

FOUNDATION INVESTIGATION
AND DESIGN REPORT

PROPOSED NEW STRUCTURAL CULVERT
AT PEEL STREET NORTH, THORNBURY
TOWNSHIP OF COLLINGWOOD
HIGHWAY 26 FROM MEAFORD TO THORNBURY

G.W.P. 57-00-00
Agreement # 3006-E-0002



I.E.
Group



Infrastructure Engineering Group Inc.
Pavement & Construction Materials Consulting Engineers
GTA • Kitchener • London • Windsor

Corporate Office
39-69 Bessemer Road
London, Ontario
N6E 2V6
tel: (519) 680-9991
fax: (519) 680-9993
email: info@lawengineering.com

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Prepared for:

Stantec Consulting Ltd.
1400 Rymal Road East
Hamilton ON
L8W 3N9

Mr. Adam Barg, P.Eng.

Prepared by:

Infrastructure Engineering Group Inc.
39-69 Bessemer Road
London, Ontario
N6E 2V6

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PART A – FOUNDATION INVESTIGATION

1.0 INTRODUCTION

This report presents the results of a foundation investigation carried out on July 9, 2009 by Infrastructure Engineering Group Inc. (IEG) on behalf of Stantec Consulting Ltd. (Stantec).

This assignment involves the rehabilitation of the pavement structure on Highway 26 from 0.2 km east of the Thornbury west limit (Peel Street) westerly 10.06 km to the Town of Meaford east limit.

It includes the rehabilitation and extension of two existing structural culverts, as well as many non-structural culvert extensions and replacements. The project also includes intersection realignments, intersection improvements, construction of two new 1.5 km long passing lanes, minor horizontal and vertical alignment improvements and electrical work. The original assignment included the re-alignment of the Blue Mountains/Meaford Town Line which has been deleted from the assignment.

Foundation investigation and recommendations are required for the design and construction of culvert replacements and extension as part of the improvement of Highway 26. Two (2) structural culverts, twenty-four (24) non-structural culverts, two shale bin replacements, and a high cut area are to be investigated. There is a change in the scope of work to include two additional culvert extensions which were not part of the original scope of work for foundation investigations, and re-allocation of the foundations investigation work for three (3) CSP culverts to the geotechnical investigation portion of this assignment. A new structural culvert located at Peel Street North in Thornbury is also added to the scope of work.

This report covers the site of the new structural culvert at Peel Street North in Thornbury, in the Collingwood Township.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes and, based on the findings, to provide geotechnical recommendations for the foundation elements. Armour Stone or gabion wing walls may be required at the inlet and outlet of the new structure.

Authorization to complete this assignment was given by Mr. Adam Barg, P. Eng., of Stantec Consulting Ltd., the TPM Consultant who is completing this assignment for MTO under Agreement # 3006-E-0002.

2.0 SITE DESCRIPTION

2.1 Site Location

It is proposed to construct a new structural culvert in conjunction with the Peel Street North realignment north of Highway 26. Based on information supplied by Stantec, the new culvert will consist of a 4 m span by 1.3 m high by 29 m in length. The location of the proposed culvert is approximately 16 m north of the centerline of Highway 26, between STA 14+035.5 and 14+064.5. Photographs of this culvert site are presented in Appendix "D".

The culvert site is located within a drainage ditch in which the ditch flows easterly. The approach embankments will be constructed on both the north and south sides of the culvert, with a maximum height of approximately 2.0 m. The embankment slopes will be typically 2.5H to 1V.

The proposed new culvert is located within a swampy area between STA 13+900 and 14+200 and spanning from the north-west towards the south-east at the Highway 26 and the Peel Street North intersection. The ground surface at the location of the proposed culvert was covered with bulrushes and reasonably dry, but partly inundated at the existing ditch located north of the proposed culvert with water level of less than 0.5 m above the ground surface at the time of field work.

Details of the proposed culvert are not known at the time of preparing this draft report. It is understood that the proposed culvert will consist of a precast concrete box culvert.

2.2 Physiography and Topography

The Town of Meaford is situated at the mouth of the Bighead River where the river enters Nottawasaga Bay, part of the Georgian Bay of Lake Huron. The subsurface of the Town of Meaford is comprised of predominately silty clay, and smooth to gently sloping topography. Pockets of sand and gravelly sands exist which also exhibit smooth to gently sloping topography. The Town is located on the coastal plain left by glacial Lake Algonquin. East of Meaford, the Algonquin shore cliff coincides with the base of the Niagara Escarpment. The coastal plain in this area consists of sand and gravel beach terraces overlying the bedrock. Overburden thickness is generally less than 5 m.

Bedrock consists of the shale and limestones of the Georgian Bay Formation. Grey, impure carbonate beds (limestone and dolomite) alternate with grey and blue/grey shale.

West of Meaford, the coastal plain consists of the same beach deposits as found in the east. To the west away from the Lake, overburden becomes a glacio-lacustrine derived silt to clayey till. Numerous drumlins of calcareous till with red shale inclusions are found in the Meaford area.

Progressing west on Highway 26 toward Owen Sound and the Niagara Escarpment, the bedrock types progress from Queenston shales, the Clinton and Cataract shales and dolomites to the cap

rock of the Amabel dolomites and limestones. Overburden thickness can be as much as 15 m, but is generally less than 5 m.

The existing ground surface Elevations range from 193.3 m to 193.9 m, with a proposed finish road grade of Elevation 195.25 m. The proposed invert of the culvert will be placed at Elevation 193.0±m.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

Three (3) boreholes were drilled and sampled to obtain data for foundation design of the proposed new structural culvert. The locations of the boreholes are shown on Drawing 1.

A Diedrich D-50 Bombardier-mounted drill rig was supplied by Walker Drilling Ltd. on July 9, 2009, for drilling and Standard Penetration Testing (SPT), following the procedures of ASTM D 1586). The boreholes were drilled using continuous flight solid and hollow stem augers. Soil samples were retrieved at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT). Samples were generally taken at intervals of depth of 0.75 m to the maximum depth of exploration.

Field pocket penetrometer was used on the retrieved SPT samples, where applicable, to determine the undrained shear strength of the cohesive soil deposits. These undrained shear strengths are used to supplement the properties of the cohesive soils. It is noted that the measured shear strength value would be slightly lower than the actual value due to sampling disturbance.

The soil samples obtained were placed in labeled containers and transported to IEG's London laboratory for further examination and laboratory testing.

The locations of the boreholes are shown on Drawing 1 and the depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
31A-1	5.72
31A-2	5.77
31A-3	3.35

Seepage and water levels were noted in each borehole during and at the completion of drilling and sampling. All boreholes were grouted with a bentonite/cement mix at completion of sampling in accordance with Ontario Regulation 903.

Our field engineer, Mr. Joseph Law, P. Eng., supervised the fieldwork and worked under the direction of the project engineer, Mr. Eric Chung, P. Eng. Our field staff cleared the location of buried utilities and logged the boreholes.

The stations, offsets and ground surface elevations at the as drilled borehole locations were surveyed by Stantec and provided to IEG for the purpose of this report.

The results of the drilling, sampling, in-situ testing and groundwater observations are summarized on the Record of Borehole sheets and enclosed in Appendix "A".

3.2 Laboratory Analysis

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all retrieved soil samples. In addition, grain size analyses and Atterberg Limit tests were performed on selected soil samples.

The results of the laboratory testing are presented on the Record of Borehole sheets (Appendix "A"), and Laboratory Test Results (Figures 1 to 6, Appendix "B").

4.0 SUBSURFACE CONDITIONS

4.1 General Subsurface Conditions

Reference is made to the Record of Borehole sheets (Appendix "A") and Laboratory Test Results (Appendix "B") for detailed subsurface soil and groundwater conditions encountered in the boreholes. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, consequently, represent transitions between soil types rather than exact planes of geological change. The soil profiles depicting the subsurface conditions on Drawing 1 will vary between and beyond the borehole locations.

The subsurface deposits at the site consist of a 0.15 m to 0.53 m thick layer of topsoil and/or organic deposits, underlain by a 0.76 m to 2.14 m thick layer of very soft to stiff silty clay. The silty clay layer generally becomes stiff at depths of between 0.5 m and 1.5 m below the existing ground surface (between Elevations 192.2 and 193.0 m). The silty clay is in-turn underlain by a 0.6 m and 0.9 m thick layer of very stiff layered silt and silty clay. The layered silt and silty clay is subsequently underlain by a sand and silt till which extends beyond the vertical extents of the boreholes at depth of between 3.35 m and 5.77 m (Elevations 190.53 m and 187.91 m).

Drilling advance was difficult at the locations of Borehole 31A-1 and 31A-3 and encountered cobbles and/or boulders within the sand and silt till which caused auger and sampler refusal. Borehole 31A-3 was terminated after 3 unsuccessful attempts to drill below a depth of 3.35 m.

4.1.1 Topsoil, Organic Silt and Fill

All three boreholes encountered topsoil of between 150 mm and 300 mm thick. The topsoil in Borehole 31A-1 is underlain by a 0.23 m thick layer of organic silt. The topsoil in Borehole 31A-3 is underlain by a 0.61 m thick layer of mixed fill consisting of silty clay and organics.

Standard penetration tests yielded “N”-values generally ranging from 1 to 2 blows per 0.3 m.

The fill is dark brown to brown in color, and the organic silt is greenish grey in color. Moisture content of the fill and organic materials measured natural moisture contents ranging from 25 to 94%. Based on the above field and laboratory test results, together and tactile examination, the organic soils and fill materials exhibited very loose compactness condition.

Unit weight of the fill was not determined due to poor sample recovery and excessively disturbed samples.

4.1.2 Silty Clay

A stratum of brown with grey mottling silty clay, with a trace to some sand is present below the fill and organic materials at all three borehole locations. The silty clay is generally interbedded with frequent silt pockets, seams and layers.

Standard penetration tests yielded “N”-values ranging from 1 to 14 blows per 0.3 m. “N”-values of less than 8 blows per 0.3 m were generally encountered to depths of between 0.6 m and 1.5 m below the existing ground surface (between Elevations 192.2 m and 193.0 m).

Four (4) grain size distribution analyses were performed on the silty clay deposit and the results are presented on Figure 1 of Appendix “B”.

Four (4) samples were tested and exhibited the following Atterberg Limits. These results are shown in Figure 2 of Appendix “B” and summarized below:

Atterberg Limits	Minimum	Maximum	Average
Liquid Limit (W_L), %	26	38	31.3
Plastic Limit (W_P), %	17	22	19.0
Plasticity Index (I_p), %	9	16	12.3

The natural moisture contents of the samples were between 17 and 37%. These results are characteristic of clayey soils of low to medium plasticity (CL-CI). The measured natural moisture contents are generally between the plastic and liquid limits and indicate that the deposit is slightly pre-consolidated.

Pocket penetrometer readings taken on the SPT samples yielded correlated unconfined shear strength of between 20 and 250 kPa.

Based on the above field and laboratory test results, together with visual and tactile examination, the silty clay deposit exhibited generally firm to stiff consistency, with near surface soft to very soft zones, and hard zones at depth.

Unit weight of the silty clay was determined on three (3) samples to be between 23.8 and 25.6 kN/m³.

4.1.3 Layered Silt and Silty Clay

A stratum of grey and reddish brown layered silt and silty clay, with a trace to some sand, and occasional fine gravel is present below the silty clay layer at all of the borehole locations.

Standard penetration tests yielded “N”-values ranging from 16 to 19 blows per 0.3 m.

Three (3) grain size distribution analyses were performed on the layered silt and silty clay deposit and the results are presented on Figure 3 of Appendix “B”.

Three (3) samples were tested and exhibited the following Atterberg Limits. These results are shown in Figure 4 of Appendix “B” and summarized below:

Atterberg Limits	Minimum	Maximum	Average
Liquid Limit (W _L), %	22	25	23.3
Plastic Limit (W _P), %	16	18	16.7
Plasticity Index (I _p), %	6	7	6.7

The natural moisture contents of the samples were between 14 and 18%. These results are characteristic of a clay soil of low plasticity (CM-CL). The measured natural moisture contents are generally at or less than the plastic limit and indicate that the deposit is pre-consolidated.

Pocket penetrometer readings taken on the SPT samples yielded unconfined shear strength of between 120 and 175 kPa.

Based on the above field and laboratory test results, together with visual and tactile examination, the layered silt and silty clay deposit exhibited generally very stiff consistency.

A single unit weight of the layered silt and silty clay was determined to be 22.7 kN/m^3 .

4.1.4 Sand and Silt Till

The layered silt and silty clay layer is in-turn underlain by a grey sand and silt till. Cobbles and boulders are present within the sand and silt till, with localized pockets of saturated sand and gravel. Slight plasticity was observed with samples taken from Boreholes 31A-1 and 31A-2.

Standard penetration tests yielded “N”-values ranging from 49 to over 100 blows per 0.3 m.

Three (3) grain size distribution analyses were performed on the silty clay deposit and the results are presented on Figure 5 of Appendix “B”.

Three (3) samples were tested and exhibited the following Atterberg Limits. These results are shown in Figure 6 of Appendix “B” and summarized below:

Atterberg Limits	Minimum	Maximum	Average
Liquid Limit (W_L), %	14	20	17.0
Plastic Limit (W_P), %	12	12	12.0
Plasticity Index (I_P), %	2	8	5.0

The natural moisture contents of the samples were between 6 and 22%. These results are characteristic of sand and silt till (ML-SM).

Based on the above field and laboratory test results, together with visual and tactile examination, the sand and silt till has a dense to very dense compactness condition.

A single unit weight of the varved clay was determined to be 25.0 kN/m^3 .

4.2 Groundwater Conditions

The groundwater condition was monitored during and upon completion of sampling. There was less than 0.5 m of water ponding on the existing ditch at the time of our field work on July 9, 2009, and assumed to be of low to moderate flow condition due to rainfall events just prior to our field work.

On completion of drilling, free groundwater was observed in the boreholes and the respective depth and elevation are summarized below:

Borehole	Water Level in Borehole, m (Elevation, m)	Remarks
31A-1	5.03 (188.66)	
31A-2	1.98 (191.70)	
31A-3	1.20 m (192.68)	Dry on second and third attempt for the short duration the boreholes were kept open.

Based on the natural moisture content of the soils samples, the groundwater encountered likely reflects water perched within the upper fill and organic materials and the near surface silty clay.

It should be noted that the groundwater level will fluctuate seasonally and in response to weather events. Under adverse conditions, water could be ponding on the ground surface under high flow conditions. It is reasonable to assume that groundwater could be similar to the water level in the swampy area west of the existing Peel Street North during high flow conditions.

PART B – FOUNDATION DESIGN

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of foundation design of the proposed new structural culvert north of Peel Street, based on our interpretation of the factual information obtained during this investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

It is proposed to construct a new structural culvert in conjunction with the Peel Street realignment north of Highway 26. Based on information supplied by Stantec, the new culvert will consist of a 4 m span by 1.3 m high by 29 m in length. The location of the proposed culvert is approximately 16 m north of the centerline of Highway 26, between STA 14+035.5 and 14+064.5. Photographs of this culvert site are presented in Appendix “D”.

The culvert site is located within a drainage ditch in which the ditch flows easterly. The approach embankments will be constructed on both the north and south sides of the culvert, with a maximum height of approximately 2.0 m. The embankment slopes will be typically 2.5H to 1V.

The proposed new culvert is located within a swampy area between STA 13+900 and 14+200 and spanning from the north-west towards the south-east at the Highway 26 and the Peel Street intersection. The ground surface at the location of the proposed culvert was covered with bulrushes and reasonably dry, but partly inundated at the existing ditch located north of the proposed culvert with water level of less than 0.5 m above the ground surface at the time of field work.

The existing ground surface Elevations range from 193.3 m to 193.9 m, with a proposed finish road grade of Elevation 195.25 m. The proposed invert of the culvert will be placed at Elevation 193.0±m.

Details of the proposed culvert are not known at the time of preparing this draft report. It is understood that the proposed culvert will consist of a precast concrete box culvert.

Based on the preliminary design drawing provided by Stantec, the proposed culvert will include construction of a new precast concrete box culvert beneath the new road alignment of Peel Street North, located west of the existing road platform. It is not known whether the existing twin CSP culverts will be removed or replaced at the time of preparing this draft report.

5.2 Closed Box Culvert

The new culvert should be designed to CAN/CSA-S6-06 and to withstand the appropriate weight of overfill, traffic loadings (CL-625-ONT), temporary construction loads and critical loading effects during construction. If the base slab does not have adequate frost cover/protection, it should be designed for frost pressures.

The proposed invert elevation of the new culvert will be placed at Elevation $193.0 \pm$ m. Based on a base slab thickness in the order of 300 mm and a bedding thickness of 400 mm, the bedding subgrade for the proposed new culvert will be placed on Elevation $192.3 \pm$ m.

The bedding subgrade of the proposed box culvert (near Elevation $192.3 \pm$ m) will consist of firm to stiff silty clay and very stiff layered silt and silty clay followed by dense to very dense sand and silt till. The subsurface soil deposit is considered capable of supporting the proposed box culvert, and can be design to a Factored Geotechnical Resistance of 250 kPa at ULS and a Geotechnical Reaction of 150 kPa at SLS.

Since the culvert site is located in a drainage ditch, localized soft pockets of silty clay could be encountered at the subgrade elevation. Sub-excavation and replacement with thickened bedding of up to an additional 0.6 m could be required for a precast concrete box culvert.

The SLS value given above is based on a maximum settlement of 25 mm. This can be achieved provided the founding subgrade is not disturbed during construction.

As per CAN/CSA-S6-06, Clause 1.9.5.6, a cut-off wall of sufficient depth and strength shall be provided at the ends of the culvert to prevent undermining. The depth of the cut-off wall should be designed cognizant of the hydraulic condition (CAN/CSA-S6-06, Section 1.9) and the frost depth of 1.4 m (OPSD 3090.101).

Foundation preparation for cast-in-place construction should be carried out in accordance with Sub-section 902.07.05.02 of OPSS 902 and Sub-section 902.07.02.02 of SSP902S01.

A 300 mm thick OPSS Granular "A" bedding and a 75 mm thick levelling granular course as per OPSS422, should be placed on the prepared subgrade to achieve a uniform support for a precast concrete culvert. The Granular "A" bedding should be compacted to 98% of the material's standard Proctor maximum dry density (SPMDD). The levelling course should consist of OPSS 1002 fine aggregates (concrete sand) and uncompacted.

5.3 Open Footing Culvert (Spread Footing Foundations)

Based on the borehole results, spread footings may be used for the culvert walls and wing walls, and designed to bear on the undisturbed, firm to stiff, silty clay at the following highest elevations and designed for bearing resistances shown below:

Borehole	Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
31A-1	192.2	250	150
31A-2	193.0	250	150
31A-3	192.3	250	150

The SLS value given above is based on a maximum settlement of 25 mm. This can be achieved provided the founding subgrade is not disturbed during construction.

The depth of the footings should be designed cognizant of the hydraulic condition (CAN/CSA-S6-06, Section 1.9) and the frost depth of 1.4 m (OPSD 3090.101).

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with Clause 6.7.4 of CAN/CSA-S6-06. It is noted that the footings should be founded below the anticipated local and general scour depths.

Immediately upon excavation, the exposed subgrade should be inspected and approved by the Geotechnical Engineer.

5.4 Engineered Fill

The excavations created by removing the localized soft materials below the proposed footings the Open Footing Culvert and below the bedding subgrade for the Closed Box Culvert will need to be backfilled with engineered fill.

Preparation for engineered fill construction should consist of removing all deleterious materials (organics and fill if encountered) to expose the native stiff silty clay subgrade. The engineered fill subgrade should be inspected and approved by the Geotechnical Engineer, or a Quality Verification Engineer (QVE) as per SSP199S48, prior to placement of the engineered fill.

The engineered fill should consist of OPSS Granular "B" Type I materials. It should be placed and compacted in thin lifts to 100% of the material's standard Proctor maximum dry density (SPMDD), as determined using Method A of OPSS 501.08.02. The lift thickness of the engineered fill should be limited to between 150 mm and 300 mm depending on the compaction equipment used, as determined in the field by the Geotechnical Engineer. The engineered fill should be compacted under the full time supervision of the Geotechnical Engineer or a QVE. Compaction tests should be carried out on each lift of fill placed to confirm that the specified degree of compaction has been achieved. Subsequent lifts of fill should not be placed until the specified degree of compaction of the current lift is achieved. A certificate of conformance shall be provided to the Contract Administrator as per the requirements of SSP199S48.

Alternatively, the engineered fill could consist of Type II 19.0 mm crushed stone placed on a Type 1 non-woven geotextile such as Terrafix 270R or equivalent. The crushed stone should be placed in 150 mm thick lifts and tamped to its tightest state under the full time supervision of the Geotechnical Engineer or the QVE.

The footings for the culvert should be designed to a SLS at 150 kPa and ULS at 250 kPa as the underlying silty clay stratum is the controlling supporting material.

5.5 Lateral Earth Pressures

The lateral earth pressures acting on the culvert walls, headwalls (wing walls), and retaining walls (reinforced concrete, armour stone or gabion etc.) will depend on the type and method of placement of the backfill materials and on the subsequent lateral movement of the structure whether it is restrained or unrestrained. The lateral earth pressures to be used in the design should be computed in accordance with Section 6.9 of the CAN/CSA-S6-06.

Granular backfill should be constructed behind the culvert walls, headwalls (wing walls), and retaining walls as per OPSD-3121.150, with particular attention to the frost taper requirement. The granular backfill should conform to OPSS 1010 for either Granular "A" or Granular "B" Type III. To maintain free draining characteristics in granular fill materials, the maximum percentage passing the No. 200 sieve (75 μ m) should be limited to 5%.

The backfill should be constructed as per OPSS 902 and 501, and SSP902S01. A perforated subdrain should be installed behind the walls with a positive outlet or wall drains as per OPSD-3190.100 to drain the granular fill above the stream water level. Alternatively, the culvert walls could be designed to resist hydrostatic pressure.

The lateral earth pressure, P_h , acting on the headwalls (wing walls), or retaining walls may be computed using the equivalent fluid pressures presented in Clause 6.9.2.3 of the CAN/CSA-S6-06, or employing the following equation based on unfactored earth pressure distributions:

$$P_h = K (\gamma h + q)$$

Where:

K = earth pressure coefficient, use value from table below

γ = unit weight of soil, = 21.2 kN/m³ for Granular "B"
= 22.8 kN/m³ for Granular "A"

h = depth below top of wall, m

q = live load surcharge pressure, equivalent fill height of 0.8 m
as per Clause 6.9.5 of CHBDC and CAN/CSA-S6-06

Wall Type	Earth Pressure Coefficient (K)	
	Granular "A" $\phi = 35^\circ$	Granular "B" $\phi = 30 \text{ to } 35^\circ$
Restrained Wall (K_o)	0.43	0.50 to 0.43
Unrestrained Wall (K_a)	0.27	0.33 to 0.27

The submerged unit weight of the backfill should be used for any submerged portion of the granular backfill when calculating the lateral earth pressure.

The above parameters are based on a horizontal back slope (not exceeding 5 degrees) behind the headwalls. A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the headwalls and retaining walls in accordance with Clause 6.9.3 of the CAN/CSA-S6-06.

The sliding resistance of the cast-in-place footings should be checked. The unfactored horizontal resistance (Clause 6.7.5, CAN/CSA-S6-06) against sliding between concrete and undisturbed, silty clay can be calculated using a coefficient of friction (friction factor) of 0.40 for concrete on silty clay soils as per Table 24.4 CFEM 4th Edition, 2006.

For a precast concrete culvert, the friction factor and adhesion should be reduced by a factor of 0.67.

Vibratory equipment for use behind the culvert walls, headwalls (wing walls) and retaining walls should be restricted in size as per current MTO practices, and should conform to OPSS 501 and SSP105S10.

5.6 Retained Soil System (RSS) or Concrete Wing Walls

It is not known if wing walls, gabion walls or Retained Soils System (RSS) will be required at this juncture along the toe of the embankment and adjacent to the culvert. If required, the wing walls, gabion walls or RSS walls should be designed in accordance with the parameters given in Section 5.4. The supplier of the RSS walls should be responsible for design of the structure such as backfill, reinforcement, and internal and external stability. Details of the wing walls, gabion walls or RSS walls have not been finalized at the time of preparing this report. The following information should be included in the contract drawing:

- length and location
- height and space constraints
- elevation of top and bottom of RSS
- performance requirement
- appearance requirement

Foundations for the wing walls and RSS walls should be designed to bear on the native, undisturbed, stiff to very stiff silty clay, at or below the highest elevations, and with the corresponding bearing resistances given in the following table:

Borehole	Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
31A-1	192.2	250	150
31A-2	193.0	250	150
31A-3	192.3	250	150

The SLS value given above is based on a maximum settlement of 25 mm. This can be achieved provided the founding subgrade is not disturbed during construction.

The sliding resistance of the wing walls should be checked. It is assumed that a Granular A bedding layer, compacted to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD), will be placed beneath the wing wall foundations. The unfactored horizontal resistance (Clause 6.7.5, CAN/CSA-S6-06) against sliding between the concrete and bedding, can be calculated using a coefficient of friction (friction factor) of 0.55 as per Table 24.4 CFEM 4th Edition, 2006. The unfactored horizontal resistance against sliding between concrete and undisturbed, competent silty clay founding soils can be calculated using an adhesion of 50 kPa, or alternatively a coefficient of friction of 0.40.

The backfill to be used for the wing walls, gabion walls or RSS walls will likely consist of imported, free-draining Granular B material with soil unit weights and earth pressure coefficients provided in Section 5.5.

5.7 New Embankment

The new embankments are up to 2 m high adjacent to the existing culvert. For construction of the new embankment, the surficial topsoil and any deleterious materials should be stripped or excavated prior to placing fill materials. The embankment widening should then be constructed as per OPSD-202.010, 202.030 and 208.010, with emphasis on adequate benching of the subgrade for receiving the embankment fill. The fill to be used for embankment construction can either be imported silty clay or granular materials, but granular materials are preferred for compaction and drainage.

Backfill adjacent to the structure should be carried out in conformance with OPSS 902, SSP902S01 and OPSD-3121.150, and the fill should be placed and compacted in accordance with OPSS 501 and SSP105S10.

Based on the findings of the field investigation, no foundation stability or settlement problems due to on the native silty clay, with anticipated embankment slope of 2.5H:1V and up to 2 m high. The fill placement should begin at the toe of the embankment, in leveled lifts and each lift

compacted to at least 98% SPMDD. Benching into the existing embankment slope at 1 m high steps is recommended as per OPSD 208.010.

After stripping, the exposed subgrade should be inspected and approved by the geotechnical engineer. The approved subgrade should then be proof-rolled using a heavy compactor, as directed by the engineer. Unless the excavation is carried out in wet weather conditions, no unusual dewatering is anticipated during stripping and preparation of the subgrade to receive the embankment fills. Where necessary, dewatering can be carried out using gravity drainage and pumping from open filtered sumps in accordance with OPSS 517 and 902, and SSP902S01, with emphasis on the requirements of OPSS 518.

Measures should be incorporated into the design and staging to ensure that the slope surfaces are protected from surface erosion in accordance with the requirements of OPSS 577. Proper erosion control measures should be implemented both during construction of the embankment fills and permanently. Erosion control during construction should be carried out by installing silt fences. Properly designed erosion control blankets could also be placed on any new embankments and adjacent disturbed embankments after completion of fill placement. A vegetative cover should be established as soon as practical upon completion of fill placement to minimize the chances of surface erosion.

Revetments such as rip-rap blanket should be provided at the toe of the slope and the ends of the culvert to prevent erosion/scour by stream action in accordance with OPSS 511 and OPSD 810.010. The design of the rip-rap blanket should be carried out cognizant of the stream hydraulics.

5.8 Excavation, Groundwater Control and Temporary Shoring

Excavation for this project will involve the construction of the box culvert or footings for the culvert walls, headwalls (wing walls) and retaining walls. Depending on the design that is finally selected, the anticipated maximum depth of excavation below the existing grade of Peel Street realignment is in the order of 2 m.

Excavation to depths of up to 2 m should not present any special difficulties using heavy excavation equipment, provided it is constructed in accordance with OPSS 501, 517, 518, 539, 577 and 902, SSP902S01 and OPSD-803.010 and 3121.150. However, the buried utilities alongside the embankments will likely be in conflict with the excavation. Excavation and protection procedures shall conform to OPSS 539 and should be reviewed with the utility companies or authorities prior to construction. Based on the subsurface soil and groundwater conditions encountered at this site, a Permit to Take Water (PTTW) in accordance with Ontario Regulation 387/04 will not be required for the purpose of excavation.

The water in the ditch can be controlled by using a CSP for temporary flow passage, temporary diversion or dam and pump method. The anticipated minor groundwater ingress can be controlled using intercept ditches and pumping from filtered sump pits.

It is noted that a “Permit To Take Water” (PTTW, Regulation 387/04) will be required from the MOE (Ministry of Environment) when the total quantity of water to be handled exceeds 50,000 litres/day while employing temporary pumping of water, flow passages through culverts, stream diversion or dam and pump method as groundwater control measures (unwatering). It may take up to 90 days for MOE to review an application and issue a permit.

All excavation must be carried out in compliance with the requirements of the Occupational Health and Safety Act (OHSA). For this purpose, the unsaturated upper fill materials and the firm to stiff silty clay is classified as Type 3 soils. Saturated fill materials should be classified as Type 4 soils.

Within the Type 3 soil, the excavation shall be cut to no steeper than 1H :1V throughout. Side slopes of 3H:1V or flatter shall be used for excavation within Type 4 soils.

The exposed stiff silty clay and very stiff layered silt and silty clay could easily be disturbed upon excavation. In this regard, a 50 mm thick layer of lean concrete should be placed on the foundation subgrade immediately after excavation and approval by the Geotechnical Engineer for any cast-in-place concrete construction.

Temporary support will not be required for construction of a new culvert and road realignment.

5.9 Frost Protection

This project is located in the Owen Sound Operations District. The design frost penetration depth for this project is 1.4 m in accordance with OPSD 3090.101. All foundations and spread footings should be provided with at least 1.4 m of soil cover for adequate frost protection. Alternatively, frost protection can be provided by equivalent thermal insulation.

5.10 Scour Depth

The footings should be founded below the anticipated local and general scour depths as per CAN/CSA-S6-06, Clause 1.9, Hydraulic Design; and CHBDC (2006) - Section 1.9. The native silty clay that will be exposed at the streambed is considered to have a permissible velocity¹ of 1.5 m/s.

¹ U.S. Department of the Army, Corps of Engineers, Hydraulic Design of Flood Control Channels, Engineering Manual EM 1110-2-1601

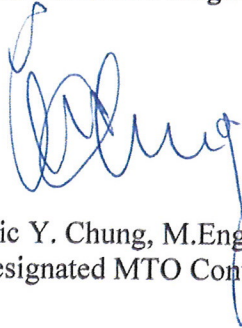
6.0 STATEMENT OF LIMITATION

We recommend that once the details of the proposed structure are finalized, our recommendations should be reviewed for their specific applicability.

The Limitations of Report, as Quoted in Appendix "C", is an integral part of this report.

We trust that we have completed the assignment within the Terms of Reference for this project. If there are any questions concerning this report, please do not hesitate to contact our office.

Yours truly,
Infrastructure Engineering Group Inc.



Eric Y. Chung, M.Eng., P.Eng.
Designated MTO Contact



Joseph Law, P.Eng.
Project Manager



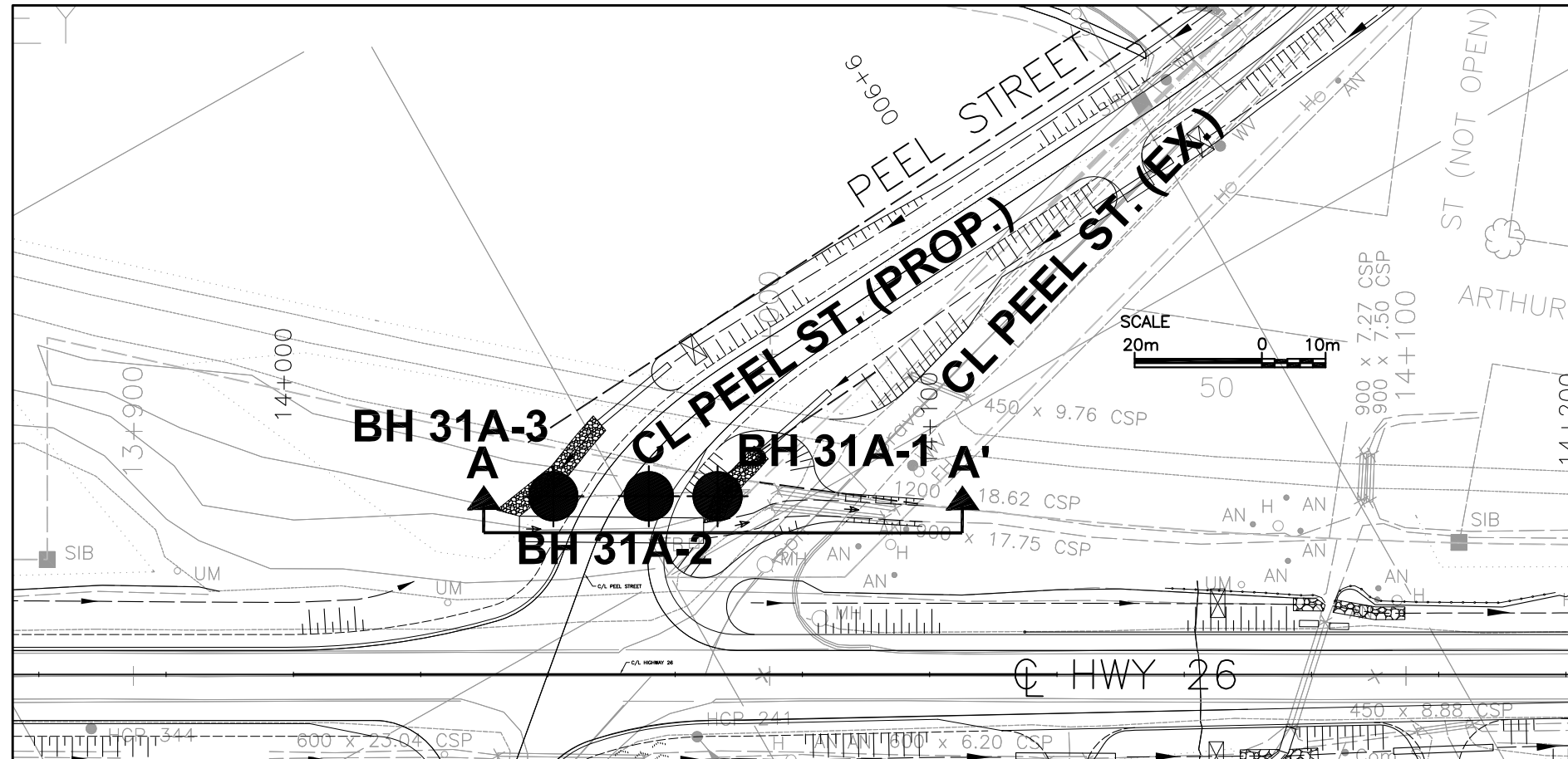
Tom O'Dwyer, P. Eng.
Quality Review Engineer



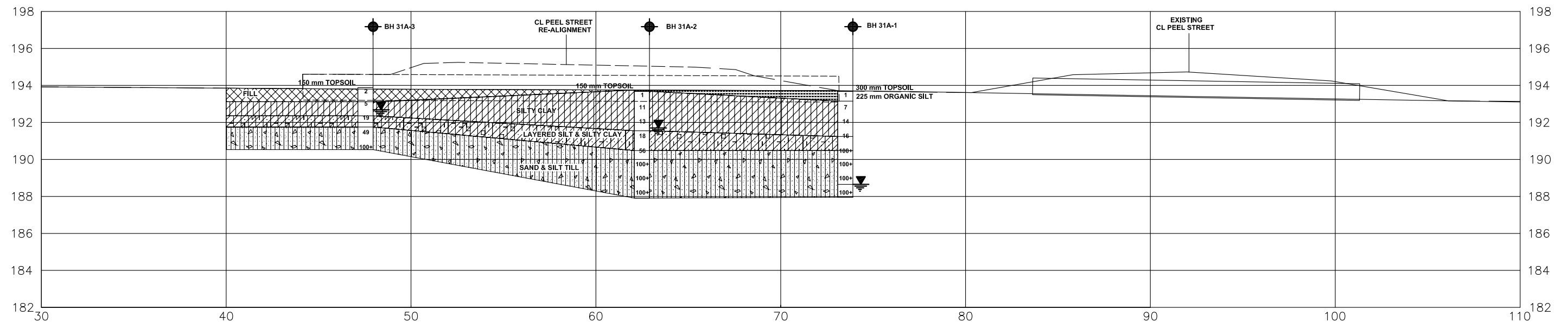
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08-1-IEG1-PEEL
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Drawing 1
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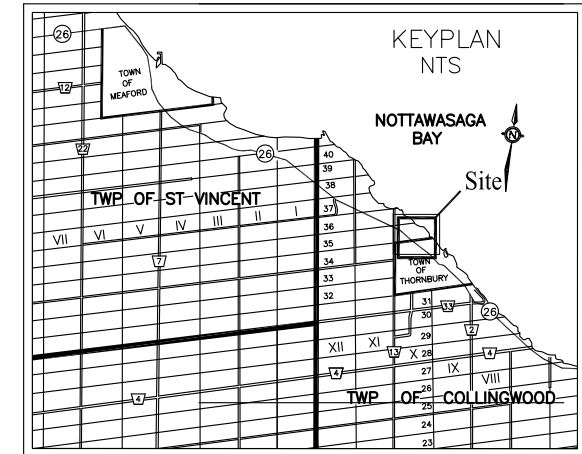
Drawing 1
Borehole Locations
And
Soil Strata



BOREHOLE LOCATION PLAN

SECTION A-A'
CENTERLIND OF CULVERT

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWNCONT No xxxx-xxxx
WP No GWP 57-00-00PEEL STREET NORTH NEW CULVERT
Highway 26
BORE HOLE LOCATIONS & SOIL STRATASHEET
1I.E. Infrastructure Engineering Group Inc.
Pavement & Construction Materials Consulting Engineers
GTA • Kitchener • London • Windsor

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- Blows/0.3m (Std Pen Test, 475 J/blow)
- Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
- Standpipe

NOTES

- THE COMPLETE FOUNDATION INVESTIGATION AND DESIGN REPORT FOR THIS PROJECT AND OTHER RELATED DOCUMENTS MAY BE EXAMINED AT THE ENGINEERING MATERIALS OFFICE, DOWNSVIEW. INFORMATION CONTAINED IN THIS REPORT AND RELATED DOCUMENTS ARE SPECIFICALLY EXCLUDED IN ACCORDANCE WITH THE CONDITIONS OF SECTION GC2.01 OF OPS GEN. COND.
- THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES AND BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES		HWY No. HWY 26			DIST	Owen Sound	
		NORTH	EAST						
31A-1	193.69	4936907	228113	SUBM'D	J.L.	CHECKED E.C.	DATE 27/07/09	SITE	Peel Street North
31A-2	193.68	4936913	228103						New Culvert
31A-3	193.88	4936920	228090	DRAWN	J.L.	CHECKED J.L.	APPROVED	E.C.	DWG 1

REVISIONS	DATE	BY	DISCRIPTION	Geocres : 41A-209
25/11/09	J.L.	Final		
27/07/09	J.L.	Draft		

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Appendix A

Explanation of Terms Used in Report

Record of Borehole Sheet

Boreholes C31A-1 to C31A-3

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 1" 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

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

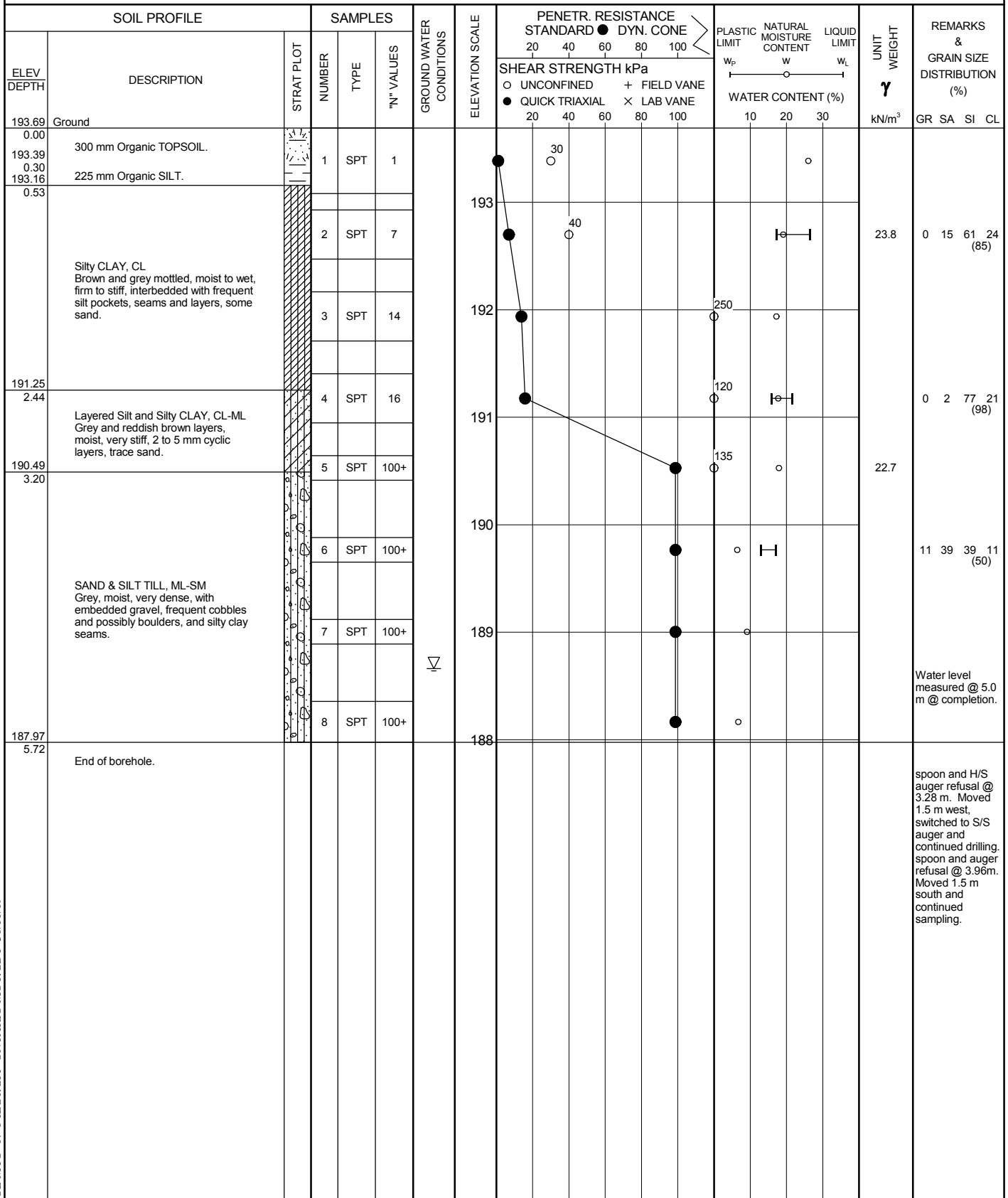
ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1. %	VOID RATIO	e_{min}	1. %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1. %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1. %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT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

RECORD OF BOREHOLE No 31A-1

1 OF 1

METRIC

W.P. GWP 57-00-00 LOCATION HWY 26, Thornbury to Meaford Northing - 4936907, Easting - 228113 ORIGINATED BY JL
DIST Owen Sound HWY 26 BOREHOLE TYPE S/S Augering, 110 mm dia., H/S Augering, 110 mm diam. ID COMPILED BY JL
DATUM Geodetic DATE 9.7.09 - 9.7.09 CHECKED BY EC



JOE MTO 07-6-IEG1.GPJ ONTARIO MOT.GDT 30/11/09

+ 3, X 3: Numbers refer to Sensitivity

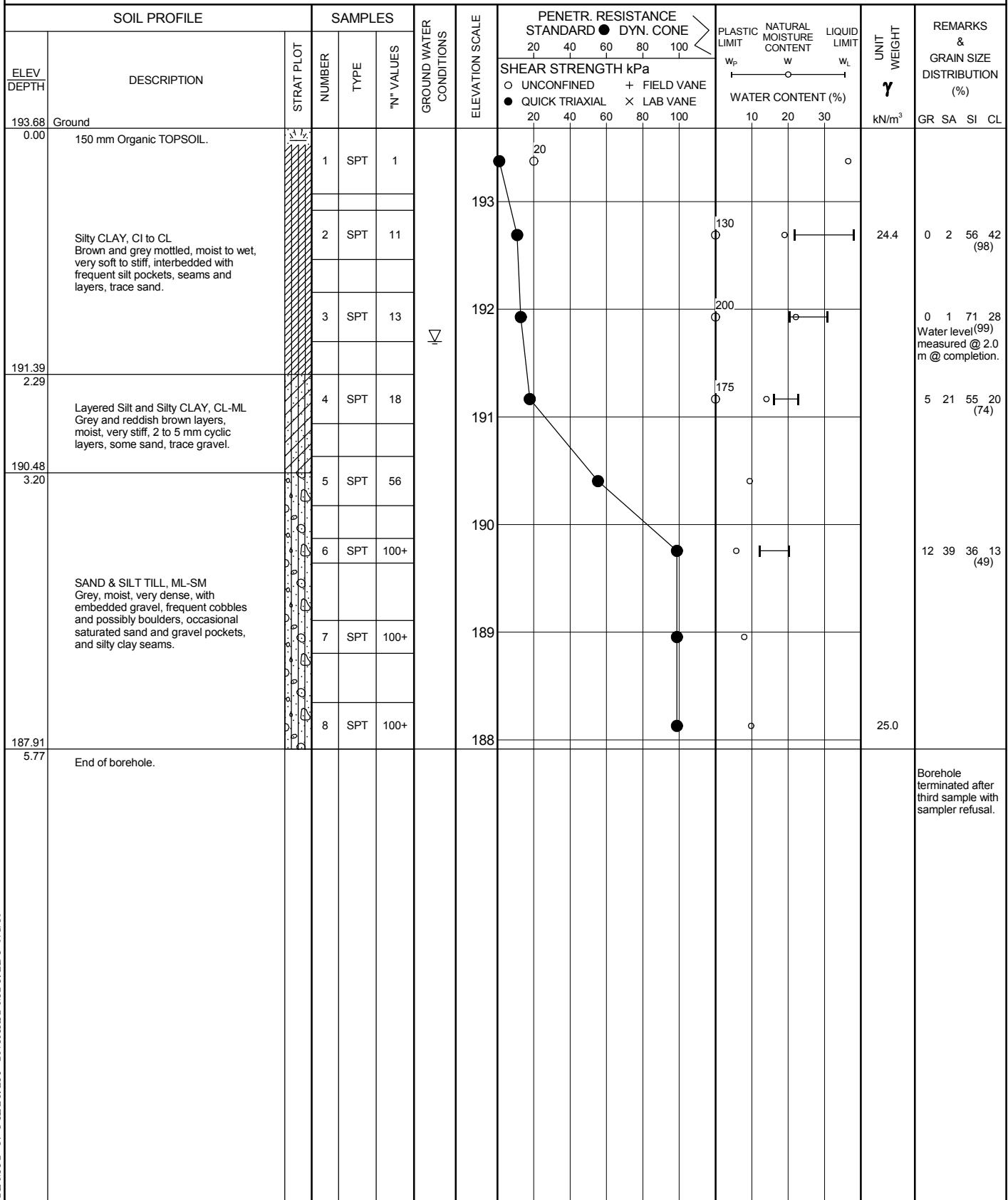
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 31A-2

1 OF 1

METRIC

W.P. GWP 57-00-00 LOCATION HWY 26, Thornbury to Meaford Northing - 4936913, Easting - 228103 ORIGINATED BY JL
 DIST Owen Sound HWY 26 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY JL
 DATUM Geodetic DATE 9.7.09 - 9.7.09 CHECKED BY EC



+ 3, X 3: Numbers refer to Sensitivity

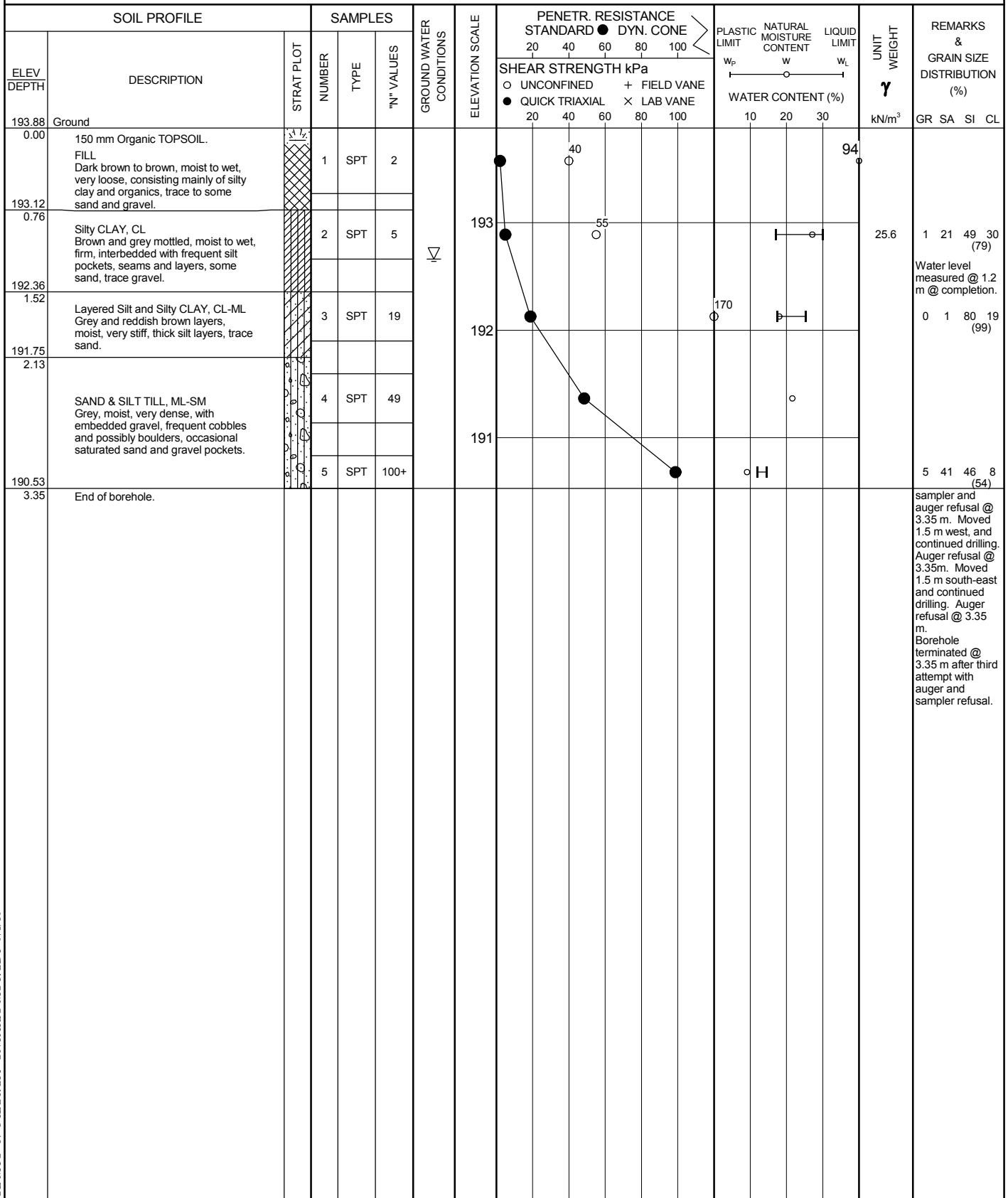
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RECORD OF BOREHOLE No 31A-3

1 OF 1

METRIC

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 DIST Owen Sound HWY 26 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY JL
 DATUM Geodetic DATE 9.7.09 - 9.7.09 CHECKED BY EC



JOE MTO 07-6-IEG1.GPJ ONTARIO MOT.GDT 9/8/09

+ 3, X 3: Numbers refer to
Sensitivity

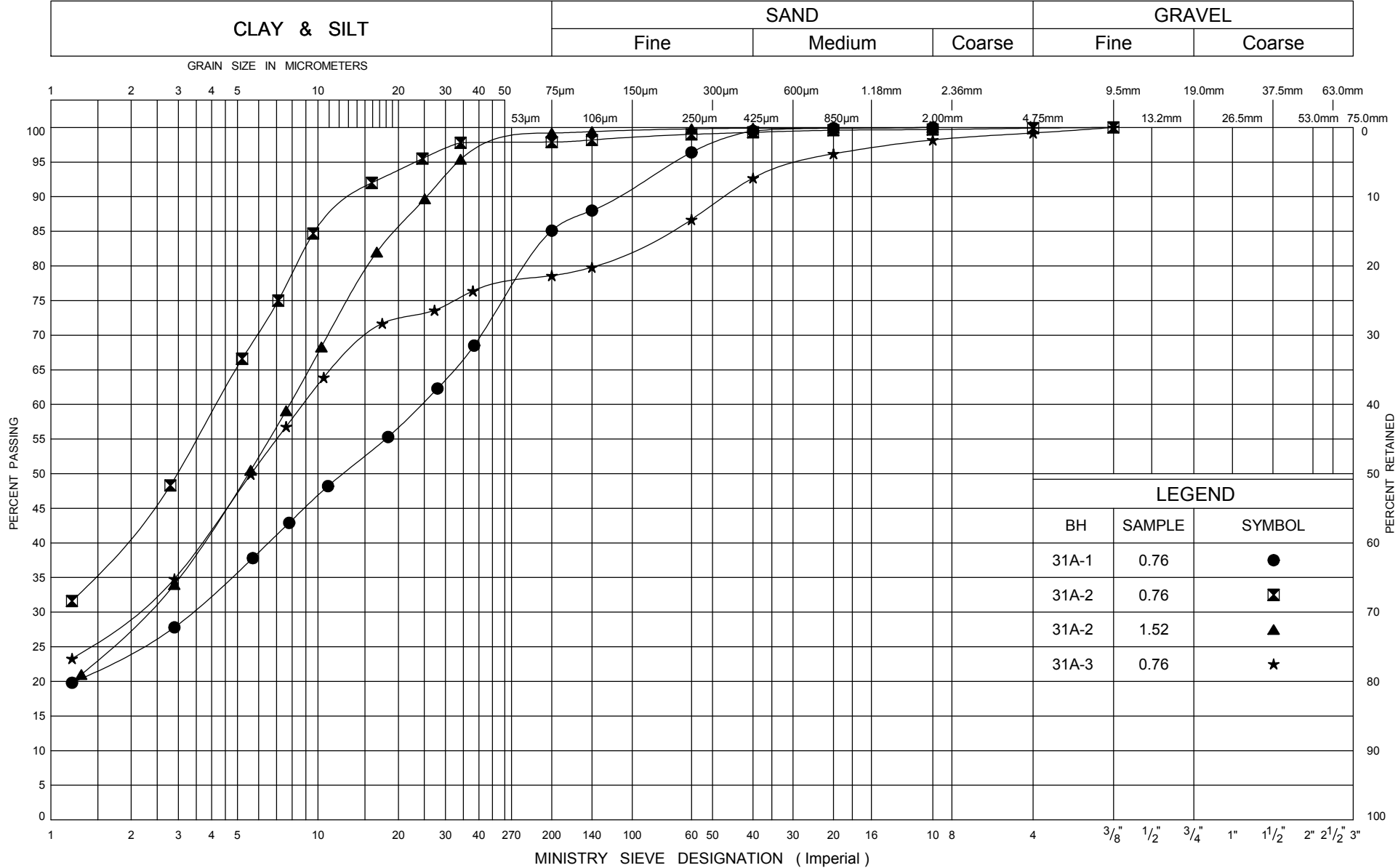
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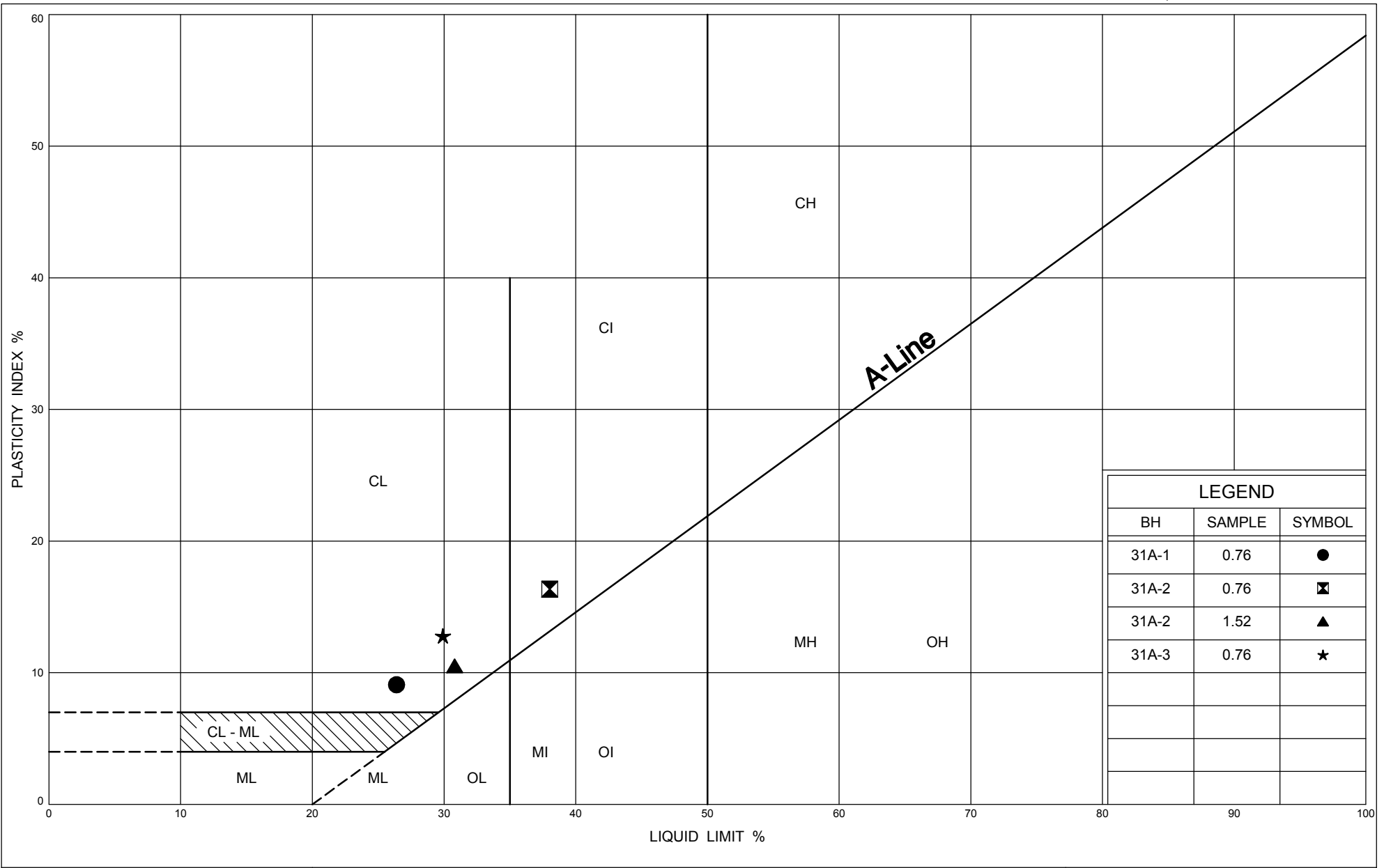
Appendix B

Laboratory Test Results

Grain Size Distribution	Figures 1, 3 and 5
Plasticity Chart	Figures 2, 4 and 6

UNIFIED SOIL CLASSIFICATION SYSTEM





Ministry of
Transportation

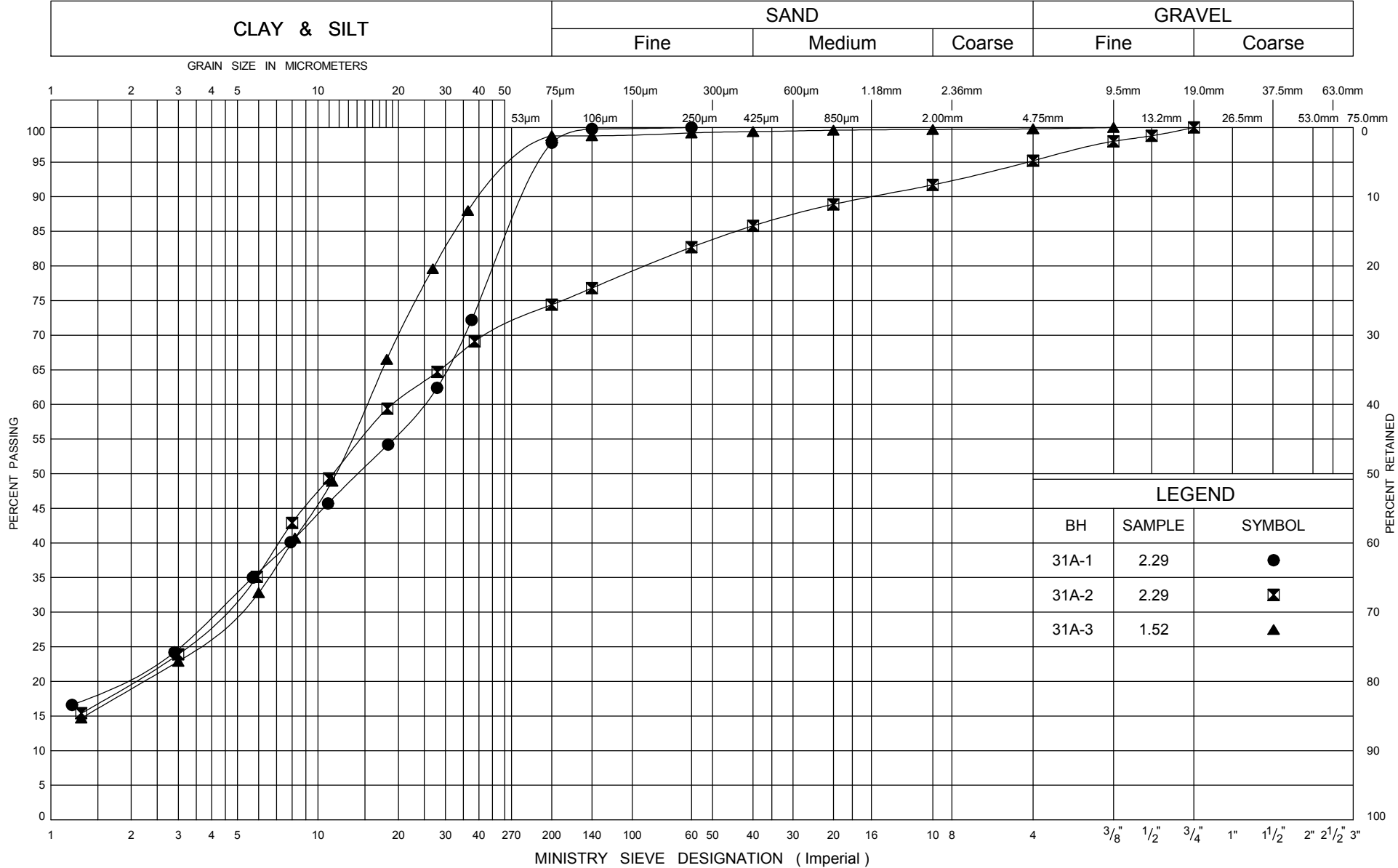
PLASTICITY CHART SILTY CLAY, CL TO CI

FIG No 2

GWP 57-00-00

HWY 26, Thornbury to Meaford

UNIFIED SOIL CLASSIFICATION SYSTEM

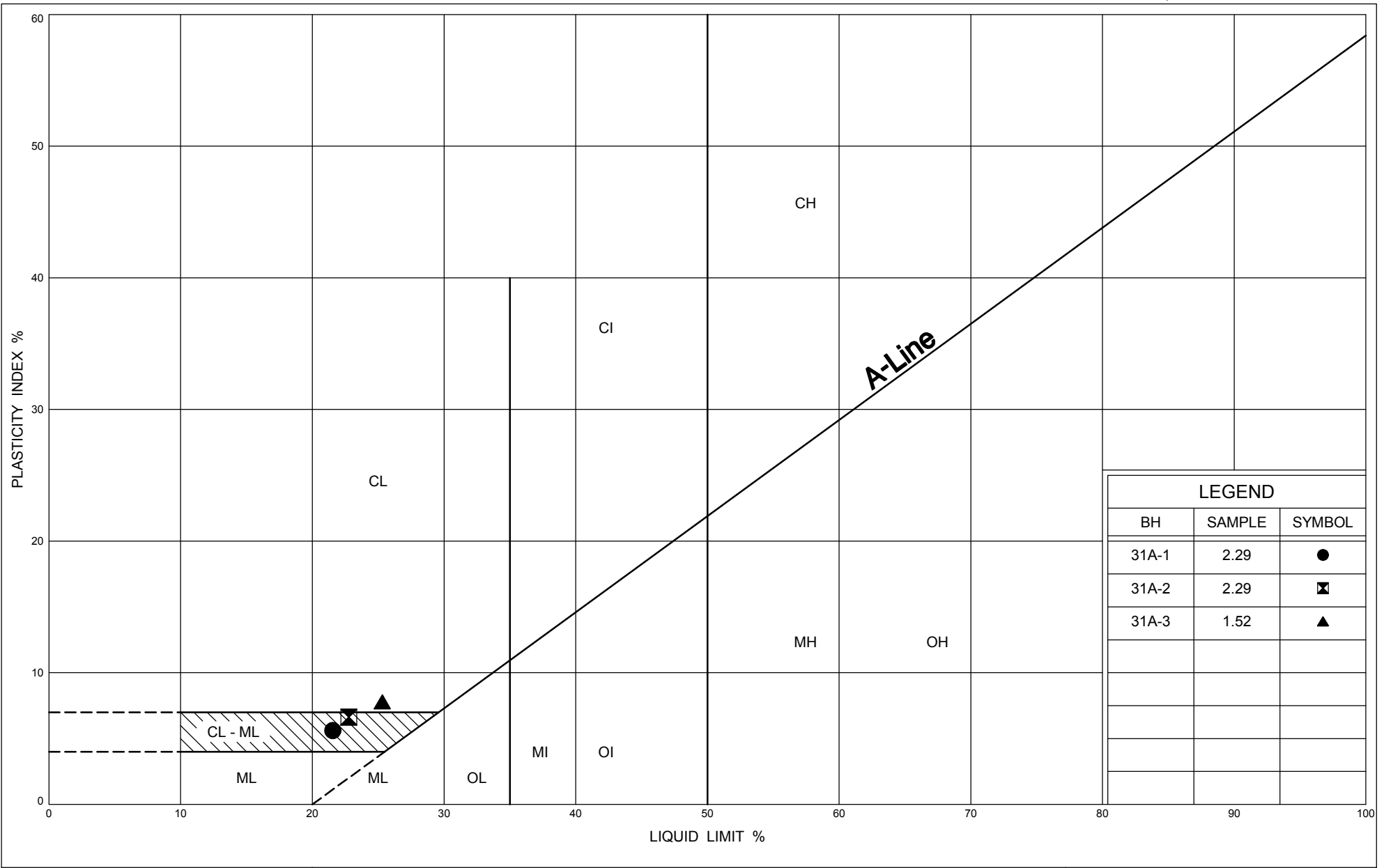


ONTARIO MOT GRAIN SIZE LARGE CULVERTS 07-6-IEG1.GPJ ONTARIO MOT.GDT 30/7/09

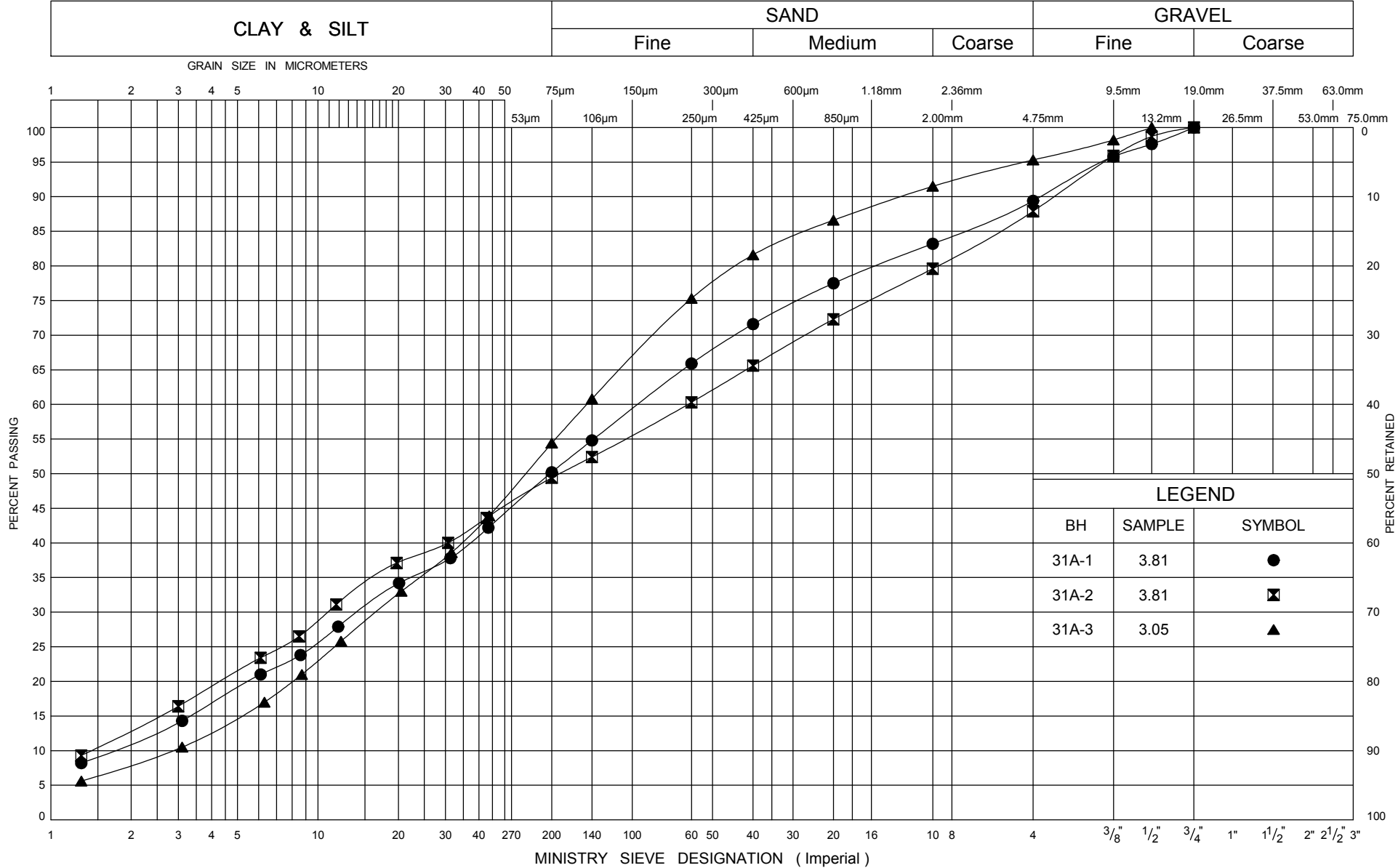


GRAIN SIZE DISTRIBUTION
LAYERED SILT AND CLAY, CL-ML

FIG No 3
GWP 57-00-00
HWY 26, Thornbury to Meaford



UNIFIED SOIL CLASSIFICATION SYSTEM

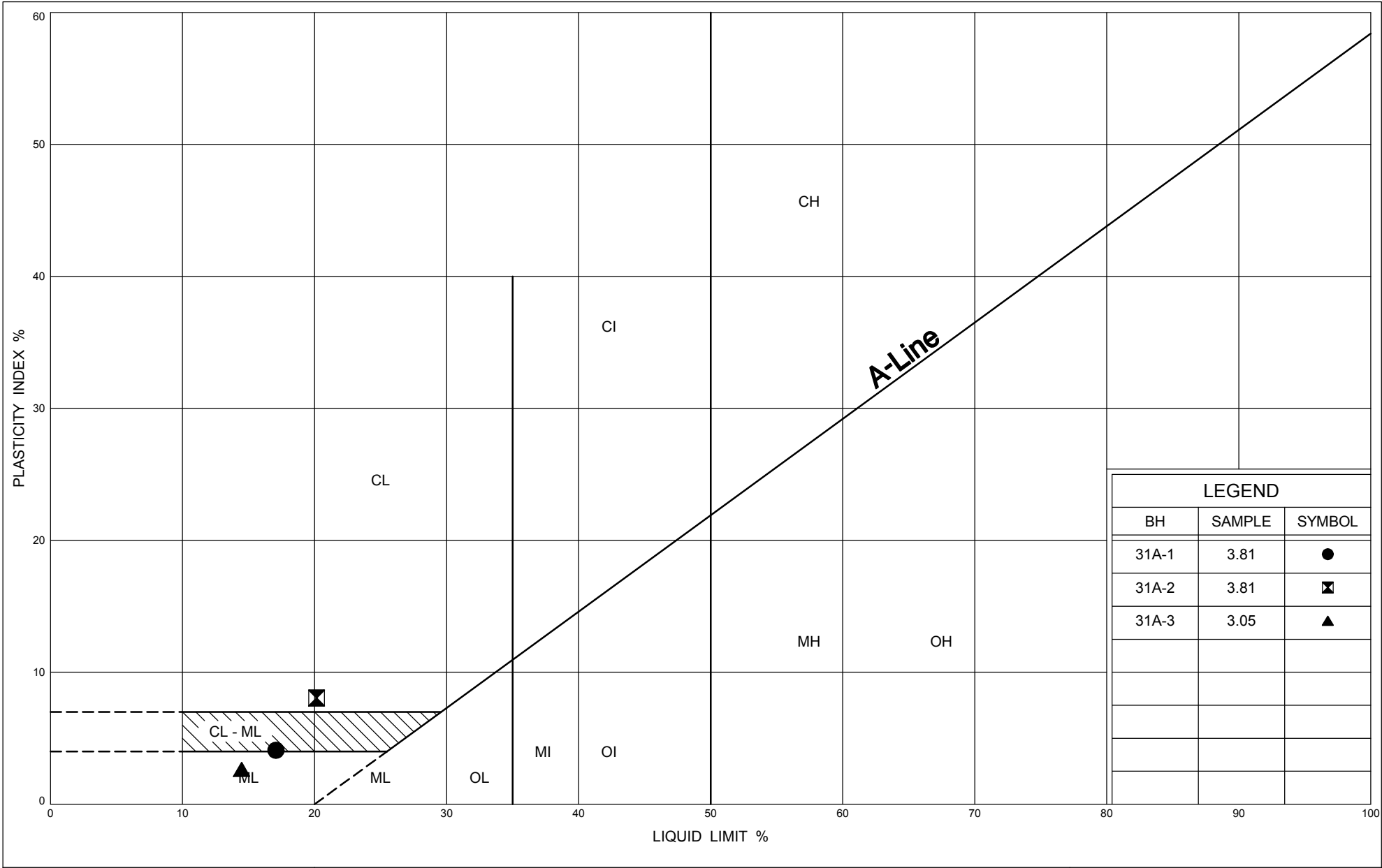


ONTARIO MOT GRAIN SIZE LARGE CULVERTS 07-6-IEG1.GPJ ONTARIO MOT.GDT 30/7/09



GRAIN SIZE DISTRIBUTION
SAND & SILT TILL WITH SILTY CLAY SEAMS, ML-SM

FIG No 5
GWP 57-00-00
HWY 26, Thornbury to Meaford



ONTARIO MOT PLASTICITY CHART LARGE CURVE 07-6-IEGI.GPI ONTARIO MOT GDT 30/7/09



Ministry of
Transportation

PLASTICITY CHART

SAND & SILT TILL WITH SILTY CLAY SEAMS, ML-SM

FIG No 6

GWP 57-00-00

HWY 26, Thornbury to Meaford

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Appendix C

Limitations of Report

APPENDIX C

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Soils Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

The benchmark and elevations mentioned in this report were obtained strictly for use in the geotechnical design of the project and by this office only, and should not be used by any other parties for any other purposes.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Infrastructure Engineering Group Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

This report does not reflect the environmental issues or concerns unless otherwise stated in the report.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, IEG recommends that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

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Appendix D
Site Photographs



Existing Peel Street looking west towards realignment



Existing Peel Street looking north-west at proposed culvert site



Proposed Peel Street North Structural Culvert Site looking south



Existing Peel Street North non-structural twin culverts inlet