SS	17		
		4	SS	13		
		5	SS	17		
	silty sand and clayey silt pockets (FILL)	6	SS	16		
257.9 / 5.2	Clay, trace sand Stiff Brown Wet	7	SS	10		0 1 26 73
		8	SS	10		
256.1 / 7.0	Silt, some sand trace clay, trace gravel Compact Brown Wet				∇* ▼*	
		9	SS	24		6 16 71 7
254.6 / 8.5	Bedrock Argillite Grey to dark grey High strength Unweathered Good to excellent quality	10	RC NQ	REC 100%		RQD 100%
		11	RC NQ	REC 93%		RQD 75%
251.6 / 11.5	End of borehole					



* 2007 04 30

∇ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

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

20, 15, 5, 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No L5-2   1 of 1   METRIC

G.W.P. 5825-05-00   LOCATION Co-ords: 5 144 790 N; 300 640 E   ORIGINATED BY F.P.
DIST 54   HWY 17   BOREHOLE TYPE Solid Stem Augers + Rotary Diamond Drilling   COMPILED BY N.S.B
DATUM Geodetic   DATE April 23, 2007   CHECKED BY NR/CN

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	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
263.0	Ground Surface										
0.0	Gravelly sand cobbles Loose Brown Moist (ROCK FILL)		1	SS	25/13cm						
262.3 0.7	Bedrock Argillite Medium to dark grey High to very high strength Slightly weathered to unweathered Poor to fair quality		2	RC NQ	REC 96%		262 261				RQD 69%
			3	RC NQ	REC 100%		260				RQD 73%
259.1			4	RC NQ	REC 42%						RQD 42%
3.9	End of borehole Sample 1: sampler bouncing * Borehole dry										

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

20
15—○—5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No L5-3

1 of 1

METRIC

G.W.P. 5825-05-00	LOCATION Co-ords: 5 144 829 N; 300 685 E	ORIGINATED BY F.P.
DIST 54 HWY 17	BOREHOLE TYPE Continuous Flight Hollow Stem Augers+ NW Casing	COMPILED BY N.S.B
DATUM Geodetic	DATE May 01, 2007	CHECKED BY NR/CN

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
266.2	Ground Surface									
0.0	Sand and gravel with pockets of asphalt Compact Brown Wet	1	AS	-		266				
		2	SS	12		265				
	pockets of clayey silt (FILL)	3	SS	15/8cm			29	44	21	6
	Cobbles and boulders (ROCK FILL)					264 263 262				
		4	SS	20/3cm						
261.0 5.2	Silt trace sand, trace gravel Compact Brown Moist					261				
		5	SS	28		260 259				
258.6 7.6	Bedrock Argillite Medium to dark grey High strength Unweathered Fair to excellent quality	6	SS	25/5cm						
		7	RC NQ	REC 72%		258	RQD 55%			
		8	RC NQ	REC 100%		257	RQD 92%			
		9	RC NQ	REC 100%		256	RQD 100%			
255.6 10.6	End of borehole Samples 3, 4 and 6: sampler bouncing									

* Borehole charged with drilling water

** Rotary drilling carried out through rock fill from 1.5 to 5.2m depth

+7, ×5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No L5-4

1 of 1

METRIC

G.W.P. 5825-05-00
LOCATION Co-ords: 5 144 790 N; 300 668 E
ORIGINATED BY F.P.
DIST 54 HWY 17
BOREHOLE TYPE Rotary Diamond Drilling
COMPILED BY N.S.B
DATUM Geodetic
DATE April 24, 2007
CHECKED BY NR/CN

SOIL PROFILE			SAMPLES					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
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE				
265.8	Ground Surface										
0.0 0.1	Cobbles (ROCK FILL)										
	Bedrock Argillite Light to dark grey High strength Unweathered Fair to good quality		1	RC NQ	REC 90%		265				RQD 85%
			2	RC NQ	REC 98%		264				RQD 72%
			3	RC NQ	REC 53%		263				RQD 53%
262.3 3.5	End of borehole										
	* Borehole dry										

+^{7}, ×^{5}: Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

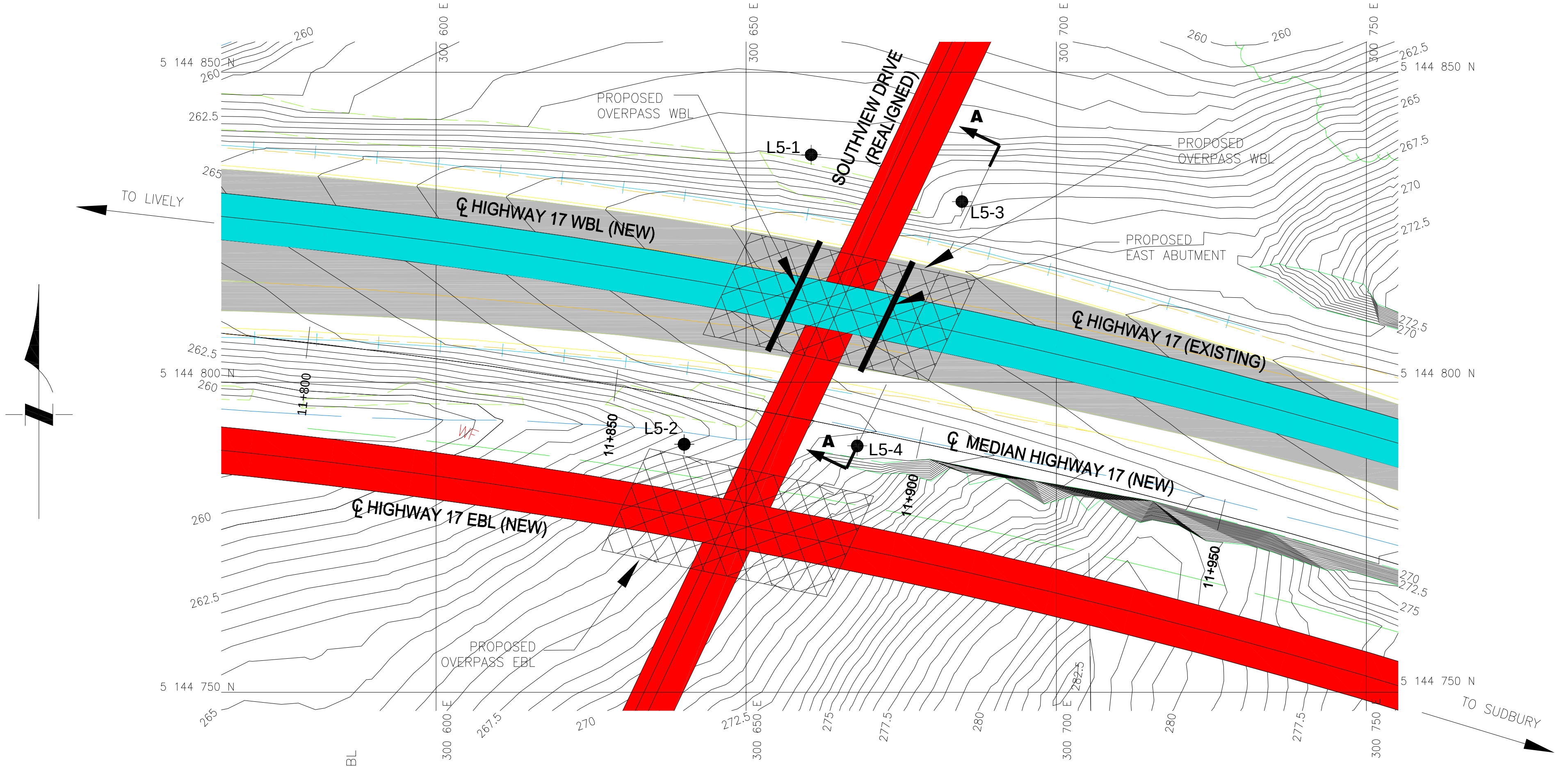
METRIC

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

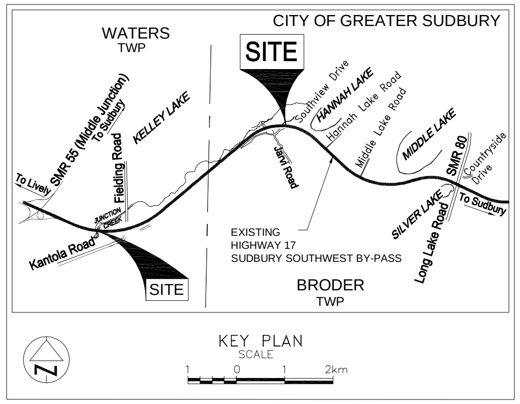
CONT No
GWP No 5825-05-00
SOUTHVIEW DRIVE OVERPASS
WESTBOUND
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN
SCALE
10 0 10 20m

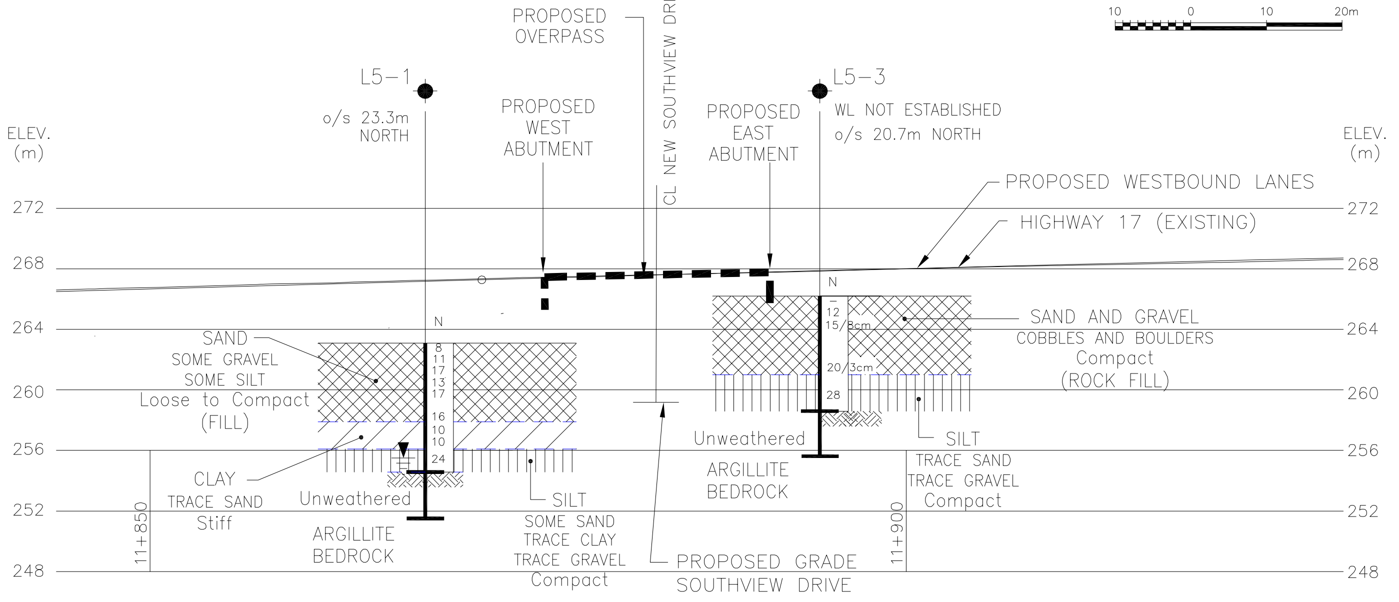


KEY PLAN
SCALE
1 0 1 2km

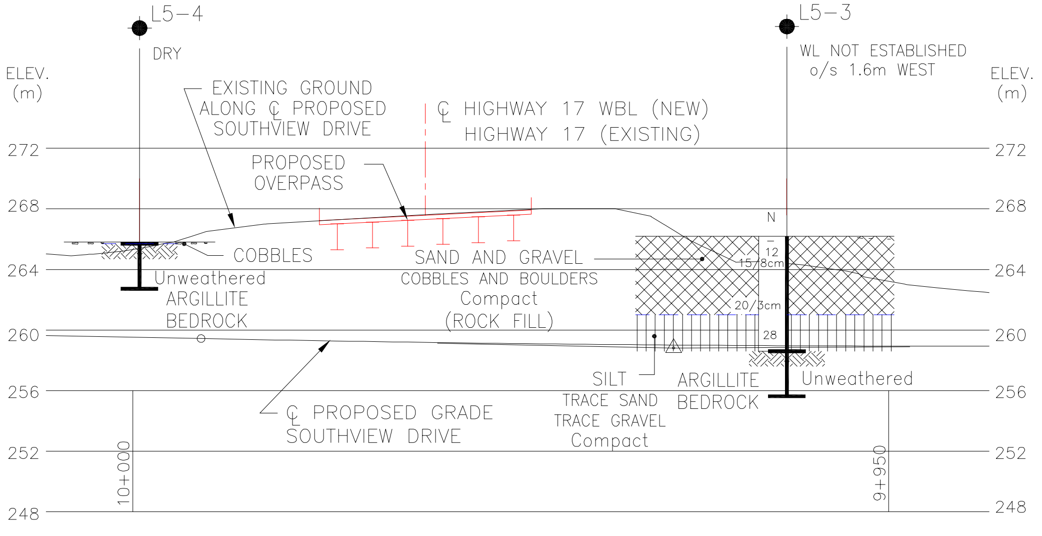
LEGEND

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

BH No	ELEVATION	COORDINATES	
		NORTHINGS	EASTINGS
L5–1	263.1	5 144 837	300 661
L5–2	263.0	5 144 790	300 640
L5–3	266.2	5 144 829	300 685
L5–4	265.8	5 144 790	300 668



℄ PROFILE
SOUTHVIEW DRIVE OVERPASS WESTBOUND



SECTION A–A

SCALE
5 0 5 10m

NOTES:

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
2. PRELIMINARY LOCATIONS OF PROPOSED ABUTMENTS WERE ESTIMATED FOR DISCUSSION PURPOSES IN THIS REPORT ONLY.
3. LOCATION OF STRUCTURE WAS CHANGED AFTER THE FIELD WORK WAS COMPLETED.

C M P Nascimento
07/08/08

B. R. GRAY
July 8 08

REF No. Stantec Drawings;599_Alternate 1.dwg; 599_base.dwg; 599_Contours.dwg; Received on 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–225

HWY No 17			DIST 54
SUBM'D MNR	CHECKED CN	DATE JULY 08, 2008	SITE 46–518/2
DRAWN NA	CHECKED CN	APPROVED BRG	DWG SDW–1

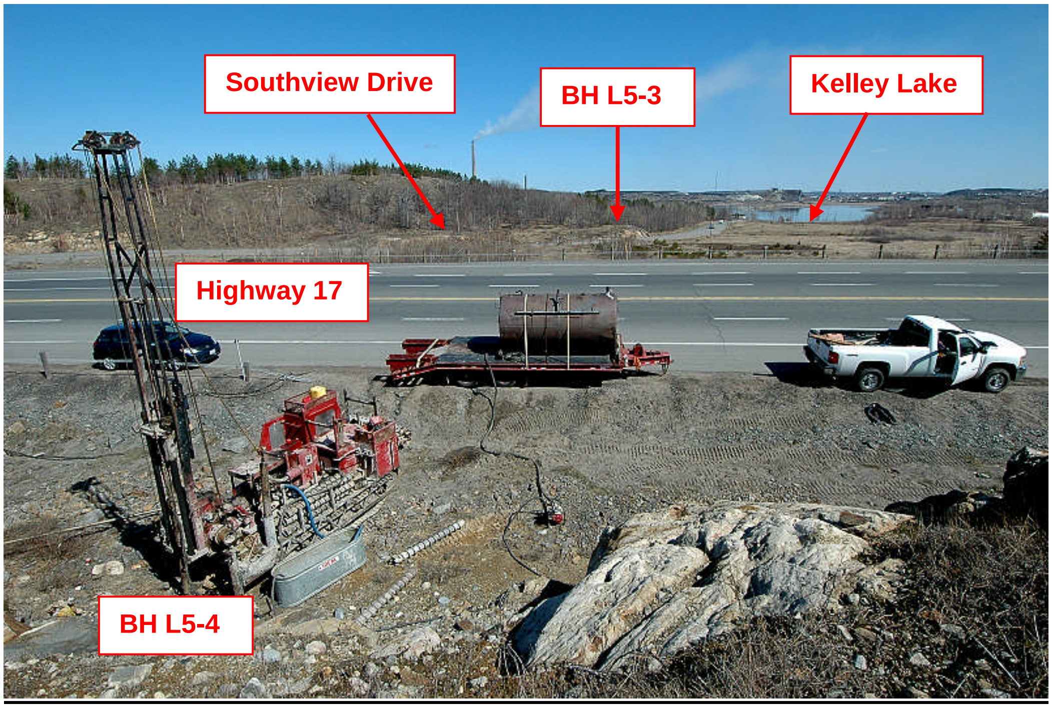
APPENDIX A

Site and Rock Core Photographs

Photograph 1: Looking west from toe of embankment north of Highway 17 at about Sta. 11+882, borehole L5-1, 170 m east of Southview Drive and Highway 17 intersection. About 6 m high rockfill embankment of the existing Highway 17 is in view. Drillers are rock coring in borehole L5-1. (April 30, 2007)

Photograph 2: Looking east from the shoulder of Highway 17, Drill rig is at about Sta. 11+906, on borehole L5-3. Rock cut of about 8 m high in view at right side of the photograph. (May 1, 2007)

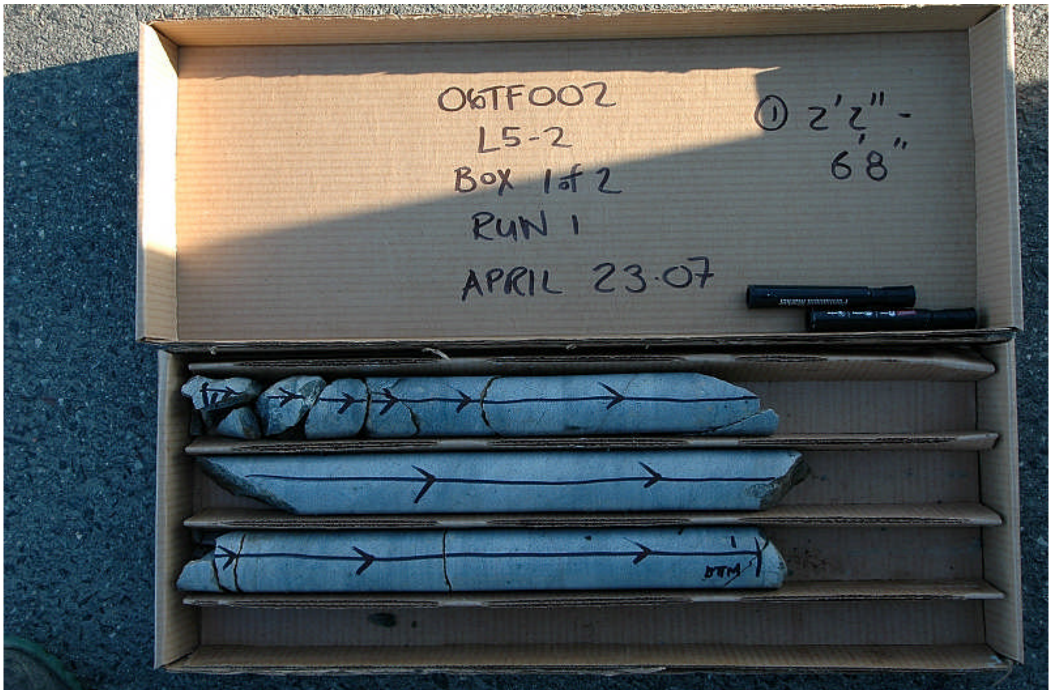
Photograph 3: Looking north from north side of Highway 17 at drill set-up on borehole L5-3, about Sta. 11+906. Low-lying swamp area covered with grass, brush and alders with a bedrock outcrop at Southview Drive in distance. (May 1, 2007)



Photograph 4: Looking north from south of Highway 17 at about Sta. 11+890, borehole L5-4. Bedrock outcrop visible in the foreground. (April 14, 2007)

Photograph 5: Rock cores from L5-1, Run 1.

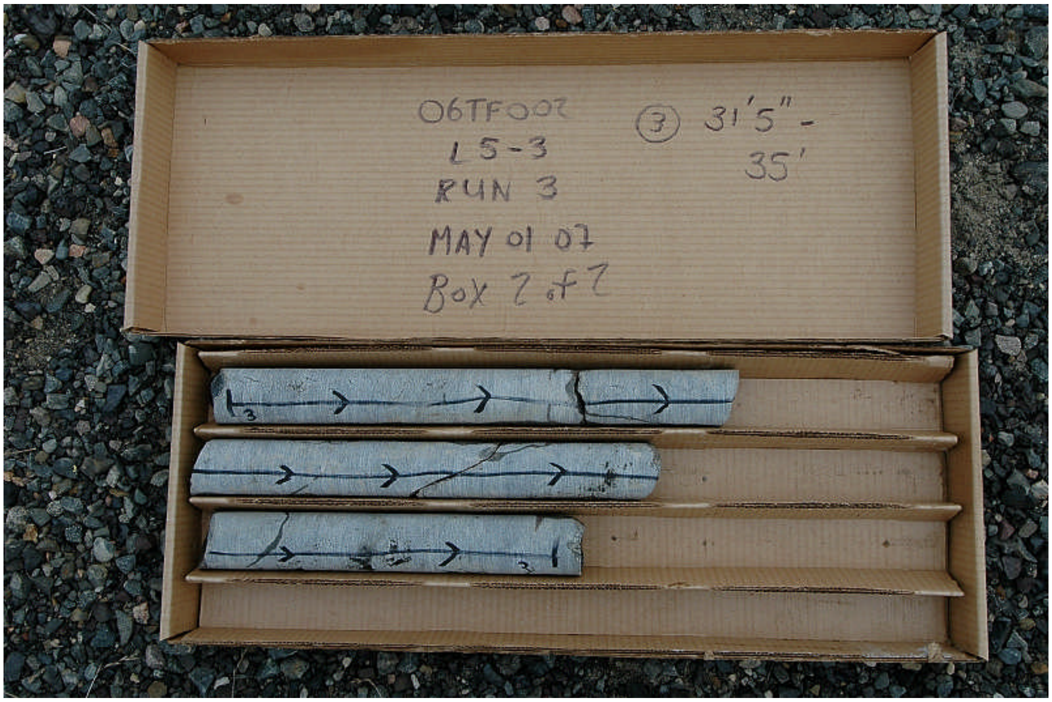
Photograph 6: Rock cores from L5-1, Run 2.



Photograph 7: Rock cores from L5-2, Run 1.

Photograph 8: Rock cores from L5-2, Runs 2 and 3.

Photograph 9: Rock cores from L5-3, Runs 1 and 2.



Photograph 10: Rock cores from L5-3, Run 3.

Photograph 11: Rock cores from L5-4, Run 1.

Photograph 12: Rock cores from L5-4, Runs 2 and 3.